BRITISH STANDARD

BS 5975:1996

Incorporating Amendments Nos. 1 and 2

Code of practice for falsework

ICS 91.220

British Standards

Committees responsible for this British Standard

The preparation of this British Standard was entrusted to Technical Committee B/514, Access and support equipment, upon which the following bodies were represented:

Association of Building Component Manufacturers
Association of Consulting Engineers
British Constructional Steelwork Association Ltd.
British Iron and Steel Producers Association
Building Employers Confederation
Concrete Society
Construction Fixings Association
Construction Health and Safety Group
Construction Industry Training Board
Convention of Scottish Local Authorities
Federation of Civil Engineering Contractors
Health and Safety Executive
Institute of Building Control
Institution of Civil Engineers
Institution of Structural Engineers
National Association of Scaffolding Manufacturers' Association
Suspended Access Equipment Manufacturers' Association
Co-opted members

This British Standard, having been prepared under the direction of the Sector Board for Building and Civil Engineering, was published under the authority of the Standards Board and comes into effect on 15 March 1996

First published March 1982
Second edition March 1996

The following BSI references relate to the work on this British Standard
Committee reference B/514
Draft for comment 90/11741 DC

ISBN 0 580 24949 2

Amendments issued since publication

Amd. No.	Date	Comments
9289	December 1996	
15090	10 December 2004	Foreword amended

Contents

Foreword

This edition of BS 5975 (BS 5975:1996) was prepared by Technical Committee B/514 and supersedes BS 5975:1982, which has been withdrawn. It introduces technical changes but does not constitute a full revision of BS 5975:1982.

Amendment No. 2 has been occasioned by the publication of BS EN 12812:2004. Technical Committee B/514 intends to fully revise BS 5975:1996 to take account of changes in legislation and practice within the next two years.

BS 5975:1996 provides recommendations on the design of falsework using permissible stress methods and on procedures for the successful management of work on site, including the appointment of a falsework coordinator. These recommendations are re-affirmed by Technical Committee B/514.

BS EN 12812:2004 now exists in parallel with this standard and specifies performance requirements for the design of falsework in accordance with one of three classes: A, B1 and B2. Limit state design methods are specified for design classes B1 and B2. It does not provide guidance for the structural design of Class A.

BS EN 12812:2004 does not provide guidance on procedures necessary for the successful management of work on site. The recommendations of the Advisory Committee on Falsework (the "Bragg Report"[1] [2])[1)] in respect of the falsework coordinator have not been included in it.

The need for authoritative guidance on falsework has been growing with the increase in scale, frequency and complexity of its applications. Many organizations have arisen in the industry to provide specialist services, and a greater reliability of performance has been sought, preferably without a cost penalty. These factors have led to the publication of this code of practice. The need for such information was also illustrated by the "Bragg Report". Prior to the preparation of this code, no previous code was known to exist anywhere in the world; during the drafting stages the main document drawn upon was thus the "Falsework Report" of the Joint Committee of the Institution of Structural Engineers and the Concrete Society [3].

This code has drawn together all those aspects that need to be considered when preparing a falsework design, and in so doing has included recommendations for materials, design and work on site. Because the success of falsework is closely tied up with its management, this code describes procedures as well as technical aspects. Recommendations are given in Section 2 on the actions that ought to be taken, and on possible ways of allocating the duties to individuals. The "Bragg Report" recommended that the duty of ensuring that all the relevant procedures and checks had been carried out be given to one individual in the construction organization, such an individual being known as the "Temporary Works Coordinator". This code endorses such action but has adopted the narrower term "falsework coordinator", because this code is concerned only with falsework and does not consider any of the other activities usually covered by the general term "temporary works", such as excavations. A full description of the duties of the falsework coordinator will be found in Section 2.

This new edition of the code brings it into line with BS 5268-2. The original document published in 1982 anticipated many of the changes incorporated into BS 5268, but a number of minor differences have arisen, which are now corrected. This code adopted two special factors for shear and compression, to bring stresses into line with common acceptable falsework practice, and these are retained. Also retained are the strength classes of BS 5268, but with minor differences to the values.

[1)] The numbers in square brackets used throughout the text of this code relate to the bibliographic references in Annex N.

This code provides guidance on the accuracy of construction required in order to be able to adopt the recommended design approaches. While there is adequate information available upon which to base the recommendations for the deviations associated with the design parameters adopted for props [7, 8], in other areas the information is limited, consequently the committee would welcome any feedback on this point.

Users of this code are reminded that it may be necessary for them to appraise third parties with whom they are in contractual relations of certain provisions in the code.

Users of this code should be in possession of the Concrete Society document *Formwork — A guide to good practice* [12], the CIRIA Report No. 108 [4] and Concrete Society Technical Report No. 5 [11].

Forces exerted by breaking waves can be large and have to be considered during the design of any structure which may be subjected to such conditions. Until such time however, as methods of calculating forces exerted on vertical walls by breaking waves can be recommended with confidence such problems should be referred to an engineer experienced in the design and construction of structures subject to wave attack.

CP3: Chapter V-2 will be superseded on publication of BS 6399-2.

The implications of this different presentation are being examined and an appropriate amendment will follow in due course. It now provides information in accordance with BS 1139-1.1, BS 1139-2.1 and BS 1139-2.2. Because of the large amount of equipment still in use, it also retains information in accordance with the corresponding 1982 editions.

This standard does not introduce limit state design, however, a number of points relating to steel have been changed to bring it into line with current practice for steel, in particular BS 449-2, including all amendments.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 141 and a back cover.

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Section 1. General

1.1 Scope

This code of practice gives recommendations for the design and use of falsework on construction sites. The definition of falsework given in **1.3.9** is broad, but this code does not give information on the support of excavations (see BS 6031) nor does it describe flying shores or the like (see CP 2004). For guidance on the design and use of access scaffolding and special scaffold structures other than falsework, reference should be made to BS 5973.

A considerable amount of falsework is of comparatively low technology, and very repetitive. Section 8 provides information to deal with a limited number of such cases, described as "standard solutions"' Guidance for producing comparable data for similar situations using, for example, particular items of proprietary equipment is also provided.

1.2 References

1.2.1 Normative references

This British Standard incorporates, by dated or undated reference, provisions from other publications. These normative references are made at the appropriate places in the text and the cited publications are listed on pages 134, 135 and 136. For dated references, only the edition cited applies; any subsequent amendments to or revisions of the cited publication apply to this British Standard only when incorporated in the reference by amendment or revision. For undated references, the latest edition of the publication referred to applies together with any amendments.

1.2.2 Informative references

This British Standard refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed in Annex N and on page 134, but reference should be made to the latest editions.

1.3 Definitions

For the purposes of this code, the following definitions, together with those given in Annex F, apply.

1.3.1 baseplate

A metal plate with a spigot for distributing the load from a standard, raker or other load-bearing member.

1.3.2 bay length

The distance between the centres of two adjacent standards measured horizontally.

1.3.3 blinding

A layer of lean concrete usually 50 mm to 100 mm thick, put down on soil such as clay to seal the ground and provide a clean bed for construction work.

1.3.4 brace

A tube placed diagonally with respect to the vertical or horizontal members of a scaffold and fixed to them to afford stability.

1.3.5 camber

The international curvature of a beam or formwork, either formed initially to compensate for subsequent deflection under load, or produced as a permanent effect for aesthetic reasons.

1.3.6 coupler

A component used to fix scaffold tubes together.

1.3.7 erection drawing

A drawing prepared prior to erection showing the arrangement and details of the falsework structure.

1.3.8 factor of safety

The ratio of ultimate load to the maximum working load.

1.3.9 falsework

Any temporary structure used to support a permanent structure while it is not self-supporting.

1.3.10 floor centre

A beam of adjustable length, usually a metal lattice or sheet metal box beam, used to support decking for a floor slab.

1.3.11 fork head

A U-shaped housing used to support joists or the like.

1.3.12 formwork and forms

The section of the temporary works used to give the required shape and support to poured concrete. It consists primarily of sheathing material (e.g. wood, plywood, metal sheet or plastic sheet) in direct contact with the concrete, and joists or stringers that directly support the sheathing.

1.3.13 foot tie

A member close to the ground, stabilizing two or more standards.

1.3.14 frame

The principal panel unit of a prefabricated falsework structure formed from welded, bolted or clamped tubular or rolled steel sections.

1.3.15 grade stress

The stress that can be safely sustained by timber of a particular strength class, or species and grade.

1.3.16 guard rail

A member incorporated in a structure to prevent the fall of a person from a platform or access way.

1.3.17 joint pin

An expanding fitting placed in the bore of a tube to connect one tube to another coaxially.

1.3.18 joist

A horizontal or sloping beam, e.g. the horizontal timbers that carry decking for a suspended concrete slab.

1.3.19 lacing

Essentially horizontal members that connect together and reduce the unsupported length of columns.

1.3.20 permissible stress

The stress that can be sustained with acceptable safety by a structural component under the particular condition of service or loading.

1.3.21 permit to load

A certificate issued to indicate that the falsework may safely be put to its designed use.

1.3.22 prop

A compression member used as a temporary support and incorporating a means for varying and fixing its length.

1.3.23 repropping

A system used during the construction operation in which the original props are removed and replaced in a sequence planned to avoid any damage to partially cured concrete.

1.3.24 scaffold

A temporarily provided structure that provides access, or on or from which persons work, or that is used to support material, plant or equipment.

1.3.25 sole plate (or sill)

A timber, concrete or metal spreader used to distribute the load from a standard or baseplate to the ground.

1.3.26 spigot pin

A pin placed transversely through the spigot and the scaffold tube or frame to prevent the two from coming apart.

1.3.27 standard

A vertical or near vertical tube.

1.3.28 stiff length (of the bearing)

The length of the bearing that cannot deform appreciably in bending.

1.3.29 strength class

A classification of timber based on particular values of grade stress.

1.3.30 strength ratio

The ratio of the grade stress to basic stress of timber.

1.3.31 strut

A member in compression.

1.3.32 toe board

An upstand at the edge of a platform intended to prevent materials or operatives' feet from slipping off the platform.

1.3.33 tower

A tall composite structure, used principally to carry vertical loading.

1.3.34 wedge

A piece of strong timber or metal that tapers in its length and is used to adjust elevation or line or to tighten falsework. Folding wedges comprise a pair of wedges laid one above the other so that their outer faces are parallel.

1.4 Symbols

For the purposes of this code, the following symbols, together with those in Annex F, apply. (See Note 1.)

Symbol (see Note 2)	**Quantity**	**Unit of measurement** (see Note 3)
A_d	area of obstruction presented by trapped debris and falsework	m^2
A_e	effective frontal area	m^2
A_w	effective area normal to flow	m^2
A_x	cross-sectional area of pipeline	mm^2
B	width of foundation	m
b	stiff length of the bearing	mm
c	undrained shear strength	N/m^2, kN/m^2
C_f	force coefficient for wind	non-dimensional
C_o	Euler critical stress	N/mm^2
C_w	force coefficient for water	non-dimensional
D	overall depth of beam or section	mm
d	depth	m
d_f	clear distance between flanges	mm
E	Young's modulus of elasticity	N/mm^2, kN/mm^2
e_o	eccentricity of beam bearing	mm

Symbol (see Note 2)	**Quantity**	**Unit of measurement** (see Note 3)
F_{bc}	maximum applied compressive bending stress	N/mm^2
F_c	maximum applied compressive axial stress	N/mm^2
F_d	force on trapped debris	N
F_f	limiting value of the frictional force	N
F_{Rc}	see Annex K	N/mm^2
F_w	force due to flowing water	N
F_x	force due to concrete pipeline	N
H	height above ground level	m
h	depth below ground level	m
H_c	compression flange restraint force	N
h_F	windward height of falsework	m
h_f	windward height of formwork	m
J	moment of inertia of stiffener	mm^4
K_1	modification factor for timber (geometrical properties)	non-dimensional
K_2	modification factor for steel (from BS 449-2)	non-dimensional
K_3	modification factor for timber (duration of load)	non-dimensional
K_4	modification factor for timber (length of bearing)	non-dimensional
K_a	active pressure factor	non-dimensional
K_p	passive pressure factor	non-dimensional
L	overall length of span length of stiffener length of compression flange length between the centre-to-centre of intersection with supporting members	mm mm mm mm
l	effective length	mm
L_b	clear length of a beam	mm
L_f	frontal length of soffit measured normal to the wind	m
L_s	clear length of a column or strut	mm
L_w	width between vertical forms	mm
m	factor for cantilever projection	non-dimensional
n	number of node points number of edge forms length obtained from stiff length of bearing, b	non-dimensional non-dimensional mm
P	force	N
p	maximum pressure in pipeline	N/mm^2
p_a	active soil pressure	N/m^2
P_B	force normal to a sloping soffit	N
p_b	allowable bearing stress	N/mm^2
p_{bc}	permissible bending stress in compressive members	N/mm^2
p_{bt}	permissible bending stress in tension members	N/mm^2
p_c	permissible axial compressive stress	N/mm^2
P_h	horizontal component of a force on a sloping formwork surface	N
P_L	force on a left hand formwork face	N

Symbol	Quantity	Unit of measurement
(see Note 2)		(see Note 3)
p_p	passive soil pressure	N/mm^2
P_R	force on a right hand formwork face	N
P_v	vertical component of a force on a sloping formwork surface	N
q	dynamic wind pressure	N/m^2
q_b	allowable bearing pressure	kN/m^2
q_w	dynamic pressure of flowing water	N/m^2
R	reaction force	N, kN
r	radius of gyration	mm
S_1	topography factor for wind	non-dimensional
S_2	ground surface conditions and height above ground factor for wind	non-dimensional
S_3	statistical (duration of exposure) factor for wind	non-dimensional
T	mean thickness of flange	mm
T_{max}	maximum thickness of compression flange	mm
t_s	web stiffener thickness	mm
t_w	web thickness	mm
U	concrete tube strength	N/mm^2
V	basic wind speed	m/s
V_H	hourly mean wind speed at height H	m/s
V_s	design wind speed	m/s
V_w	speed of water flow	m/s
W	vertically applied force total load on girder	N kN
W_m	maximum wind force during life of falsework	N
W_w	maximum wind force during working operations	N
Y_s	minimum yield stress	N/mm^2
z	elastic modulus	cm^3
β	aerodynamic solidity ratio	non-dimensional
γ	soil density	kg/m^3
Δ_b	out-of-straightness of a beam	mm
Δ_s	out-of-straightness of a column or strut	mm
Δ_v	inclination from vertical	mm
η	shielding factor	non-dimensional
θ	angle from horizontal	degrees
μ	coefficient of static friction	non-dimensional
σ	factor for slenderness ratio	non-dimensional
φ	angle of internal friction	degrees
φ_w	geometric solidity ratio	non-dimensional

NOTE 1 The symbols given in the above list should not be confused with those given in Annex F, where some of the symbols represent different quantities.

NOTE 2 Unless included in the description of the quantity, symbols may have (further) subscripts 1, 2, etc.

NOTE 3

a) Where more than one unit of measurement is given, reference should be made to the relevant application.

b) $1\ N/m^2 = 1$ Pa. $1\ N/mm^2 = 1\ MN/m^2 = 1$ MPa.

1.5 Legislation

The following legislation relates to falsework operations and compliance with this is always a statutory requirement:

Health and Safety at Work etc. Act 1974; Factories Act 1961;

Construction (General Provisions) Regulations 1961 (S.I. 1961, No. 1580);

Construction (Lifting Operations) Regulations 1961 (S.I. 1961, No. 1581);

Construction (Working Places) Regulations 1966 (S.I. 1966, No. 94);

Construction (Health and Welfare) Regulations 1966 (S.I. 1966, No. 95);

Construction (Design and Management) Regulations 1994 (S.I. 1994, No. 3140);

The Management of Health and Safety at Work Regulations 1992.

Section 2. Procedures

IMPORTANT NOTE This section discusses procedural aspects in general terms and it is not the intention of this section to limit the user of this code to any particular method of work. The details of procedures and responsibilities will be influenced by various factors, including the size of the scheme and/or of the organization(s) responsible, and the terms of any contract under which the work is being performed.

2.1 General

2.1.1 Introduction

Collection and application of technical data of the type set out in Section 3 to Section 8 represents only a part of the action necessary to ensure the preparation and execution of a safe and economical falsework scheme. Satisfactory technical solutions can only be assured when adequate control and communication are achieved during both the design and execution of a scheme, and before embarking upon new work every effort should be made to ensure that effective work procedures are developed and put into use. Experience has shown that the use of proper procedures leads to improvements in safety and economy in operation.

One of the main aims of any method of work should be to minimize the chance of errors being made and to maximize the chance of errors being discovered if they are made. To this end there needs to be effective communication of requirements between all levels of the construction organizations involved in a scheme, whether large or small, and whether they are concerned primarily with the permanent works or the temporary works. An effective system of checking is also needed (see **2.6**).

2.1.2 Variety of falsework

The basic procedures to be applied in all falsework are the same, and only the scale of application differs. The recommendations of this code give guidance to those responsible for aspects of design, selection of materials, erection, checking, use, maintenance and dismantling of falsework. It may be possible in the simpler and more commonplace cases for standard solutions to replace individual designs. Some information on standard solutions is given in Section 8 but the standards required and the need for checking are in no way reduced in comparison with the larger, more complex, and specifically designed schemes.

2.1.3 Procedure when more than one organization is involved

While on many occasions the design and construction of permanent works and the attendant falsework will directly involve only one or two organizations, it is becoming increasingly common for the design and/or construction of a part or the whole of a falsework scheme to be undertaken by specialist subcontractors, so introducing further organizational interfaces. Common arrangements include the following and combinations thereof:

a) equipment may be hired, with the supplier carrying out the design, or supplying the basic design data;

b) equipment may be erected using operatives who are not direct employees of the main construction organization, e.g. the supplier of the equipment may also erect it;

c) use of a design produced by another organization;

d) the complete operation may be carried out by another organization.

NOTE More than one additional organization may be involved.

When work is being carried out by different organizations, the organizational interface may be manifested on site as physical interfaces between different phases of the scheme, e.g. it is common for one organization to prepare and provide the foundations upon which another subsequently erects the main falsework structure. The physical interface in this example is particularly critical, but in all cases it is important that the physical limits and interface conditions are clearly defined and that the work procedures adopted are able to take effective account of such matters.

2.2 Formality of procedures

2.2.1 It is important to establish responsibility for, and the scope of, the work of all those organizations and individuals involved in preparing and carrying out falsework. These should be clearly defined, both in respect of the structure or structures with which they are concerned, and the duties they should fulfil.

In those instances where, as will often be the case with the smaller schemes, the conception and direction of the falsework is undertaken by an individual, the degree of formalization of procedures may be minimal, although it is necessary to communicate requirements effectively. Where more people are concerned in the direction and checking of falsework, a formal procedure should be used in order that responsibilities may be properly allocated amongst them and for controlling the communication of requirements and actions.

The key items in both cases are:

a) responsibility for each of the actions set down in this code should be specifically allocated;

b) these responsibilities should be clearly defined;

c) all instructions should be clear and complete.

2.2.2 The main items for which responsibility should be established are:

a) the design brief (see **2.3** and **6.2**);

b) the concept of the scheme;

c) the design, drawing out and specification of the falsework;

d) the adequacy of the materials used;

e) the control of erection and dismantling on site, including maintenance;

f) the checking of design and construction operations;

g) the issuing of formal permission to load and dismantle the falsework (see **2.8**).

Appropriate procedures can be implemented and the required standards achieved by different management arrangements. These should be established to suit both the nature of the falsework to be carried out and the particular management arrangements of the construction organization(s) concerned. Whatever form of organizational direction and control is set up, it is essential that it has the capability to handle the technical requirements of the falsework adequately. For the smaller job, more than one activity may be undertaken by one individual. In larger jobs, delegation of responsibility for particular tasks will probably take place, but it is important for the avoidance of misunderstandings that those who delegate keep in touch on a day-to-day basis with the activities. Care should be taken that a conflict of responsibilities does not arise.

Of equal importance is the need to ensure that the various responsible individuals do not work in isolation from each other but rather that there is some means adopted to provide effective coordination of all tasks. It is therefore recommended that a falsework coordinator be appointed whose duty it should be to ensure that all the actions required are supervised and performed in accordance with the recommendations of this code (see **2.5.2**).

In many instances, the general details regarding the partitioning of responsibilities amongst individuals or groups of individuals will be governed by the requirements of a contract or contracts agreed between the parties concerned. Indeed, in order to avoid ambiguity as to final responsibilities it is advisable that some form of contract is agreed before work commences.

2.3 Design brief

A brief, summarizing the significant facts, should be prepared (see **6.2.1**) to serve as the starting point for subsequent decisions, design work, calculations and drawings. All concerned should contribute towards the preparation of the brief. An assessment of the brief should also indicate how instructions can most effectively be provided for the direction of the falsework operations.

2.4 Communication of requirements

Falsework designs and schemes should be complete with detailed proposals for the foundations, connections to other works and the formwork, before being offered for use on site. Where the design brief indicates that standard and familiar equipment and materials can be used in accordance with standard details and instructions, it will only be necessary to provide information to those persons on site as to the manner and extent to which the items should be used and maintained to ensure safe practice. These directions should be given in conjunction with an arrangement drawing or sketch. The drawings and any related instructions issued to site for the direction of the work may be supported by, for example, standard details, or guides to the use of proprietary systems of falsework equipment.

The drawings and instructions resulting from the design should generally be adequate to direct the work on site and to enable this to be checked. Particular care should be paid to ensuring that the information supplied is easy to understand, and that it draws attention to the need to resolve any details or unusual situations. The positions of each component and the nature of its cross-connections to other components should be shown. Drawings should be uniquely numbered, clearly identified as to location for use, and cross-referenced to any other data with which they should be used. It is desirable to adopt standard methods of noting essential information on the drawings and to use suitable, easily read scales. To aid the location and identification of particular areas, consideration should be given to applying a system of grid references to the scheme.

Where construction is not straightforward, the falsework drawings are unlikely to set down all the parameters of use. In such cases a method statement or construction plan should be prepared on site and made available to all concerned. Where there are any limitations in sequence or timing, these should be set down explicitly as part of the design for site use.

Instructions should be provided indicating how the supporting foundations should be prepared and emphasizing the need to compare the conditions found on the site with those assumed in the initial design for the purpose of confirming their validity.

2.5 Coordination and supervision

2.5.1 General

Work on site should be the subject of careful direction, supervision and inspection to ensure that the falsework structure is constructed safely in accordance with the agreed design with materials of agreed quality, and that only when all checks have proved satisfactory is the structure first loaded, and then dismantled in accordance with an agreed procedure.

Communication tends to be one of the major problem areas of falsework because of the multiplicity of actions normally required when falsework is being constructed and put into service. Such activities may be widely separated in time and place and it is therefore essential that lines of communication and responsibility are explicit. To facilitate progress, a methodical approach should be adopted and it is recommended that comprehensive job notes are maintained.

2.5.2 Falsework coordinator

2.5.2.1 When a construction organization decides to appoint a falsework coordinator (see **2.2.2**), he should normally be directly responsible to their site manager. It is important that his appointment gives him adequate authority to carry out his tasks. In order to provide adequate authority to stop work if it has not been carried out satisfactorily, it may be appropriate for the falsework coordinator to be made responsible for the provision of formal permission to load, which when signed would thereby certify that all appropriate steps had been taken. The falsework coordinator should not permit erection to continue beyond any critical stage until it is to the standard specified. Once the falsework has been checked and passed, it should not be altered until that loading stage has been completed, and the design allows for it to be dismantled or altered. In demanding circumstances it will be advisable for those checking critical items to sign when the work is to their satisfaction. To ensure the independence of checks, the falsework coordinator should delegate the task to another if he himself has carried out any of the activities requiring checking. His appointment should be made known to all concerned.

2.5.2.2 The principal activities of the falsework coordinator should be to:

a) coordinate all falsework activities;

b) ensure that the various responsibilities have been allocated and accepted;

c) ensure that a design brief has been established with full consultation, is adequate, and is in accord with the actual situation on site;

d) ensure that a satisfactory falsework design is carried out;

e) ensure that the design is independently checked for:

1) concept;

2) structural adequacy;

3) compliance with the brief;

f) where appropriate, ensure that the design is made available to other interested parties, e.g. the structural designer;

g) register or record the drawings, calculations and other relevant documents relating to the final design;

h) ensure that those responsible for on-site supervision receive full details of the design, including any limitations associated with it;

i) ensure that checks are made at appropriate stages covering the more critical factors (see **6.1.3** and **7.4**);

j) ensure that any proposed changes in materials or construction are checked against the original design and appropriate action taken;

k) ensure that any agreed changes, or corrections of faults, are correctly carried out on site;

l) ensure that during use all appropriate maintenance is carried out;

m) after a final check, issue formal permission to load if this check proves satisfactory;

n) when it has been confirmed that the permanent structure has attained adequate strength, issue formal permission to dismantle the falsework.

2.6 Checking

At all stages, commencing with the design brief, it is necessary to check that the information being used is correct and that the work carried out is as specified. This is necessary, not only to ensure that a safe and adequate falsework structure is constructed, but also to facilitate rectification should this prove necessary; it is frequently difficult to do this later. Details of checking procedures are given in **6.1.3** and **7.4**.

2.7 Alterations

Changes in the requirements of the design brief should be recorded therein as well as incorporated into the design and drawings (see also **7.4.2**). The effects of any proposed alterations should be checked against the original design and appropriate action taken.

2.8 Loading the falsework

NOTE See also **7.5**.

As a means of exercising a degree of control over the rate of progression once the stage has been reached that the falsework may be loaded, it may be advisable to have a formal procedure for giving permission to load, as indicated in **2.5.2**, perhaps in the form of a "Permit to load". In simple cases, a single permit, made out as soon as the falsework has been satisfactorily checked, may be appropriate. In complex, or larger cases, permits may be required at different stages, both as regards loading and for different areas. It will normally be desirable to limit the period of validity of the permit, as subsequent modifications may take place. The area over which loading may take place, and to what extent, should also be set down in the permit, e.g. it may be "reinforcement only".

2.9 Dismantling

When the structure being supported by the falsework has become self-supporting, the latter may be dismantled subject to a previously agreed procedure and/or restrictions imposed by the permanent works (see **7.6**). As at the loading stage, a permit to dismantle or unload the falsework may be appropriate, particularly where this takes place in stages. It may be appropriate to issue it in conjunction with the designer of the permanent structure.

Section 3. Materials and components

3.1 General considerations

3.1.1 Suitability of materials

All materials, when new, should have complied with the requirements of the appropriate British Standard(s).

3.1.2 Identification and properties

Materials and components should be clearly and readily identifiable by shape or size. Where this is not possible they should be marked.

No material should be used unless its properties are known or established, and its application should be within these limits.

3.1.3 Handling of materials and components

Materials and equipment used in falsework will, in most cases, be used many times. In choosing these, account should be taken of the rough conditions under which they will be handled.

Robust material will obviously suffer less damage and mechanical aids that help in this respect should be considered.

3.2 Testing and inspection

3.2.1 Testing

Where doubt exists as to the quality, by composition or condition, of the material or component it may be submitted to a test of the load/deflection type, wherever possible in accordance with an appropriate British Standard specification.

Testing on site is not normally recommended, as even the most simple tests require disciplined test procedures.

If the performance of an item of equipment is not known, or is in doubt and calculations are impracticable, tests should be carried out to determine necessary values. It should be realized that a single test may give misleading results.

3.2.2 Inspection

Materials and components used for falsework should be examined for damage or excessive deterioration before use and if found unsuitable they should be repaired or scrapped. Accepted engineering principles should be applied in determining whether the element requires rejection or remedial work. The integrity of welds should be visually examined, and for a proper examination, paint coatings may have to be removed.

Careful inspection is essential in preventing unfit second-hand material from being used. Welds should be checked and material that is bent, distorted or badly corroded should be scrapped or sent for repair. Items made from steel should be kept in a well-painted condition. Timber should be inspected between uses not only for evidence of physical damage but for signs of attack by decay, rot or insect attack or the development of shakes or splits. Where the surface of the timber is so obscured by dirt, concrete or other coatings as to prevent inspection for defects and deterioration, it should not be used structurally.

Detailed recommendations for the reuse of used materials are given in **3.3** to **3.9**.

3.3 Steelwork (other than scaffold tube)

NOTE Specific recommendations for tubular steel scaffolding and manufactured components are given in **3.8** and **3.9** respectively.

3.3.1 Quality of steel

Except in cases where particular properties are desirable, weldable structural steel of a higher grade than grade S 275JR (grade Fe 430B) of BS EN 10025 should not be specified. If the material properties are not known, steel should be assumed to be mild steel grade S 235 JR (grade Fe 360B) of BS EN 10025 for design purposes (but see **3.3.6** for welding). For design purposes, old steel of grade 43A may be assumed to be equivalent to grade S275 of BS EN 10025 if it was manufactured in accordance with the 1986 or 1990 editions of BS 4360. Old steel of grade 43A manufactured to previous editions of BS 4360 should be assumed to have a strength midway between grades S235 and S275.

3.3.2 Identification of steel

It is of the utmost importance that the correct quality, size and mass of section should be used. Each element should be clearly marked to assist erection and inspection. The permanent marks, formed when a section is rolled, may not give any indication of quality, mass or true size. Attention is drawn to the differences between rolled steel joists (RSJ) and universal beam (UB) sections, the latter often having thinner webs and dimensions that are nominal only and vary with the particular mass per unit length.

Where steelwork other than mild steel is used for particular properties, it should be so marked as to be readily distinguishable and any connections should be designed to obviate substitution errors.

3.3.3 Permissible stresses

The permissible stresses should be based on the allowable stresses given in BS 449-2:1969 according to the steel being used, as follows.

a) The BS 449 allowable stresses for grade 43 should be used for grade S275 steel to BS EN 10025 and for old grade 43 steel to BS 4360:1990 or 1986.

b) For old grade 43 steel to editions of BS 4360 prior to 1986 the BS 449 allowable stresses for grade 43 steel should be divided by 1.1.

c) For grade S235 steel to BS EN 10025 and for unidentified mild steel, the BS 449 values for grade 43 steel should be divided by 1.2. See Annex A, Table A.2.

3.3.4 Section properties

Where the cross-sectional area of a steel section has been reduced, the actual dimensions should be measured and section properties calculated on the basis of the least cross-sectional area. If such measurements are not possible owing to heavy corrosion or any other reason, load testing should be carried out.

3.3.5 Fatigue

There is normally no need to consider fatigue when designing falsework.

3.3.6 Welding and rectification of steelwork

Different steels require different welding techniques. In general, welding of unidentified steels should not be undertaken for structural work. Steelwork that has been repaired by welding may be used provided that the remedial work has been carried out in accordance with BS 5135. The quality of steel being repaired should first be identified.

Bent sections should be carefully straightened and the work should be carried out preferably by means of straightening rolls, hydraulic press or other controlled method.

Where, for any member to be used as a strut, the lack of straightness is clearly visible to the eye, or exceeds the appropriate tolerances recommended in **7.3.2**, the member should be straightened or discarded.

3.4 Timber

3.4.1 Timber quality

3.4.1.1 *General*

Timber for falsework should be stress graded in accordance with BS 4978 or other appropriate standards of other countries. A full range of stresses for both softwood and hardwood is given in BS 5268-2. That standard also gives a simplified approach to stress grading by using nine rationalized strength classes, enabling a variety of species to be used efficiently. Data on strength classes SC3 to SC5 are given in Table 1.

Details of equivalent timber grades satisfying these various strength classes are given in Table 2 and Table 3. Alternatively, timber may be graded directly to a particular strength class.

3.4.1.2 *Minimum quality*

It is recommended that strength class SC3 timber should be the minimum quality adopted for general use although any timber may be used provided proper account is taken of the appropriate stresses.

A species/grade combination for a higher strength class may safely be used where a lower strength class is specified. The highest grade is M75. Others in descending order, are: SS; M50; GS. MSS and MGS are equivalent to SS and GS respectively. For grading to NLGA rules, the strongest is Selected (Sel) followed by No. 1, No. 2 and No. 3 or a sequence of "Const", "Std", "Util" and "Stud".

Table 1 — Basic stresses and moduli of elasticity for the wet condition

Strength class	Bending stress parallel to grain N/mm²	Tensile stress parallel to grain N/mm²	Compressive stress perpendicular to grain N/mm²	Shear stress parallel to grain N/mm²	Modulus of elasticity	
					Mean N/mm²	Minimum N/mm²
SC3	4.24	2.56	1.32	0.60	7 000	4 600
SC4	6.00	3.60	1.44	0.64	7 900	5 300
SC5	8.00	4.80	1.68	0.90	8 600	5 700

NOTE 1 This table is based on BS 5268-2:1991. The values take into account the modification factor for the wet condition in that code.

NOTE 2 Timber in the wet condition has a moisture content greater than 18 %.

Table 2 — Equivalent strength classes for softwoods graded in accordance with BS 4978

Standard name	Strength class		
	SC3	SC4	SC5
Imported			
Douglas fir-larch (Canada)	GS	SS	—
Douglas fir-larch (USA)	GS	SS	—
Hem-fir (Canada)	GS, M50	SS	M75
Hem-fir (USA)	GS	SS	—
Parana pine	GS	SS	—
Pitch pine (Caribbean)	GS	—	SS
Redwood	GS, M50	SS	M75
Sitka spruce (Canada)	SS	—	—
Southern pine (USA)	GS	SS	—
Spruce-pine-fir (Canada)	SS, M50	SS, M75	—
Western whitewoods (USA)	SS	—	—
Whitewood	GS, M50	SS	M75
British grown			
Corsican pine	M50	SS	M75
Douglas fir	SS, M50	—	M75
European spruce	M75	—	—
Larch	GS	SS	—
Scots pine	GS, M50	SS	M75
Sitka spruce	M75	—	—

NOTE Machine grades MGS and MSS may be substituted for the GS (general structural) and SS (special structural) grades respectively.

Table 3 — Equivalent strength classes for North American softwoods graded in accordance with NLGA and NGRDL joist and plank grades[a]

Standard name and origin	Strength class		
	SC3	SC4	SC5
Douglas fir-larch (Canada)	No. 1, No. 2	Sel	—
Douglas fir-larch (USA)	No. 1, No. 2	Sel	—
Hem-fir (Canada)[a]	No. 1, No. 2	Sel	—
Hem-fir (USA)	No. 1, No. 2	Sel	—
Sitka spruce (Canada)[a]	Sel	—	—
Southern pine (USA)	No. 1, No. 2, No 3	—	Sel
Spruce-pine-fir (Canada)[a]	No. 1, No. 2	Sel	—
Western whitewoods (USA)[a]	Sel	—	—

NOTE The classifications in this table apply only to timber of a size not less than 38 mm × 114 mm.

[a] NLGA [National grading rules for dimension lumber (Canada)].
NGRDL [National grading rules for softwood dimension lumber (USA)].

3.4.1.3 ***Hardwood and softwood***

Many hardwoods are denser and stronger than softwoods but the strength classification system applies equally to both. For load-bearing wedges and packings, the higher strengths available only from the denser hardwoods will be desirable. Most straight-grained hardwoods whose density exceeds approximately 650 kg/m^3 will be suitable for such wedges. The following list details some commonly available, hardwood species suitable for use as load-bearing wedges and packing:

Standard name	Botanical species
ash	*Fraxinus excelsior*
beech	*Fagus sylvatica*
greenheart	*Ocotea rodiaei*
jarrah	*Eucalyptus marginata*
karri	*Eucalyptus diversicolor*
keruing	*Dipterocarpus spp*
oak	*Quercus spp*

3.4.1.4 ***Stress graded timber***

Timber that has been accepted as stress graded carries appropriate marking, comprising a stamp on at least one face, edge or end to indicate the grade. In the case of timber machine stress graded in accordance with BS 4978, the BSI Kitemark usually appears. Sometimes, as a result of the process of machine stress grading, the timber has coloured marks at intermediate points indicating the grade near each point. The lowest strength found at any of these intermediate points dictates the overall classification of the piece.

Machine stress graded timber is assessed about its lesser dimension, so if used on edge, a check should be made to see that there are no splits or knot combinations that would result in down grading.

3.4.2 Modification factors

3.4.2.1 ***General***

The basic stresses given in Table 1 should be multiplied by the appropriate modification factors in **3.4.2.2** to **3.4.2.10**.

3.4.2.2 ***Moisture content***

The data given in Table 1 relates to timber with a moisture content greater than 18 %. If, in the exceptional case, the timber has a lower moisture content, higher stresses may be used in accordance with BS 5268-2:1991. However, these are seldom appropriate for falsework.

3.4.2.3 ***Size***

Timber swells across the grain as it gets wetter. Because timber is sold as the "dry size", under site conditions it will normally be larger. Table 4 gives appropriate values for modification factor K_1. In all calculations, actual and not nominal sizes of timber should be used.

Table 4 — Modification factor, K_1, by which the geometrical properties of timber for the dry exposure condition should be multiplied to obtain values for the wet exposure condition

Property	K_1
Thickness, width and radius of gyration	1.02
Cross-sectional area	1.04
First moment of area, section modulus	1.06
Second moment of area	1.08

3.4.2.4 ***Time***

The basic stresses given in Table 1 are for timber under load for 50 years but timber will safely sustain higher stresses for shorter periods. Table 5 gives appropriate values for modification factor K_3. The effect is cumulative over the life of the timber, unless the time lapse between loadings is at least equal to the time during which loads have been imposed.

Table 5 — Modification factor, K_3, for duration of load on falsework

Duration of loading	K_3
1 year	1.2
1 month	1.3
1 week	1.4

3.4.2.5 ***Compression perpendicular to the grain***

There should be no wane at bearing positions. The stresses given in Table 1 to Table 9 assume that there will be no wane or other loss of section at these points; if there is, the stresses should be halved. In no case should wane exceed that permitted in BS 4978.

Stresses for compression in Table 1 may be increased by a factor of 1.2 to give values appropriate to falsework.

Where the bearing area is short along the grain, a higher stress may be taken, provided that there is at least 75 mm of timber each side of the bearing. Appropriate values for modification factor K_4 are given in Table 6.

3.4.2.6 ***Compression parallel to the grain***

If it is intended to use timber as struts, reference should be made to BS 5268-2:1991 for appropriate stresses.

Table 6 — Modification factor, K_4, for bearing stresses

Max. length of bearing mm	K_4
10	1.74
15	1.67
25	1.53
40	1.33
50	1.20
75	1.14
100	1.10
150 or greater	1.00

3.4.2.7 ***Depth-to-breadth ratios***

There is a risk of tall timbers turning sideways. Table 7 gives the ratios that should not be exceeded.

Table 7 — Maximum depth-to-breadth ratios

Degree of lateral support	Maximum depth-to-breadth ratio
No lateral support	2:1
Ends held in position	3:1
Ends held in position and member held in line, e.g. by purlins or tie-rods	4:1
Ends held in position and compression edge held in line, e.g. by direct connection of soffit formwork	5:1
Ends held in position and compression edge held in line, e.g. by direct connection of soffit formwork together with adequate bridging or blocking spaced at intervals not exceeding six times the depth	6:1
Ends held in position and both edges held firmly in line	7:1

3.4.2.8 ***Shear***

The values for stress parallel to the grain given in Table 1 may be increased by a factor of 1.5 to give the permissible stress across the grain.

The maximum stress should be calculated initially by dividing the shear force by the cross-sectional area. This value should then be multiplied by 1.5 (the distribution is parabolic). Care should be taken to see that splits are not positioned where a high shear is expected (see **3.4.4**).

3.4.2.9 ***Modulus of elasticity***

The modulus of elasticity varies with moisture content, but not with time and thus the modification factor K_1 is applicable, but not K_3. (See also **3.4.2.10**.)

3.4.2.10 ***Load sharing***

Where a single member may carry its full share of the load unaided by its neighbours, the appropriate working stresses and the minimum value of modulus of elasticity should be used, but if members are spaced not more than 610 mm apart, and there is adequate provision for lateral distribution of loads by means of decking or joists spanning at least four supports, stresses may be increased by a factor of 1.1.

For the calculation of deflection of load-sharing systems, the mean value for the modulus of elasticity should be used.

3.4.2.11 ***Depth***

The data for bending stress parallel to the grain given in Table 1 and Table 9 are applicable to timber which is 300 mm deep. For beams between 72 mm and 300 mm in depth, slightly higher figures are appropriate. These permissible stresses are given in Table 8. For timbers more than 300 mm deep, see **14.6** of BS 5268-2:1991, which makes adjustments using factor K_7.

This approach may also be used for timbers with a depth less than 300 mm as an alternative to Table 8.

Table 8 — Permissible bending stress parallel to the grain for beams of different depths for general falsework applications

Nominal depth range (mm)	up to 72	73 to 100	101 to 125	126 to 150	151 to 200	201 to 225	226 to 300
Timber class (N/mm²)							
SC 3	7.36	7.10	6.93	6.79	6.58	6.49	6.29
SC 4	10.42	10.05	9.80	9.61	9.31	9.19	8.90
SC 5	13.89	13.40	13.07	12.81	12.41	12.25	11.87

3.4.2.12 ***Permissible stresses***

The stresses given in Table 9 may be used for calculations if the following conditions apply (the modification factors described in **3.4.2** have already been applied):

a) the timber has been accepted as appropriate to the class concerned;

b) sizes for dry timber are used;

c) the duration of the load does not exceed one week;

d) there is no wane;

e) the bearing length does not exceed 75 mm, there is at least 75 mm of timber each side of the bearing, and take-up is not critical;

f) the depth-to-breadth ratios of Table 7 are not exceeded.

Table 9 — Permissible stresses and moduli of elasticity for general falsework applications

Strength class	Bending stress parallel to grain N/mm²	Tensile stress parallel to grain N/mm²	Compressive stress perpendicular to grain N/mm²	Shear stress perpendicular to grain N/mm²	Modulus of elasticity E	
					Mean N/mm²	Minimum N/mm²
SC 3	6.29	3.73	2.63	1.32	7600	5000
SC 4	8.90	5.24	2.87	1.40	8600	5700
SC 5	11.87	6.99	3.35	1.97	9200	6100

3.4.3 Timber that is not stress graded

Where it is not possible to obtain appropriate stress graded timber, commercially graded timber may be used instead provided unsuitable material is removed from the batch; this should be carried out in accordance with the visual stress grading rules of BS 4978.

Care should be taken that what is being supplied is not the reject fraction from stress grading. Commercial grade timber that may prove suitable is listed in Table 10.

Table 10 — Commercial grade timber suitable to produce mainly class SC3 timber

Standard name	Commercial description
Czech whitewood (Ligna)	Unsorted
Finnish, Swedish and Polish redwood and whitewood	Fifths “V”
Pacific coast Douglas fir, hemlock, western whitewoods and spruce	1 and 2 merchantable
Russian redwood and whitewood	Fourths “IV”

3.4.4 Grading reused timber

Where the origin and grading of the material is no longer known, it can be regraded, when the species is known, although this should be done by a person capable of such work.

Cross cutting will not reduce the grading, but when timber is ripsawn after stress grading, the original grading will no longer be valid.

Timber should be discarded if any of the following are present:

a) it has been painted such that it prevents assessment;

b) there is any sign of rot, fungal or chemical;

c) there are cuts in the edge greater than one-twentieth of the thickness of the section;

d) there are holes in the two outer thirds;

e) there is other mechanical damage;

f) there is any undue distortion of shape;

g) in the case of timber to be used for beams or columns, there is a piece containing any fissures at slopes greater than 1:6 for class SC3 or 1:10 for class SC4.

In cases c) and d), the design may be based on the remaining good section.

Timber should never be reused without a careful inspection.

3.5 Concrete and concrete components

3.5.1 Mix design

The concrete should, in general, comply with the permanent works specification appropriate to its intended location, e.g.:

a) blinding;

b) spread footings;

c) piles;

d) pile caps;

e) structural members.

Relaxation of the specification, however, may be permitted to allow for its temporary function. The following are unlikely to be of significance over a short period:

1) conversion of high alumina cement;

2) attack of Portland cement concrete in sulfate-bearing ground;

3) attack of reinforcement due to calcium chloride.

3.5.2 Blinding concrete

Blinding concrete is intended only to prevent local degradation of the soil and to provide an even surface. Its thickness should be not less than 50 mm. The allowable bearing pressure should be the same as that of the underlying soil or weak rock, up to a maximum of 2 000 kN/m^2.

3.5.3 Spread footings

Mass concrete intended to transfer load direct to the ground and more than 100 mm thick should be regarded as a spread footing. Spreading from the perimeter of load at an angle of 45° to the axis of the load may be assumed in calculating the bearing pressure on the ground.

3.5.4 New structural members

New structural members should be designed and constructed in accordance with the specifications for the permanent works except in so far as their temporary function permits relaxation of serviceability requirements. For example, it may be possible to:

a) reduce cover to the reinforcement;

b) neglect long-term creep deflections;

c) accept greater crack widths;

d) accept higher deflections.

Care should be taken to ensure that the concrete is of adequate strength before any load is applied to it. Guidance on the age at which load may be applied to structural members is given in CIRIA Report No. 136 [6]

3.5.5 Second-hand structural members

Any concrete member not specifically designed for a particular project should not be used unless all the following are satisfied:

a) the position and sizes of the reinforcement and the strength of the concrete are known or can be ascertained;

b) the member is free from excessive cracking, spalling, or visible or known defect;

c) the member has been subjected to a design check and found adequate for its proposed use.

3.5.6 Precast concrete

Precast concrete members forming part of a falsework system should be manufactured in accordance with the recommendations of BS 8110-1, -2 and -3. In the case of symmetrical sections having the principal reinforcement on only one face, the members should be clearly marked to indicate the way they should be used or, alternatively, the reinforcement should be designed to enable the member to serve its design performance whichever way up it is erected.

Special attention should be paid to concrete used as permanent formwork and subsequently used in composite construction. In some instances temporary supporting falsework may be required during the placing of in situ concrete on top. Consideration should be given to stresses and deflections that may be locked into the precast concrete members before composite action is assured.

3.6 Brickwork and blockwork

3.6.1 Characteristic strength

Characteristic strengths for brickwork and blockwork should not exceed those recommended in BS 5628-1.

3.6.2 Rate of building

Where brickwork is to be load bearing, the rate of building should be restricted to a maximum height of 1.5 m per day for normal construction in bricks with a compressive strength of up to 35 N/mm^2. No one portion should rise more than 1 m above the level of adjacent brickwork. Bricks of higher compressive strength usually have a higher density and lower absorption and because of this it may be necessary to limit the height of the building per day still further.

3.6.3 Age of loading

If the loading is to be applied less than 28 days after completion of the brickwork or blockwork, characteristic strengths in compression should be calculated in accordance with Table 2 of BS 5628-1:1978. Brickwork or blockwork should not be loaded at less than 7 days and preferably not less than 14 days after completion of the work.

3.6.4 Reinforced brickwork and blockwork

Only new materials should be used for reinforced brickwork and blockwork, which should satisfy the recommendations of BS 5628-1.

3.6.5 Salvaged bricks and blocks

Salvaged bricks and blocks may be used for falsework other than in situations given in **3.6.4** provided that they are sound and free from contamination (e.g. gypsum plaster) that might adversely affect the mortar or its bond with the bricks or blocks, or have any other adverse effect on the structure.

The permissible characteristic strength in compression should be calculated as recommended in BS 5628-1, using a unit crushing strength of the bricks of 3 N/mm^2 for stock bricks and 5 N/mm^2 for other types.

Where salvaged bricks are used, it should be assumed that unreinforced brickwork possesses no tensile strength.

3.7 Other materials

3.7.1 General

Where the material chosen for falsework is not among those described in **3.3**, **3.4**, **3.5** and **3.6**, the approach described in this clause should lead to the use of satisfactory working stresses. In all cases where British Standard codes of practice exist, the design and usage of these materials should conform to the recommendation of the appropriate clauses of those codes (e.g. for aluminium use BS 8118).

3.7.2 Permissible stresses

The supplier of the material should be called upon to provide sufficient information and test data to enable suitable permissible stresses to be established. Two of the variables requiring consideration are creep and the duration of loading.

3.7.3 Deterioration

Unfamiliar materials may deteriorate in unexpected ways but at least the following should be considered:

a) fatigue;

b) heat;

c) chemical deterioration;

d) organic or ultraviolet deterioration;

e) animal or bacteriological action;

f) deformation or lack of straightness;

g) local adaptations that have caused loss of cross-sectional area.

Such deterioration should be identified by examination of materials when their use is proposed and allowance made, if necessary after testing.

3.8 Steel scaffold tubes, couplers and other fittings

3.8.1 Equipment in general use

Most equipment in use in the UK comprises 4.0 mm thick (8 SWG) steel tube of 48.3 mm outside diameter complying with the requirements of BS 1139. New steel tube should comply with BS 1139-1.1:1990. A large quantity of steel tube made in accordance with BS 1139-1:1982 is still in use. Properties in accordance with these two standards are given in Annex B. Test loads are also given in BS 1139 for various couplers and fittings.

Steel and aluminium alloy tubes should not normally be used in the same structure, but if it is necessary to do so the difference in their properties should be allowed for in the design. Their properties should be obtained from the appropriate British Standards.

3.8.2 Non-standard scaffold tubes

Attention is drawn to the existence of steel scaffold tubes having a wall thickness less than 4 mm and that do not comply with the requirements of BS 1139, but that do have the same 48.3 mm outside diameter. Such tubes are in general use in some countries abroad, but are not recommended for use in the UK owing to the risk of inadvertent substitution.

Tubes of a higher grade, greater thickness and diameter are available as well as open-jointed tube. Data for tubes other than those complying with the requirements of BS 1139 are not given in this code.

3.8.3 Loads on struts

Safe working loads on mild steel scaffold tube struts are given in Annex B. The allowable stresses adopted are in accordance with BS 1139. An overall K_2 factor of 2.00 is used; this will allow for some corrosion and will be appropriate for the eccentricity of load application and transmission through couplers. If other tubular members are used, the material is likely to comply with the requirements of BS EN 10210-1 in conjunction with BS 4848-2 and for suitable permissible loads the appropriate data from these British Standards should be used in the formula indicated in Annex B.

The tolerances specified for erection should be in accordance with **7.3.2.3**. Where it is expected that loads will not be applied within these tolerances, the safe working load may be considerably lower and should be specifically calculated.

To determine a safe working load for the strut of length L between the centre-to-centre of intersections with supporting members, its effective length l should be calculated as described in **6.7.2**.

3.8.4 Straightening of scaffold tubes

Bent steel tube may be straightened by using a crow or a reeling machine. This operation should be carried out under competent supervision. Reeling should not be used to straighten bent aluminium scaffold tube. A bent scaffold tube should not be used until straightened by an approved method and returned to within permissible tolerances (see Annex B).

3.8.5 Corrosion and deterioration of steel scaffold tube and fittings

Steel scaffold tubing that is corroded to the extent of rust flaking or severe pitting of the surface should be cleaned of all loose material. It should then be measured for external diameter and checked against the minimum requirements of BS 1139 as given in the summary of properties and tolerances in Annex B. Particular attention should be paid to the 75 mm length at the end of the tube, as this tends to corrode more than the rest of the tube.

Scaffold tube should be free from cracks and should have the ends cut square, smooth and free from burrs. Sections of tube that have been seriously deformed or creased by abuse should be cut out and discarded. Where tubes have become thin or split at the ends, these should be cut off, all such cuts being at right angles to the axis of the tube.

3.8.6 Scaffold fittings

Annex B gives safe working loads for couplers and some other fittings complying with the requirements of both the 1982 and 1991 editions of BS 1139. In many cases, scaffold fittings have strengths greater than the minima required by BS 1139. Should the fittings be tested in accordance with the methods specified in BS 1139 and the failure loads are higher than the minimum required, the working loads may be based on the failure loads actually obtained by applying a factor of safety against slipping of 2.0 where this loading is required.

Where working loads higher than 6.3 kN arrived at by this means are used in design, particular care should be taken to ensure that only fittings of the type tested are used.

Care should be taken not to substitute right angle or swivel couplers with others, such as putlog couplers, having a lower load carrying capacity than 6.3 kN or 5.3 kN respectively.

NOTE For loads on supplementary couplers see **6.7.1**.

3.8.7 Deterioration of scaffold couplers and fittings

Fittings should be inspected before use and should not be used if they are damaged in any way that would reduce their efficiency. It is necessary to ensure that the threads are not damaged so that there can be no confusion over whether the fitting has been tightened to the tube or to the damaged thread. Damaged fittings should be removed from the site for remedial treatment before use.

Baseplates should be checked to ensure the integrity of the welds, the squareness of the base to the stem and the planeness of the base.

3.9 Manufactured components for falsework

3.9.1 Types of manufactured components

The following are examples of manufactured components:

a) framed tower components for vertical load bearing;

b) adjustable steel props;

c) adjustable steel centres for bridging between beams;

d) bridging girders;

e) soldiers.

3.9.2 Design and testing of manufactured components

When structural analysis of a component or system is not practical, tests will be necessary to establish the performance. Annex C gives recommendations for testing falsework equipment.

3.9.3 Information from the supplier

Information necessary for the design, erection, use, dismantling and maintenance of the components offered as falsework should be available from the supplier.

This information should relate to the properties of the individual components, their use in expected assemblies, and any specific requirements for inspection and maintenance, and should include the following:

a) the intended uses for the components, how they can be identified, and the appropriate dimensions and masses;

b) whether the components comply with the requirements of an appropriate British Standard or standard;

c) details of connections where loads are received into one or more components, transferred from one to the other, and transmitted to other supports such as foundations;

d) recommended maximum working loads for various conditions of use, such as different extensions and eccentricities, together with details of any necessary bracing or lacing;

e) any limiting deflection conditions.

Continuing quality control of manufacture should be ensured.

3.9.4 Factors of safety

Where the strength of a manufactured component cannot be ascertained by applying the design criteria recommended in this code, it should be determined by applying a minimum factor of safety against collapse of 2.0 when tested in a condition equivalent to the worst acceptable condition described in this code (see Section 7) or actual site condition likely to apply. An appreciably larger factor of safety will be appropriate for materials that have a wide scatter of strengths.

3.9.5 Framed tower components for vertical load bearing

Prefabricated components usually assemble into rectangular towers, of which the perpendicular load-bearing members are frequently in pairs, but towers may also be square or triangular, or have many legs. There may be separate bracing members, but in most cases assembly is simple. The adjustment needed to make the tower vertical will be very small if the base is level.

Because a number of assumptions have to be made to enable BS 449-2 to be used for calculations, testing may need to be carried out. A height of at least 5 m should be adopted, and appropriate horizontal loading included in the test. Where some lateral restraint is needed, this should be clearly stated in the instructions for use.

The components should be made within the tolerances assumed in the design. Because there are many joints, the total tolerance may be significant. There will be some take up at the joints as the load is applied, and the anticipated distribution of load in the legs may become modified. Allowance should be made so that the final level at the top is that required.

Where testing of prefabricated towers is required, the procedures described in DD 89 should be followed.

NOTE DD 89 (now withdrawn) covered the testing of prefabricated towers. The procedures are currently being updated and will be issued as a CEN standard.

3.9.6 Adjustable steel props

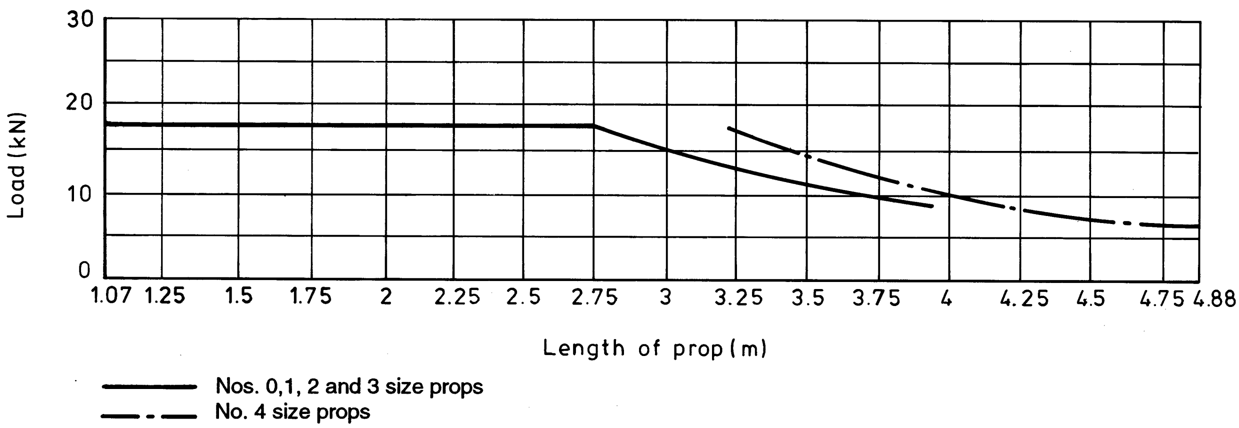
Steel props should comply with the requirements of BS 4074, in which the construction and sizes of adjustable steel props are specified. A prop comprises telescopic tubular sections with top and bottom plates approximately 150 mm square. The transfer of load between the two parts is by an adjustable collar and a pin. The pin is of high tensile steel, and should only be replaced with one of the appropriate type. Table 11, based on BS 4074, gives the height ranges of props according to their size number.

Figure 1a) gives the safe working loads for props erected to the maximum erection tolerances permitted under the recommendations of **7.3.2.2**. A prop will carry its maximum safe working load when it is plumb and loaded concentrically. Where concentric loading can be guaranteed, e.g. by placing the loading member in a forkhead and twisting it to ensure the member is central, the higher loadings indicated in Figure 1b) may be used.

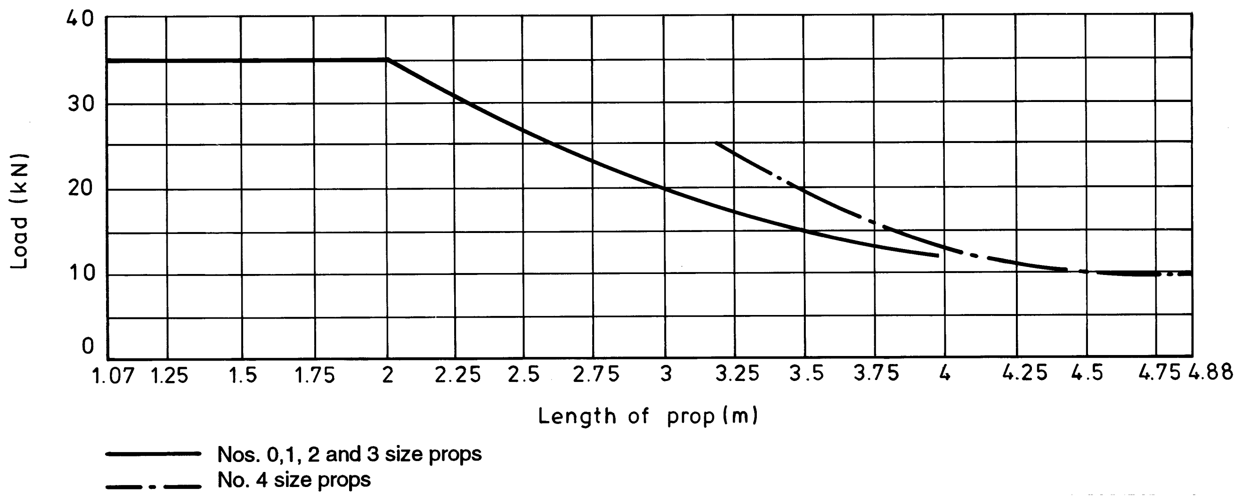
Where testing of props is required, such testing should be carried out in accordance with BS 5507-3.

Table 11 — Adjustable steel prop heights

Prop size no.	Range of prop height mm	
	min.	max.
0	1 070	1 820
1	1 750	3 120
2	1 980	3 350
3	2 590	3 960
4	3 200	4 870



a) Safe working loads for props erected 1.5° maximum out-of-plumb and with up to 25 mm maximum eccentricity of loading (i.e. concentric loading cannot be ensured)



b) Safe working loads for props erected 1.5° maximum out-of-plumb and with concentric loading ensured

Figure 1 — Safe working loads for props

3.9.7 Floor centres

Floor centres normally comprise two (or more) fabricated steel beams, usually of lattice or box construction, that can slide within each other to give the desired length. An overlap at the middle provides strength and continuity between the two sections. The ends are flat plates level with the top surface, which transfer the shear forces to the supports.

Floor centres are usually made with a built-in camber because they deflect significantly under load. However, it is difficult to achieve a precise result, and where accuracy is important, or where the loading is too great for the span, an intermediate prop may be considered. However, as some floor centres cannot accept a prop, and the correct location is very important, it is imperative to follow the instructions supplied with the equipment.

Where the capacity is to be established by test, floor centres should be tested in accordance with BS 5507-1. The safe working load of these floor centres should be used with a factor of safety of 2.0 related to the onset of yield. The design of the floor centres should be such that visible deformation will take place before final collapse occurs.

3.9.8 Bridging girders

Many types of proprietary systems are in use, e.g. the Bailey Bridge, and most are created by the assembly of panels or other structural components. It is important to refer to the information provided with the equipment concerning the correct method of assembly and lifting. Where torque spanners are required in erection it is essential to see that they are correctly set. Where lateral bracing is specified between girders this should always be fixed before any other loads are applied to the girders. The recommendations in the information supplied with the equipment and the relevant assembly instructions should be used both in design and erection, to check the assembly stability and support arrangements.

Because of the long life of these panels, together with the nature of the loading to which they may have previously been subjected when used in bridges, it is important to inspect the panels for signs of fatigue in addition to signs of damage or corrosive deterioration (see Annex D). Where systems are pinned rather than bolted together, particular care should be taken in assessing deflections and any initial movement of connections. In critical cases, preloading should be considered.

3.9.9 Military trestling systems

Various types of military trestling are still in use but it is recommended that work utilizing this equipment is only designed or executed by persons having specialized knowledge of the equipment. Attention is drawn to the reduced capacity of some of the more recently manufactured components.

The members should be carefully examined for signs of corrosion or damage, particularly at the column channel section between batten plates.

No holes should be drilled in the trestling members, nor additions made, without the approval of a structural designer familiar with this type of equipment.

Section 4. Loads applied to falsework

4.1 General

Falsework should be designed to cater for:

a) self-weights;

b) imposed loads;

c) environmental loads.

Wherever possible, the precise loading conditions and the exact weights of materials should be ascertained. If estimated allowances are used for design purposes, a check should be carried out when details of the falsework and loading conditions are known and the design confirmed or amended as necessary.

4.2 Weights of materials

Where not given in this code, self-weights of materials should be taken from BS 648 and imposed loads taken from BS 6399-1. Recommended figures for the masses and densities of some commonly used materials are given in Annex E. For the purposes of this code, a standard density value for reinforced concrete of 2 500 kg/m^3 is recommended, with aggregates of normal density and up to 2 % reinforcement (for guidance on variations to this figure see Annex E).

4.3 Self-weights

The total self-weight should include the self-weight of:

a) the falsework structure;

b) any ancillary temporary works connected to the falsework, e.g.:

1) access ramps;

2) hoist and other tower structures;

3) loading storage platforms;

4) raking and flying shores;

c) the formwork;

d) any permanent works elements forming an integral part of the formwork and falsework.

4.4 Imposed loads

4.4.1 General

Imposed loads should include the loading from:

a) permanent works, e.g. reinforcement and concrete;

b) construction operations, including:

1) working areas;

2) storage areas;

3) pedestrian traffic;

4) vehicular traffic;

5) static plant;

6) mobile plant.

4.4.2 Permanent works loading

The permanent works loading should be assessed from the self-weight of the permanent structure supported by the falsework.

Where impact may occur in the placing of permanent works elements, an extra vertical load should be assumed, which will require assessment in each case. For example, for a unit weighing 5 t (tonnes), with normal crane operation, an additional 50 % of the unit's mass should be allowed. For a piece of steelwork weighing 100 t, crane control would normally be more careful and an allowance of an extra 10 % may then be appropriate.

The local effect of impact and heaping during concreting operations is catered for in the loading given for working areas (see **4.4.3.1**).

The loading effects of movements due to shrinkage, creep, thermal changes and post tensioning are considered in Section 6.

When concrete is placed against or upon formwork surfaces, they will be subjected to forces that may have vertical and horizontal components. In some cases, e.g. where void formers or top formwork are used, uplift forces may be created.

The forces exerted by concrete should be calculated using either the method set out in CIRIA Report No. 108 [4] or an estimate of the concrete pressure (expressed in kN/m^2) may be based conservatively on 25 times the depth of pour (in m). The latter method should be used where the concrete will apply a full hydrostatic head to the formwork as would be the case when using certain admixtures.

4.4.3 Construction operations

4.4.3.1 ***Working areas***

Allowance should be made for access and working area loading on the falsework.

A loading of 1.5 kN/m^2 is recommended to allow for the following situations:

a) construction operatives;

b) hand tools and small equipment commonly used in reinforced concrete construction;

c) materials required for immediate use;

d) the common situations of impact and heaping of concrete occurring during placing operations.

For this loading allowance to be valid, the concrete should not be dropped into the reception arrangements or on to horizontal surfaces from a free height greater than 1 m, nor should the concrete be allowed to heap and accumulate on the formwork to a height more than three times the depth of the slab, with a limit in area for any such accumulation to this height of 1 m^2 (see Figure 2). Should it be necessary to exceed these limitations, allowance for the additional loading should be made in the design.

Where allowance has only to be made for access and inspection purposes, a loading of 0.75 kN/m^2 should be adequate.

These loadings are for access and working conditions. They are not intended to allow for the stacking of materials or for the loads of other equipment and plant for which specific provisions should be made as described in **4.4.3.2** and **4.4.3.4**.

4.4.3.2 ***Storage areas***

Where materials to be stored on the working area produce concentrated or distributed pressures greater than 1.5 kN/m^2, provision should be made in the design for the additional loading. This provision should either extend over the whole of the working area or the storage areas should be specifically designated and clearly marked on the drawings and on the site. The limits of the area, the nature of the materials to be stored and the height to which the materials may be stacked, hence the intensity of loading, should be clearly defined and communicated in a form suitable for use and understanding on site. Similar precautions will be necessary to guard against excessive storage of materials on a recently formed deck before this is self-supporting.

4.4.3.3 ***Traffic***

Where passage is required through or adjacent to areas on which falsework operations are taking place, appropriate arrangements should be made for the protection of vehicles and people using these access ways.

Where vehicles or people are required to be able to enter falsework working areas, provision should be made for the resulting loadings. Where no entry or passage of vehicles or people is allowed for in the falsework design, adequate arrangements with barriers and notices should be provided to prevent such passage. Protective arrangements to minimize the consequences of impacts should include means to give early warning to drivers by the use of notices, load gauges, light-weight screens, etc.

The following loading allowances are recommended:

a) pedestrians (members of the public): 4.0 kN/m^2;

b) pedestrian access barriers: 0.75 kN/m applied in any direction;

c) vehicles on falsework: use appropriate axle or wheel loads with allowance for impact of 11.25 t maximum;

d) vehicle crash barriers: 7.5 kN/m applied in any direction.

NOTE Reference should be made to BS 6180 for special barriers.

4.4.3.4 ***Plant (including construction vehicles)***

Imposed loads from items of plant should include the self-weight of the plant plus any load it will carry.

a) *Vibration*. In general, plant vibration is unlikely to cause any significant increase in loading. However, the loosening effects on bolts, wedges and other friction connections should be considered, particularly when external vibrators are used.

b) *Dynamic loading from loads moving vertically*. The static loading of the moving item should be increased by 25 % when using mechanically operated lifting gear, and by 10 % when using manually operated lifting gear.

c) *Dynamic loading from loads moving horizontally*. This loading results from moving plant, or from loads being deposited by lifting equipment positioned on or off the falsework or being carried across the falsework by plant or on moving equipment. The design should allow for a horizontal force in any of the possible directions of movement equivalent to 10 % of the static load of the moving items where the rate of travel cannot exceed 2 m/s. Without this speed limitation the horizontal force should equate to 33⅓ % of the moving load.

d) *Impact*. The consequences of suspended loads impacting on falsework should be considered in the light of the damage that would result.

e) *Concrete pumping*. Where concrete is being placed by pump or pneumatic placer through a pipeline carried on the falsework, the additional force due to the concrete pipeline, F_x (in N), applied to the falsework may be calculated from the expression:

$$F_x = 0.25pA_x$$

where

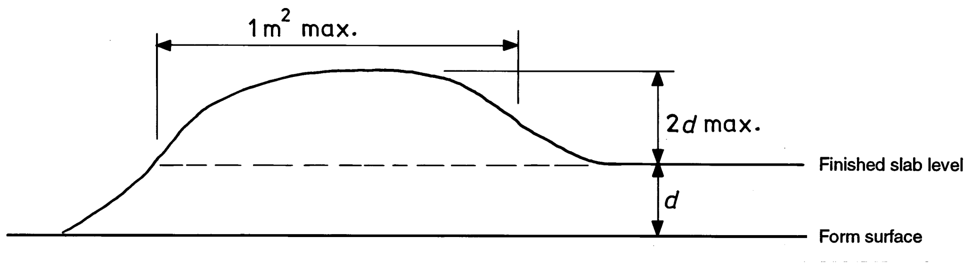
p is the maximum pressure in the pipeline (in N/mm^2);

A_x is the cross-sectional area of the pipeline (in mm^2).

The maximum pressure in the pipeline, p, is dependent upon a number of factors but the following pressures are not likely to be exceeded:

1) 5 N/mm^2, for mechanical pumps;

2) 0.7 N/mm^2, for pneumatic placers.



d is the depth of the slab.

Figure 2 — Controlled heaping of concrete

4.5 Environmental loads

4.5.1 Wind loading

4.5.1.1 *General*

The wind acts upon all surfaces over which it passes whether these are normal or inclined to the wind direction and may apply pressure or suction forces to those surfaces. The exposure of a falsework structure to wind produces different loading conditions for differing wind directions and speeds, at the various stages of falsework erection.

The basic data for the design of buildings to resist wind forces are contained in CP 3:Chapter V-2, which contains fundamental data concerning wind speeds and forces for different exposure conditions together with guidance on the application of this data to different building types and structures.

This code extracts from CP 3:Chapter V-2 the data needed to calculate wind forces on falsework structures.

4.5.1.2 *Information required by the falsework designer*

The following information is necessary for the calculation of wind forces:

a) location of site;

b) topography of site and exposure of the site to open wind conditions;

c) duration of falsework operations and use;

d) general arrangement of the structure, including its dimensions;

e) numbers and sizes of falsework members from which to determine the areas on which the wind will act;

f) details of the attachments to the falsework or items imposed upon it, e.g. formwork that will be subjected to wind forces.

4.5.1.3 *Design wind speed,* V_s

The assessment of wind speed should take into account the positioning and exposure of the falsework to the wind. The design wind speed is based upon the basic wind speed, V, appropriate to the district in which the falsework is to be erected. The basic wind speed is the 3 s gust speed estimated to be equalled or exceeded on the average once in 50 years. Values for V should be selected from Figure 3. The design wind speed, V_s (in m/s), is given by the expression:

$$V_s = VS_1S_2S_3$$

where

V is the basic wind speed (in m/s);

S_1 is the topography factor for wind (see **4.5.1.4**);

S_2 is the ground surface conditions for height above ground factor for wind (see **4.5.1.5**);

S_3 is the statistical factor (duration of exposure) for wind (see **4.5.1.6**).

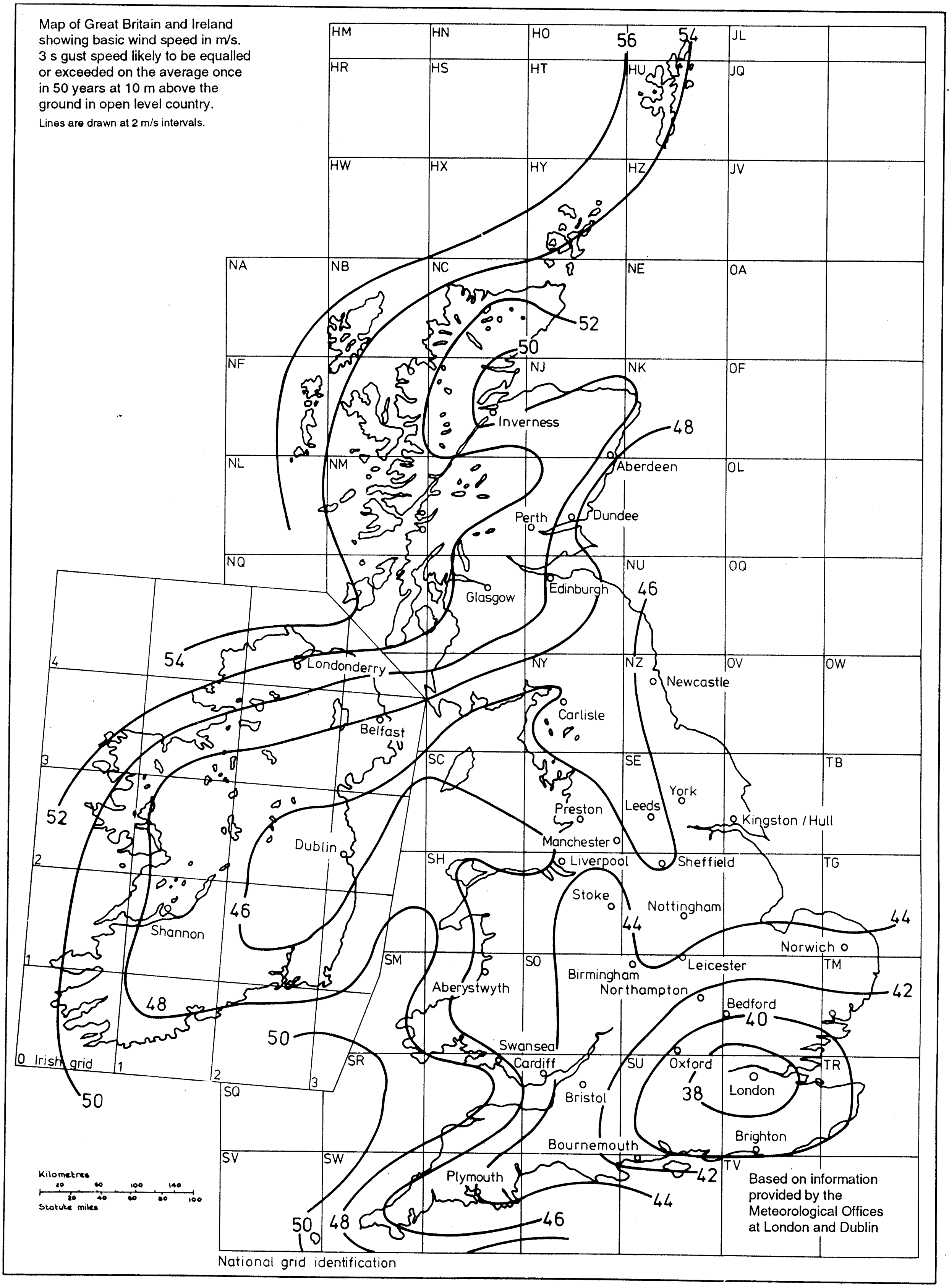
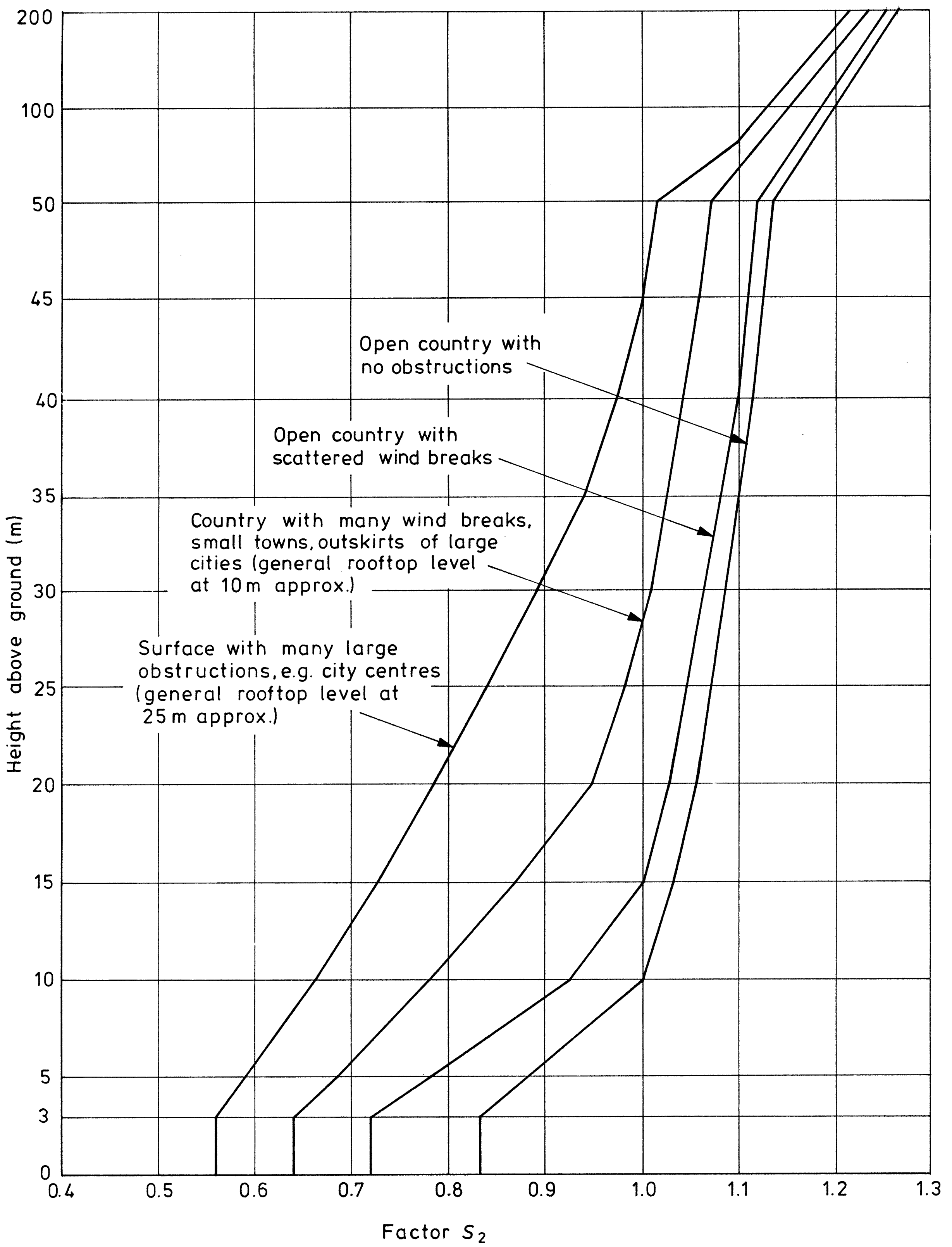


Figure 3 — Basic wind speed, V



NOTE This graph is based upon class A individual member values from CP 3:Chapter V-2.

Figure 4 — Ground surface conditions and height above ground factor for wind, S_2

4.5.1.4 ***Topography factor for wind,*** S_1

Unless special topographical effects are present the value of S_1 should be taken as 1.0. In very exposed sites and some valleys that are so shaped that funnelling of the wind occurs with accelerated wind blowing up the valleys, a value for S_1 of 1.1 should be used. In steep-sided enclosed valleys sheltered from winds, wind speeds may be less than normal and a value for S_1 of 0.9 may be adopted. Caution should be used when applying the reduced factor of 0.9.

4.5.1.5 ***Ground surface conditions and height above ground factor for wind,*** S_2

The factor S_2 takes into account the effects of ground roughness and the variation of wind speed with height above the ground. Values of S_2 may be estimated from Figure 4.

The height should either be taken on the top of the structure or the height may be divided into convenient parts and the design wind speed calculated using a value of S_2 corresponding to the height above ground of the top of that part.

The height above ground should be above the general level that can be expected to influence air movement and not measured from the local platforms or hollows. Where the falsework is to be erected on or adjacent to a cliff or similar discontinuity, reference should be made to Annex D of CP 3:Chapter V-2:1972 for specific directions.

4.5.1.6 ***Statistical (duration of exposure) factor for wind,*** S_3

The factor S_3 is based on the risk of winds of a given severity occurring during a given period. It will be seen from Table 12 that the factor varies with the design life of the falsework, and increases as the design life of the structure increases. Since the maximum design wind speed has a low probability of occurrence with a short construction period, it is recommended that a value for factor S_3 of 0.77 be applied as appropriate for falsework being erected and used for less than two years. Other values of S_3 should be taken from Table 12. No value for S_3 below 0.77 should be used.

4.5.1.7 ***Dynamic wind pressure,*** q

Having calculated V_s, the dynamic wind pressure to be used in subsequent calculations should be obtained from Table 13.

The greatest dynamic wind pressures applicable for all falsework erected for a maximum of 2 years and not exceeding 30 m in height are:

a) 1230 N/m^2, in England and Wales;

b) 1670 N/m^2, anywhere in the United Kingdom.

These greatest dynamic wind pressures are calculated from the expression for V_s in **4.5.1.3**, where:

V equals the highest basic wind speed in each of the areas;

S_1 equals 1.1 (very exposed site);

S_2 equals 1.1 (open country with no obstructions);

S_3 equals 0.77 (a two-year period).

Table 12 — Values of wind speed factor, S_3 (for probability level 0.63)

Life of the falsework years	**Wind speed factor,** S_3
Less than 2	0.77
2 to 5	0.83
5 to 10	0.88
over 10	1.00

Table 13 — Dynamic wind pressure, q, for various design wind speeds

Dynamic wind pressure q (in N/m^2), for a design wind speed, V_s (in m/s), of:										
V_s[a]	**0** N/m^2	**1.0** N/m^2	**2.0** N/m^2	**3.0** N/m^2	**4.0** N/m^2	**5.0** N/m^2	**6.0** N/m^2	**7.0** N/m^2	**8.0** N/m^2	**9.0** N/m^2
10	61	74	88	104	120	138	157	177	199	221
20	245	270	297	324	353	383	414	447	481	516
30	552	589	628	668	709	751	794	839	885	932
40	981	1 030	1 080	1 130	1 190	1 240	1 300	1 350	1 410	1 470
50	1 530	1 590	1 660	1 720	1 790	1 850	1 920	1 990	2 060	2 130
60	2 210	2 280	2 360	2 430	2 510	2 590	2 670	2 750	2 830	2 920
70	3 000									

[a] Example. The dynamic wind pressure for a design wind speed of 46.0 m/s is 1 300 N/m^2.

4.5.1.8 ***Maximum wind force during life of falsework, W_m, on windward face***

The maximum wind force during the life of the falsework, W_m (in N), acting on the windward face should be calculated from the expression:

$$W_m = qA_eC_f\eta$$

where

q is the dynamic wind pressure (in N/m^2);

A_e is the effective frontal area of the member or frame normal to the wind direction (in m^2) (see **4.5.1.9**);

C_f is a force coefficient related to the shape of the member or frame subjected to wind action (see **4.5.1.10**);

η is a shielding factor.

In the case of an unshielded windward face, η will have a value of 1. However, it is unrealistic to assume a full wind force on a falsework erected in an excavation or surrounded by walls as in basements or reservoirs, where the sides effectively shield it. Likewise, a falsework may be effectively shielded in some directions from a full wind force by abutments or already constructed walls. In such cases η should be taken as 0.5 and thus the calculated total wind force on the shielded part of the falsework and formwork may be reduced to one-half when considering the wind force in the shielded directions. **4.5.1.11** considers the effect of shielding on the force experienced by the inner members of the falsework structure.

The total wind force on the structure is the vectorial summation of the forces on the individual parts exposed to the wind. Attention is drawn to the circumstances dealt with in **4.5.1.13**, which recognizes an upper limit to the generated wind force. The load may be assumed to be uniformly distributed over the area under consideration.

4.5.1.9 ***Effective area subject to wind forces***

Since the wind can come from a number of directions, the most unfavourable load conditions should be taken. In most instances this will correspond to the wind acting in the direction normal to the surface of the member or frame.

The effective areas for the calculation of wind loads will comprise the shadow areas of:

a) individual members, e.g. tubes, beams, props;

b) frames, e.g. proprietary fabricated supports;

c) formwork, e.g. beam sides, soffits, wall forms, together with supporting primary and secondary beams of the falsework;

d) access ways, materials and plant carried by the falsework.

For square and triangular towers, the effective area is calculated for one face only (the force coefficients given in **4.5.1.10**, to allow for the shielding of the leeward sides, are overall force coefficients for the towers for all wind directions).

Where the falsework is braced, the effective area of horizontal and vertical members should be increased by 20 % to allow for bracing and fittings, unless specifically measured.

4.5.1.10 ***Force coefficient, C_f, for falsework***

The values of force coefficient, C_f, for falsework given in Table 14 apply to frames and towers where the geometric solidity ratio φ_w, which is the effective area of a frame (including bracing or bracing allowance) normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind, lies between limits of 0.025 and 0.225. When this geometric solidity ratio is outside these limits, reference should be made to CP 3:Chapter V-2.

Table 14 — Force coefficient, C_f, for falsework

Item	Shape	C_f
Individual members	Flat sided and channel members	2.0
	Circular section	1.2
Single frame	Flat sided	1.8
	Circular section	1.2
Square lattice towers	Flat sided	3.7
	Circular section	2.2
Triangular lattice towers	Flat sided	3.0
	Circular section	1.7
Universal beams and columns		1.6

4.5.1.11 ***Shielding factor, η for falsework***

The forces calculated in **4.5.1.8** give the full value of the wind force on the windward face of the structure. Should either the structure or a portion of it be sheltered from the full force of the wind, the values should be reduced by multiplying by a shielding factor, η.

Generally, for systems of adjustable props, framed towers and scaffolding as falsework, little shielding effect is obtained and η = 1.0.

Where there are multiple frames of similar geometry and spacing, and wind force on the third and subsequent frames should be taken as equal to that on the second frame. When summing the total wind force on the falsework structure comprising a large number of members in each direction there may be an upper limit to the wind force generated and guidance is given in **4.5.1.13**.

In other cases the shielding factor, η should be obtained from Table 15 using the appropriate values of the spacing ratio and aerodynamic solidity ratio, β. The spacing ratio is equal to the distance, centre-to-centre, between frames, beams or girders measured parallel to the wind direction divided by the least overall dimension of those frames measured at right angles to the direction of the wind. The aerodynamic solidity ratio, β,is equal to the geometric solidity ratio φ_w (see **4.5.1.10**), multiplied by a coefficient with a value of 1.6 for flat members, and 1.2 for circular and combinations of flat and circular members.

Table 15 — Shielding factor, η

Spacing ratio	Shielding factor, η, for an aerodynamic solidity ratio, β, of:							
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8 and over
Up to 1.0	1.0	0.96	0.90	0.80	0.68	0.54	0.44	0.37
Up to 2.0	1.0	0.97	0.91	0.82	0.71	0.58	0.49	0.43
Up to 3.0	1.0	0.97	0.92	0.84	0.74	0.63	0.54	0.48
Up to 4.0	1.0	0.98	0.93	0.86	0.77	0.67	0.59	0.54
Up to 5.0	1.0	0.98	0.94	0.88	0.80	0.71	0.64	0.60
Over 5.0	1.0	0.99	0.95	0.90	0.83	0.75	0.69	0.66

4.5.1.12 ***Wind forces on formwork and reinforcement***

4.5.1.12.1 *General*

The falsework will usually be required to resist the loads caused by the wind acting on the formwork or other items the falsework is supporting. The formwork may comprise edge, beam, or wall forms with associated support members. In general, when calculating the wind force it is convenient to consider the force as having two components, the one acting on the soffit and its associated support work, i.e. blowing under the formwork, the other acting on the edge forms and the upper surface of beam or wall forms. These components may be calculated separately and combined, together with the calculated wind forces on the falsework itself, to give the total wind force on the whole structure (see **4.5.1.13**).

Values for C_f, A_e, and η for use in the expression for W_m in **4.5.1.8** are given in **4.5.1.12.2** and **4.5.1.12.3**. The value for q will be that appropriate to the site. For the purpose of this subclause, formwork sloping at a gradient of not more than 1 in 20 should be treated as being horizontal.

4.5.1.12.2 *Wind forces on formwork prior to placement of concrete*

a) *Soffit and support with wind blowing parallel to secondary beam (see Figure 5).* The wind forces on a soffit and its associated supporting beams where the wind is blowing parallel to the secondary beam should be calculated using the following values:

C_f = 2.0
η = 1.0
$A_e = d_1 L_f$

where

d_1 is the depth from the top of the soffit to the bottom of the primary beam measured normal to the soffit (in m);

L_f is the frontal length of the soffit measured normal to the wind (in m).

b) *Soffit and support work with wind blowing parallel to primary beams (see Figure 6).* The wind forces on a soffit and its associated supporting beams where the wind is blowing parallel to the primary beams should be calculated using the following values:

C_f = 2.2
η = 1.0
$A_e = d_2 L_f$

where

d_2 is the depth from the top of the soffit to the bottom of the secondary beam measured normal to the soffit (in m);

L_f is the frontal length of the soffit measured normal to the wind (in m).

The effect of wind on the ends of the primary beams may be ignored.

c) *Edge formwork with two forms [see Figure 7a)].* Where there are only two edge forms, the wind force on each edge form, considered above the general soffit level, should be calculated using the following values for C_f and A_e

C_f = 1.8
$A_e = d_1 L_f$

where

d_1 is the height of the edge form under consideration, measured normal to the soffit (in m);

L_f is the frontal length of the edge form measured normal to the wind (in m).

The value of η will vary according to the degree of shielding of the forms. For the windward form, with no shielding, η should be taken as 1.0. For any leeward edge form η will vary in accordance with the shielding effect of the windward edge form. The shielding effect is dependent upon the spacing, L_w (in m), of the two edge forms, A and D, as follows:

1) when $L_w \leqslant 3d_1$ $\eta = 0$;

2) when $L_w \geqslant 8d_1$ $\eta = 1$.

Intermediate values for η should be interpolated linearly between these limits.

Regardless of whether or not the windward and leeward forms are of dissimilar heights, only the height of the windward form need be used when assessing shielding effects.

d) *Edge formwork with more than two forms [see Figure 7b)]*. For groups of more than two edge forms, the values for C_f and A_e should be taken as in item c). Where there are several groups of forms each group should be considered separately, e.g. in Figure 7b) there are four possible groups AB, BC, CD and AD and the shielding effect of spacings, $L_{w,1}$, $L_{w,2}$ and $L_{w,3}$, should be considered as for L_w in item c).

1) When estimating the extent of any shielding of form B, the values of $L_{w,1}$ and d_1 should be used for the calculation.

2) Where the distance $L_{w,3}$ is less than 1 m and/or where d_3 is less than $0.25d_4$, then the force on, and the shielding effect of, form C may be ignored, and the force on form D should be estimated for the group AD as in item c) (i.e. considering the shielding effect of d_1 and the spacing L_w). The value for C_f of 1.8 takes such instances into account.

e) *Combined beam and soffit formwork (see* Figure 8). Where the slab under construction incorporates beams, the wind force acting on such formwork should be calculated in a similar manner to that adopted for items c) and d), i.e. considering the forces on the side form faces and on the soffit and supporting beams as two separate items

1) Where the windward "beam 1" is an edge beam, or there is a length of slab cantilevering from it (see Figure 8a) and Figure 8b) respectively), the wind force on face A should be calculated using the following values:

C_f = 1.8

η = 1.0

$A_e = d_1L_f$

where

d_1 is the overall depth from the top of the edge form to the underside of the beam (in m);

L_f is the frontal length of the formwork measured normal to the wind (in m).

The force on form B should be calculated as for item c).

2) In the case of "beam 2", to the leeward of "beam 1", if the distance $L_{w,4}$ is less than $6d_3$, the force on this beam may be ignored. Where $L_{w,4}$ is greater than $6d_3$, the wind force on E or F (see Note) should be calculated using the following values:

C_f = 1.8

η = 1.0

$A_e = d_3L_f$

where

d_3 is the overall depth of the beam (in m);

L_f is the frontal length of the beam measured normal to the wind (in m)

NOTE The wind will act on either side E or F according to whether the wind is travelling over or under the soffit, i.e. whether the beam forms are upstands (see Figure 8a)) or downstands (see Figure 8b)). The value of 1.8 for C_f takes into account any forces on the other side of beam 2.

3) For subsequent beams to the leeward of "beam 2", the forces on each beam should be considered separately and in the manner recommended in item 2), taking $L_{w,4}$ as the distance between the beam in question and the adjacent windward beam.

4) The forces on the soffit and supporting beams should be calculated in accordance with items a) and b), as appropriate, and should be added to the values calculated in accordance with items 1) to 3) to obtain the total wind force on the combined beam and soffit formwork.

4.5.1.12.3 *Wind forces on formwork after placement of concrete*

The considerations in **4.5.1.12.2** apply prior to the placement of the concrete. Once concrete has been placed in the forms (as indicated by the dotted lines in Figure 7 and Figure 8) then the wind can be taken as acting on the outer forms only, and should be calculated as for **4.5.1.12.2** a), b) and c). The values for C_f take into account any negative pressure being developed on the downwind side of the leeward edge.

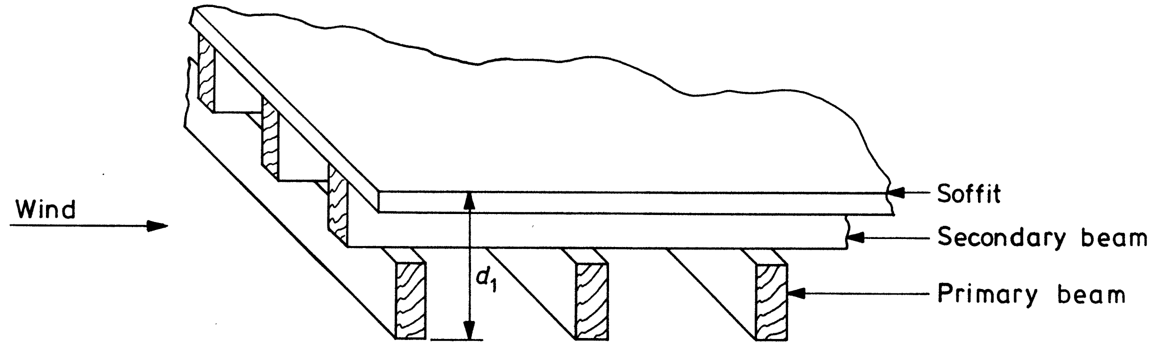


Figure 5 — Wind forces on soffit: wind blowing parallel to secondary beams

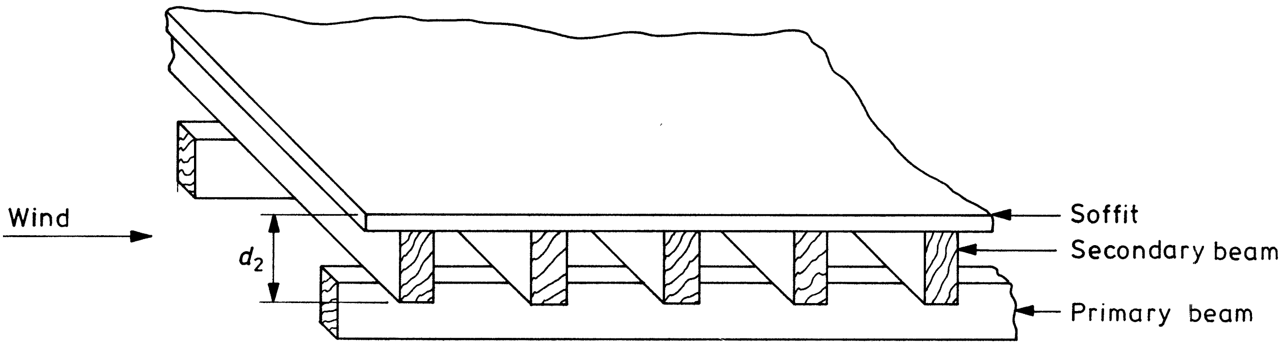


Figure 6 — Wind forces on soffit: wind blowing parallel to primary beams

4.5.1.12.4 *Wind force on reinforcement and completed sections of the permanent works*

The wind forces on erected reinforcement may need to be considered should such reinforcement present a larger frontal area to the wind than the associated formwork. Where construction takes place on the formwork as a number of distinct concreting operations, the wind forces on the completed sections should be taken into account when considering the wind forces acting on the formwork and falsework.

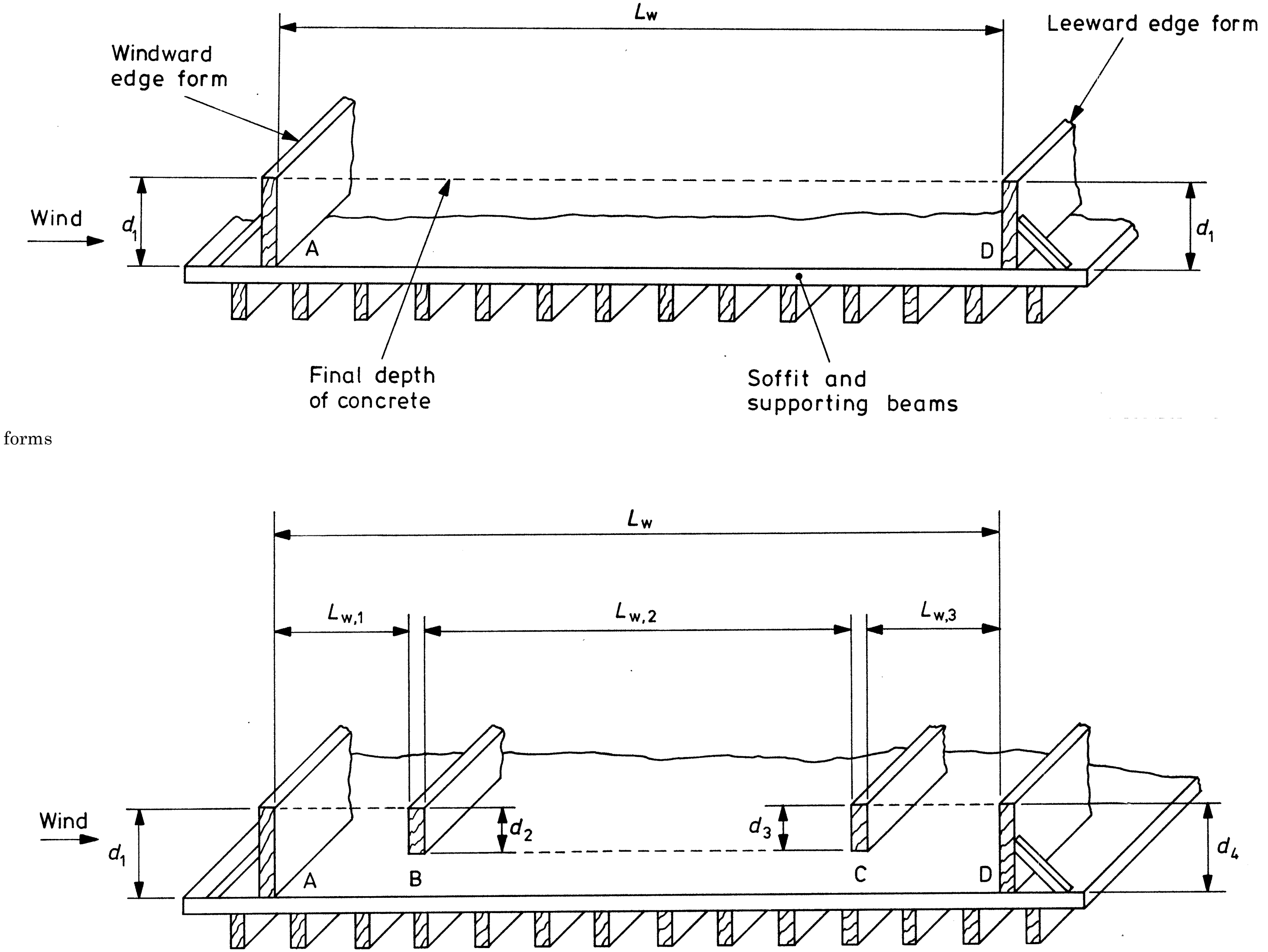
4.5.1.13 ***Upper limit to wind force***

Providing the wind is constrained to pass through the falsework and does not escape up through the top of the falsework or laterally out through the sides, it has been shown by tests and calculations that the maximum wind force cannot exceed that calculated for a fully sheeted windward face to the falsework. Provided that these conditions are met, the falsework and associated formwork can be regarded as a notional continuous impermeable face. The actual maximum wind force will not exceed the value of the area of this notional impermeable face multiplied by a value of $1.2q$, with a centroid of such a force at the height of $0.5\ (h_f + h_F)$ above the founding level (see Figure 9).

where

- q is the dynamic wind pressure (in N/m^2);
- h_f is the windward height of formwork (in m);
- h_F is the windward height of falsework (in m).

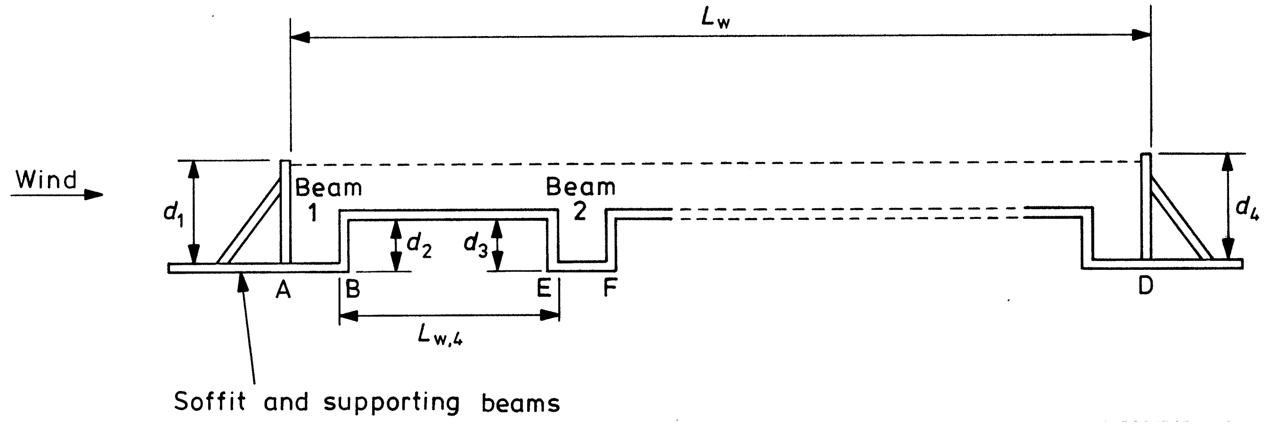
To this calculated force on this notional windward impermeable face should be added the loads generated on downstream parapets or similar caused by the wind crossing over the top of the assembly. This total force should be compared with that force calculated in accordance with **4.5.1.3** to **4.5.1.12** and the lower value used for the design of the falsework for those wind conditions.



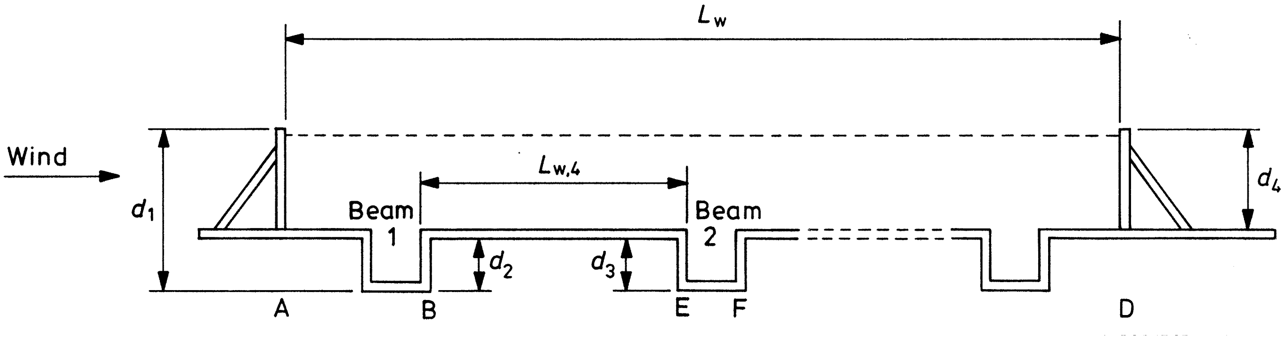
a) Two forms

b) More than two forms

Figure 7 — Wind forces on edge formwork



a) Formwork with upstands



b) Formwork with downstands

Figure 8 — Wind forces on combined beam and soffit formwork

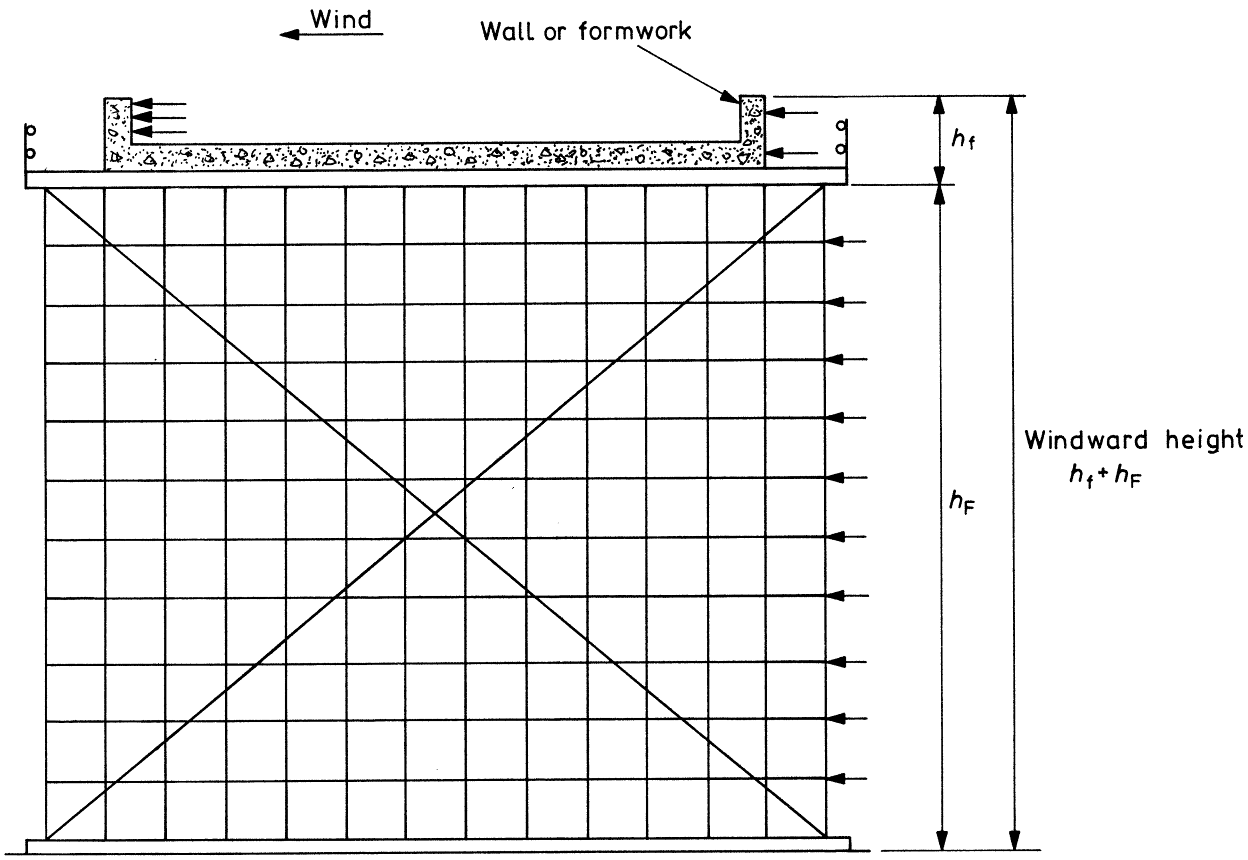


Figure 9 — Wind loading: combined formwork and falsework

4.5.2 Water

4.5.2.1 *General*

As far as possible, falsework should be founded in the dry.

Where it is necessary for falsework supports to be placed in flowing water, the effect of the forces caused by the flow should be considered. These will include:

a) dynamic pressure of water;

b) impact from floating objects;

c) increased frontal area, and head of water due to trapped debris.

The information necessary to permit these effects to be assessed may be obtained from a study of the local topography and hydrology and from the appropriate river, dock and harbour, coastal and local authorities.

The structure should be checked for flood conditions. Consideration should be given to the possible existence of buoyancy or uplift forces.

4.5.2.2 *Forces produced by flowing water*

The dynamic pressure of flowing water, q_w (in N/m^2), is derived from Bernoullis' equation and is given by the expression:

$$q \quad = 500\ V_w^2$$

where

V_w is the speed of water flow (in m/s)

The force due to the water flowing around falsework members, F_w (in N), is given by the expression:

$$F_w = q_w C_w A_w$$

where

C_w is the force coefficient for water appropriate to the falsework members under consideration;

A_w is the effective area normal to the flow (in m^2).

The following are some values of C_w:

1.86, for flat surfaces normal to flow;

0.63, for cylindrical surfaces;

0.03, for well streamlined surfaces.

NOTE A river flow of 3 m/s is usually described as moderately rapid.

Where there are successive rows of falsework members exposed to flowing water it is possible that some shielding protection is provided to the downstream members by those upstream. The following factors will contribute to the total force being applied to the falsework in the water:

a) the area of obstruction to the water flow presented by the first line of falsework members;

b) any further decrease in the width available for the passage of the water as it passes through the falsework;

c) the increase in obstruction to the flow that would result from the trapping of debris on the faces of the falsework.

Shielding may be taken into account providing the falsework is so arranged that a clear-cut water pattern is developed at the upstream members to provide protection deliberately to regular lines of falsework members in the direction of flow. Where such arrangements are made as a feature of the design, the total force calculated, may be reduced, in the case of the shielded members, by 20 %.

4.5.2.3 ***Trapped debris effect***

The accumulation of debris will produce a force on the falsework that may be calculated as for that on a rectangular cofferdam. This force, F_d (in N), is given by the expression:

$$F_d = 666 A_d V_w^2$$

where

A_d is the area of obstruction presented by the trapped debris and falsework (in m^2);

V_w is the speed of water flow (in m/s).

This effect will be dependent upon the rate of flow, the amount and nature of the floating debris, the nature of the obstruction and the depth of the water.

4.5.2.4 ***Wave action***

When falsework is erected in or adjacent to water, it may be subjected to wave forces. In marine locations this is a probability but elsewhere it is a possibility that should be considered. For further information see Annex F.

4.5.3 Snow

Depending upon seasonal, geographic and construction factors, snow may accumulate and form into drifts on the permanent and temporary works causing additional loading. Only in exceptional cases will this loading exceed the allowance of 1.5 kN/m^2 recommended in **4.4.3.1** and is therefore unlikely to present a problem in areas supporting permanent works, but ancillary areas should be checked.

NOTE The density of powder snow is approximately 80 kg/m^3. For further information see BS 6399-3.

4.5.4 Ice

In conditions of freezing rain or drizzle, melting snow, or fog or cloud at temperatures below 0 °C, ice may form on members, increasing their self-weight and effective frontal area to the wind.

In considering the effect of ice formation on the design of members of the falsework, the following should be taken into account:

a) the probability of appreciable ice formation is very small;

b) recorded ice formation in the UK relates to the months of November to March inclusive for short durations;

c) the small likelihood of ice formations occurring when erection is at the most critical stage (i.e. almost complete and awaiting its imposed loading);

d) no work will normally take place on falsework affected by the formation of ice of a thickness of greater than 1 mm owing to the hazard this presents to the operatives until such ice has been safely removed.

In the absence of more definite information, the following design criteria should be adopted:

1) maximum ice density is 920 kg/m^3;

2) maximum ice thickness surrounding members is unlikely to exceed 25 mm;

3) maximum gust speed to be used, where ice has formed, as the basic wind speed for design purposes, should be taken as 1.5 V_H, where V_H (in m/s) is the hourly mean wind speed at height H (in m) and is given by the expressions:

$$V_H = 15\,(H/10)^{0.17}$$

$$V_H \not> 28$$

NOTE A fuller presentation of the subject is given in Annex F of CP 3:Chapter V-2:1972.

4.5.5 Earth pressure

4.5.5.1 *General*

Earth pressures should be assessed using the formulae given in **4.5.5.2**, **4.5.5.3** and **4.5.5.4** for the particular limiting conditions stated. Alternatively, the design may be carried out in accordance with the recommendations of the Civil Engineering Code of Practice No. 2, prepared by the Civil Engineering Codes of Practice Joint Committee [9].

The formulae given in **4.5.5.2**, **4.5.5.3** and **4.5.5.4** neglect wall or skin friction giving higher active and lower passive pressure intensities.

Calculated pressures will only be mobilized where equivalent strains occur and these will be dependent upon the elastic modulus of the soil and of the structural element as interacting members. Thus the restraining effect of struts, ground anchors or tie-backs may create higher active pressures than those from the basic active condition formulae, possibly locally approaching those from the passive condition formulae.

The minimum active soil loading should not be less than that caused by a fluid having a density of 480 kg/m^3.

Where the soil is totally submerged, the soil pressure should be based on the submerged density together with full hydrostatic pressure. In general, values for the angle of internal friction, φ, do not alter significantly for the submerged condition.

4.5.5.2 *Cohesive soils*

a) *Active soil pressure*. The active soil pressure, p_a (in N/m^2), is given by the expression:

$$p_a = 9.81\,\gamma h - 2c$$

where

γ is the soil density (in kg/m^3);

h is the depth below ground level (in m);

c is the undrained shear strength (in N/m^2).

Often, the calculated active soil pressure will be negative and the minimum equivalent fluid pressure will apply (see **4.5.5.1**).

b) *Passive soil pressure*. The passive oil pressure, p_p (in N/m^2), is given by the expression:

$$p_p = 9.81\,\gamma h + 2c$$

where γ, h and c are as defined in item a).

c) *Soil density*. The soil density, γ,may vary from 1 600 kg/m^3 to 2 240 kg/m^3 for the normal condition, and from 640 kg/m^3 to 1 280 kg/m^3 for the submerged condition.

d) *Undrained shear strength*. The average value for the undrained shear strength, c, is normally between 5 kN/m^2 and 75 kN/m^2 for the depth range 0 m to 6 m.

1) For normally consolidated cohesive soils, i.e. recent and alluvial deposits, c should be based on the average undrained shear strength test results for the depth in question, but biased towards the low values. Values for c will generally range from 5 kN/m^2 to 40 kN/m^2 diminishing with depth, the higher values being associated with the desiccated crust.

2) For over-consolidated cohesive soils, i.e. deposits that have been subjected to loads in excess of those due to the present over-burden, c should be based on the undrained shear strength test results, but allowance should be made for:

i) duration of loading and unloading;

ii) drainage and ground water conditions;

iii) softening effects;

and, in general, very much reduced values for c should be used, probably of the order of half the average indicated cohesion for the depth being considered. Values for c will generally vary from 15 kN/m^2 to 75 kN/m^2.

4.5.5.3 *Cohesionless soils*

a) *Active soil pressure*. The active soil pressure, p_a (in N/m^2), is given by the expression:

$$p_a = 9.81\ K_a\ \gamma h$$

where

K_a is the active pressure factor, and for normal conditions is given by the expressions

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi}$$

$$K_a \ngtr 0.33$$

where

φ is the angle of internal friction (in degrees);

γ is the soil density (in kg/m^3);

h is the depth below ground level (in m).

b) *Passive soil pressure*. The passive soil pressure, p_p (in N/m^2), is given by the expression:

$$p_p = 9.81\ K_p\ \gamma h$$

where

K_p is the passive pressure factor, and for normal conditions is given by the expressions:

$$K_p = \frac{1 - \sin\varphi}{1 + \sin\varphi}$$

$$K_p \nless 3$$

where

φ is the angle of internal friction (in degrees);

γ and h are as defined in item a).

c) *Soil density*. The soil density, γ may vary from 1 400 kg/m^3 to 2 100 kg/m^3 for the normal condition, and from 1 000 kg/m^3 to 1 300 kg/m^3 for the submerged condition.

d) *Angle of internal friction*. The angle of internal friction, φ, may vary from 30° for loose material to 45° for dense material.

4.5.5.4 *Mixed soils*

For soils containing both granular and cohesive material extreme care is required in assessing soil parameters.

a) *Active soil pressure*. The active soil pressure, p_a (in N/m^2), is given by the expression:

$$p_a = 9.81\ K_a\ \gamma h - 2c\ \sqrt{K_a}$$

where K_a, γ, h and c are as defined in **4.5.5.2** and **4.5.5.3**.

b) *Passive soil pressure*. The passive soil pressure, p_p (in N/m^2), is given by the expression:

$$p_p = 9.81\ K_p \gamma h + 2c\ \sqrt{K_p}$$

where

K_p, γ, h and c are as defined in **4.5.5.2** and **4.5.5.3**

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Section 5. Foundations and ground conditions

5.1 General

5.1.1 Introduction

This section is only a guide to the assessment of critical properties of the ground conditions beneath the falsework and their potential effect on the safety of the falsework. In some cases, where ground conditions are not straightforward, it will be necessary to obtain the advice of an engineer experienced in geotechnics.

BS 8004 sets out the methods of assessment of ground engineering properties by site investigations and the importance of different properties to the behaviour of particular strata when subjected to loading, vibrations, variation in water content or environmental influence. It describes the different ways in which foundations can be provided in the ground to provide adequate safe bearing capacities and control of deflections or other movements. However, it should be noted that BS 8004 relates almost entirely to permanent foundations. This section of this code sets out some of the special considerations relating to falsework foundations.

5.1.2 Slope and stability

The slope of the ground may cause stability problems and it is recommended that the slope of the surface on which the falsework rests should not exceed 1 in 6 unless approved by a suitably qualified engineer specializing in geotechnics. The stability of the ground above and below the falsework site should also be considered.

5.1.3 Depth of foundations

Falsework foundations in general are set at a very shallow depth, compared with those of permanent structures, which brings them within the zone affected by seasonal moisture content changes, frost action, scour, etc. These are considered in **5.2** to **5.11**.

5.2 Site investigation for falsework foundations

It is essential that the type, depth, lateral and vertical variations of the ground underlying and adjacent to a falsework site be assessed to determine the safe bearing performance of the ground.

Initially, the nature of the soil and/or rock should be identified, and placed within one or more of the classifications given in Table 16. Some advice on identification and description of soils is given in Table 17, but for further information on the identification and classification of soils and rocks for engineering purposes, reference should be made to BS 5930.

In many instances there will be no doubt as to the uniformity of the soil and/or rock under the area on which the falsework will be founded and the strength and characteristics of the strata will be clearly adequate to receive the applied loads safely. In cases of variable strata, or where any doubt exists concerning the adequacy of the knowledge of the soil or rock below for the safe calculation of settlements under load, or the general stability of the site of which the area under or adjacent to the falsework forms a part, a site investigation involving borings and/or trial pits should be undertaken to obtain sufficient information to ensure the safety of the foundations to the falsework.

When a site investigation is undertaken for the permanent design, it is desirable that any additional data likely to be needed for the falsework should also be obtained. This will normally be the properties of the soils and rocks at shallow depths, up to one and a half times the width of the falsework structure. Essential properties of the soils and rocks are in situ density, moisture content and shear strength characteristics. Also requiring study are underground cavities, previous site use, mining and underground services. For further guidance on site investigation, reference should be made to Annex G and to BS 5930

Table 16 — Presumed allowable bearing pressure under vertical static loading

Group	Class	Types of rock and soil	Presumed bearing pressure kN/m^2	Remarks
1 Rocks	1	Hard igneous and gneissic rocks in sound condition	10 000	These values are based on the assumption that the foundations are carried down to unweathered rock (in the sense of the type described). A check should be made for discontinuities, e.g. bedding planes.
	2	Hard limestones and hard sandstones	4 000	
	3	Schists and slates	3 000	
	4	Hard shales, hard mudstones and soft sandstones	2 000[a]	
	5	Soft shales, soft mudstones and very soft sandstone	600 to 1 000	
	6[b]	Hard sound chalk, soft limestone	600[b]	
	7	Thinly bedded limestones, sandstones, shales	To be assessed after inspection on site. May act as class 14 or even class 18.	
	8	Heavily shattered rocks		
2 Non-cohesive soils	9	Compact gravel, or compact sand and gravel	$\geqslant$ 600	These values of bearing pressure are applicable to a width of foundation, *B*, not less than 1 m. Ground water level assumed to be a depth not less than *B* below the base of the foundation. Where ground water level may rise to within *B* below the base, adopt 50 % of the presumed bearing pressure.
	10	Medium dense gravel, or medium dense sand and gravel	200 to 600	
	11	Loose gravel, or loose sand and gravel	< 200[c]	
	12	Compact sand	> 300	
	13	Medium dense sand	100 to 300	
	14	Loose sand	< 100[c]	
3 Cohesive soils	15	Very stiff boulder clays and hard clays	300 to 600	Group 3 is susceptible to long-term consolidation settlement as well as immediate settlement. For foundations less than 1 m below the free surface of the ground, consideration should be given to the loss of strength resulting from softening.
	16	Stiff clays	150 to 300	
	17	Firm clays	75 to 150	
	18	Soft clays and silts	< 75	
	19	Very soft clays and silts	Not applicable. Require special foundation attention.	
4	20	Peat and organic soils	Require special foundation attention.	Large settlement.
5	21	Made ground or fill	Suspect, unless controlled as to materials and placement (see **5.9**).	

NOTE See modification factors in **5.5**.

[a] Weakly-cemented sandstones in the extreme may be crumbled by fingers and fall into group 5.
[b] Disturbed chalk deteriorates rapidly, giving very poor bearing value. There is also the danger of holes and caves.
[c] Large settlement could occur owing to vibration, e.g. pile driving or heavy traffic.

5.3 Testing of soils

Details of the tests commonly employed in site investigations should be in accordance with BS 1377. Tests are carried out in the laboratory on samples taken from boreholes or trial pits, or alternatively in situ. Suggested test programmes are given in Annex G.

Direct loading tests carried out on site are sometimes used as an alternative to the testing of samples or testing within boreholes.

5.4 Allowable bearing pressures

The maximum allowable bearing pressure for any soil or rock should be selected to provide adequate safety against ultimate failure and limit the settlement of the falsework to an acceptable value. Such settlement will be composed of immediate and long-term movements. This settlement will depend on the foundation soil and rock types, characteristics and variability, the plan dimensions of the structural foundation, and the intensity and nature of the applied load. However, settlements or movements may also occur owing to other factors such as adjacent structural loads, vibrations, adjacent excavation, undermining, scour, floods, variations in ground water level and even seasonal moisture content changes. The natural dip or jointing of strata or the natural fall of the ground surface may lead to creep or other movements of incipient instability.

Where a site investigation has been carried out it is possible, from a suitable test programme that has established the critical criteria for the properties of the ground, to evaluate the ultimate bearing capacity of the critical strata.

Where the ultimate bearing capacities of the critical strata have been directly established from a soils-testing programme, it may be possible to obtain advice on the allowable bearing pressure to be used in the design of the support to the falsework from the engineer in charge of the test programme. A factor of safety between two and three should normally be adopted.

When such a site investigation has not been carried out, the allowable bearing pressure for falsework can be derived from the presumed allowable bearing pressures given in Table 16. Table 16 is derived from Table 1 of BS 8004:1986 with some adjustments. The values relate to permanent foundations. Modification factors are given in Clause **5.5**.

The method by which the allowable bearing pressure has been determined should be clearly stated in the calculation sheets. A final check on the design assumptions should be made when the ground had been excavated, prior to foundation construction.

5.5 Modification factors applied to presumed bearing pressures

5.5.1 Modification factor for reliability of site information

In the case of homogeneous ground and where there is no test data, the values of presumed allowable bearing pressures given in Table 16 may be used for the support of vertical loads.

The excavation for the foundations of the permanent structure will frequently provide a check on the assumptions previously made and an opportunity to determine the nature of the ground conditions. Should no such opportunity be obtained and no other direct examination of the ground be possible, the presumed bearing pressures given in Table 16 should be multiplied by a factor of 0.75.

5.5.2 Settlements of the foundations underneath the falsework

In the design of the falsework full account should be taken of the effects of settlement of the foundations, both uniform and differential. Differential settlement can be caused by various conditions, such as variation in ground under the falsework when part of the falsework is being supported on the permanent work, and variations in loading. Both uniform and differential settlement of the falsework may also have an effect on the permanent works, especially during the initial set and early gain in strength of permanent works concrete, and can result in damage to these works.

Where the soil on which the falsework rests is susceptible to settlements and such movement would be of consequence to the falsework system, the presumed allowable bearing pressures given in Table 16 should be multiplied by a factor of 0.75 for soils in groups 2 and 3 in order to restrict the magnitude of the settlement. Where long-term settlement or a settlement of any magnitude is likely to prove unacceptable, a more detailed settlement analysis should be carried out.

Table 17 — Field identification and description of soils[a]

	Basic soil type	Particle size mm	Visual identification	Particle nature and plasticity	Composite soil types (mixtures of basic soil types)	
Very coarse soils	BOULDERS	200	Only seen complete in pits or exposures.	Particle shape: Angular Subangular Subrounded Rounded Flat Elongate	**Scale of secondary constituents with coarse soils**	
	COBLES	60	Often difficult to recover from boreholes.		**Term**	**% of clay or silt**
Coarse (non-cohesive) soils (over 65 % sand and gravel sizes)	GRAVELS	coarse 20	Easily visible to naked eye; particle shape can be described; grading can be described. Well graded: wide range of grain sizes, well distributed. Poorly graded: not well graded. (May be uniform: size of most particles lies between narrow limits; or gap graded: an intermediate size of particle is markedly under-represented.)		slightly clayey / slightly silty } GRAVEL or SAND	under 5
		medium 6			— clayey / — silty } GRAVEL or SAND	5 to 15
		fine 2		Texture:	very clayey / very silty } GRAVEL or SAND	15 to 35
	SANDS	coarse 0.6	Visible to naked eye; very little or no cohesion when dry; grading can be described. Well graded: wide range of grain sizes, well distributed. Poorly graded: not well graded. (May be uniform: size of most particles lies between narrow limits; or gap graded: an intermediate size of particle is markedly under-represented.)	Rough Smooth Polished	Sandy GRAVEL / Gravelly SAND } Sand or gravel and important second constituent of the course fraction	
		medium 0.2			For composite types described as: clayey: fines are plastic, cohesive: silty: fines non-plastic or of low plasticity	
		fine 0.06				
Fine (cohesive) soils (over 35 % silt and clay sizes)	SILTS	coarse 0.02	Only coarse silt barely visible to naked eye; exhibits little plasticity and marked dilatancy; slightly granular or silky to the touch. Disintegrates in water; lumps dry quickly; possess cohesion but can be powdered easily between fingers.	Non-plastic or low plasticity	**Scale of secondary constituents with fine soils**	
		medium 0.006			**Term**	**% of sand or gravel**
		fine 0.002			sandy / gravelly } CLAY or SILT	35 to 65
	CLAYS		Dry lumps can be broken but not powdered between the fingers; they also disintegrate under water but more slowly than silt; smooth to the touch; exhibits plasticity but no dilatancy; sticks to the fingers and dries slowly; shrinks appreciably on drying usually showing cracks. Intermediate and high plasticity clays show these properties to a moderate and high degree, respectively.	Intermediate plasticity (Lean clay) High plasticity (Fat clay)	— CLAY : SILT	under 35
					Examples of composite types (Indicating preferred order for description) Loose, brown, subangular very sandy, fine to coarse GRAVEL with small pockets of soft grey clay Medium dense, light brown, clayey, fine and medium SAND	
Organic soils	ORGANIC CLAY, SILT or SAND	Varies	Contains substantial amounts of organic vegetable matter.		Stiff, orange brown, fissured sandy CLAY Firm, brown, thinly laminated SILT and CLAY	
	PEATS	Varies	Predominantly plant remains usually dark brown or black in colour, often with distinctive smell; low bulk density.		Plastic, brown, amorphous PEAT	

[a] This table is based on Table 6 of BS 5930:1981.

Table 17 — Field identification and description of soils[a] *(continued)*

Compactness/ strength		Structure				Colour
Term	**Field test**	**Term**	**Field identification**	**Interval scales**		
Loose	By inspection of voids and particle packing.	Homo- geneous	Deposit consists essentially of one type.	**Scale of bedding spacing**		Red
Dense		Inter- stratified	Alternating layers of varying types or with bands or lenses of other materials.	**Term**	**Mean spacing** mm	Pink Yellow
			Interval scale for bedding spacing may be used.	Very thickly bedded	over 2 000	Brown Olive Green
				Thickly bedded	2 000 to 600	Blue
Loose	Can be excavated with a spade; 50 mm wooden peg can be easily driven.	Hetero- geneous	A mixture of types.	Medium bedded	600 to 200	White
		Weathered	Particles may be weakened and may show concentric layering.	Thinly bedded	200 to 60	Grey
Dense	Requires pick for excavation; 50 mm wooden peg hard to drive.			Very thinly bedded	60 to 20	Black etc.
				Thickly laminated	20 to 6	Supplemented as necessary with:
Slightly cemented	Visual examination; pick removes soil in lumps which can be abraded.			Thinly laminated	under 6	Light
						Dark Mottled etc. and
Soft or loose	Easily moulded or crushed in the fingers.	Fissured	Break into polyhedral fragments along fissures. Interval scale for spacing of discontinuities may be used.			Pinkish
Firm or dense	Can be moulded or crushed by strong pressure in the fingers.					Reddish Yellowish
Very soft	Exudes between fingers when squeezed in hand.	Intact	No fissures.			Brownish etc.
		Homo- geneous	Deposit consists essentially of one type.	**Scale of spacing of other discontinuities**		
Soft	Moulded by light finger pressure.					
Firm	Can be moulded by strong finger pressure.	Inter- stratified	Alternating layers of vary types. Interval varying types. Interval scale for thickness of layers may be used.	**Term**	**Mean spacing** mm	
Stiff	Cannot be moulded by fingers. Can be indented by thumb.					
Very stiff	Can be indented by thumb nail.	Weathered	Usually has crumb or columnar structure.	Very widely spaced	over 2 000	
				Widely spaced	2 000 to 600	
				Medium spaced	600 to 200	
Firm	Fibres already compressed together.			Closely spaced	200 to 60	
Spongy	Very compressible and open structure.	Fibrous	Plant remains recognizable and retain some strength.	Very closely spaced	60 to 20	
Plastic	Can be moulded in hand, and smears fingers.	Amorphous	Recognizable plant remains absent.	Extremely closely spaced	under 20	

[a] This table is based on Table 6 of BS 5930:1981.

5.5.3 Ground water levels

The presumed allowable bearing pressures given in Table 16 only apply where the ground water level lies at a depth below the foundation greater than the width of that foundation. Continued flooding or wet weather will soften clay soils. Where site flooding and/or high ground water levels are likely to be experienced the presumed allowable bearing pressures in Table 16 should be multiplied by the factor given in Table 18.

Table 18 — Ground water level modification factor

Condition	Modification factor for:		
	Cohesive soils	Non-cohesive soils	Rocks
Ground water level at *B*, or less, below level of foundation (where *B* is the width of foundation)	1.0	0.5	1.0
Site liable to flooding	0.67	0.5	1.0

5.6 Simple foundations on sands and gravels

Where lightly loaded foundations on sands and gravels are proposed, the allowable bearing pressure can be assessed from standard penetration tests and is based on settlements rather than shear failure since the latter is a rare phenomenon with granular soils. The exception to this is a foundation less than 1 m wide, where shearing may occur.

5.7 Simple foundations on cohesive soils

In cohesive soils, the allowable bearing pressure can be determined from shear tests. The following equations may be used; they incorporate a factor of safety against failure of 3, which is the value normally adopted.

a) *Strip foundations*. The allowable bearing pressure, q_b (in kN/m^2), is given by the expression:

$$q_b = 1.7c$$

where

c is the undrained shear strength (in kN/m^2).

b) *Square foundations*. The allowable bearing pressure, q_b (in kN/m^2), is given by the expression:

$$q_b = 2c$$

where

c is the undrained shear strength (in kN/m^2).

It is important to note that the allowable bearing pressures derived from these expressions are not linked to any particular values of settlement.

5.8 Heavy vibrations

Deposits or layers of granular materials, if not in a fully compacted state, are liable to consolidation and settlement if they are subjected to vibrations either from the falsework above, from adjacent operations (e.g. piling) or the passage of heavy traffic. Allowances for this possibility cannot be made by modification factors applied to the presumed bearing pressures. Either the granular materials should be compacted or the sources of vibration stopped for the critical period of the falsework.

Some uniformly graded sands and silts may be adversely affected by the vibration from compaction of concrete above the falsework.

5.9 Fill material

Where falsework is to be carried on filling of unknown origin or quality it should be investigated, since filling may have abrupt variations in composition, compaction and strength.

Where falsework is supported on a compacted fill whose properties have been determined, it is important to ensure that both the fill and the underlying ground are protected so that no disturbance or loss of material results from the movement of water or environmental changes (see **5.11**).

In cases where the filling material is variable in consistency and unable to receive and transmit loads uniformly, a minimum depth of 0.5 m of the fill should be removed and replaced by well compacted and stabilized granular material of known bearing capacity.

Allowable bearing pressures for the foundations of falsework on fully compacted controlled filling should not exceed:

a) 200 kN/m^2, for broken rock;

b) 150 kN/m^2, for well graded sands and gravels;

c) 100 kN/m^2, for uniform sands and hard shaley clays;

d) 50 kN/m^2, for firm to stiff clays;

e) 25 kN/m^2, for soft clay.

Poorly compacted or suspect material should be considered as being similar to a soft clay.

5.10 Piles

Where it is necessary to transfer the load from the falsework through weak strata to underlying strata of firm soil or rock, piling should be considered and a specialist experienced in assessing the pile type in relation to the soil conditions and in the driving and forming of the pile system required should be consulted. BS 8004 describes the several factors controlling the selection and design of piles and pile groups.

5.11 Protection of the foundation area

The area covered by the foundations under the falsework should be considered in relation to the general topography of the surrounding ground and the likelihood of outside influences affecting it and steps taken to safeguard it. The continued safe performance of the ground under and round the falsework foundations will depend on their remaining unaffected by:

a) local influences of water from water courses, extreme rainfall, melting snow or burst water mains;

b) severe frosts or excessively dry and hot weather;

c) movements of surrounding ground subjected to excavation, filling or other changes;

d) all pressures applied by other adjacent construction operations.

In many instances, parts of the foundations to the falsework incorporate concrete bedding and it is good practice to extend this concrete as a blinding layer over the entire area under the falsework and any surrounding areas used as working areas or that are susceptible to environmental conditions and which by virtue of changes could jeopardize the performance of the soils under the falsework.

The embedding of sole plates in both cohesive and non-cohesive soils will increase bearing capacity and give protection against the effects of scour, softening, frost and seasonal moisture changes. Where this is not feasible it is good practice to bed and haunch sole plates with concrete.

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Section 6. Design of falsework

6.1 General concepts

6.1.1 Introduction

The initial premise upon which those responsible for the final design of the falsework should base their scheme should be the previously agreed design brief (see **2.3** and **6.2**), although it should be recognized that it may prove necessary to accommodate alterations in the design when, for instance, the final details of the permanent works are known, or as experience is gained during the erection of the falsework.

6.1.2 Direction of falsework design

The preparation of design calculations, drawings and specifications should be undertaken in a manner similar to the methodic procedures (including checking) applied to the design of permanent works. This code provides guidance for systematic design, construction and checking procedures for falsework.

To facilitate the preparation of design calculations and their subsequent checking it is recommended that a sequence of presentation and format such as the Concrete Society's proposals [12] for the standard presentation of calculations be followed.

6.1.3 Checking the design

Prior to the commencement of any construction work, the proposed falsework design should be subject to a check for concept, adequacy and correctness (see **6.4.1**) by a person or persons independent from those directly responsible for the design. The ability of the checker and his remoteness or independence from the falsework designer should be greater where new ideas are incorporated or the falsework structure is complex. The check may be carried out in the same office by someone not involved in the design.

During the various stages of construction, those persons primarily involved with the design of the permanent and/or temporary works may wish to inspect the falsework structure, and it is often helpful if the falsework designer checks the work on site, owing to his familiarity with the design.

6.2 Design brief and basic approach

6.2.1 Design brief

This brief is the assembly of all the relevant data affecting the design of the falsework. It is important that it be assembled early to allow sufficient time for all subsequent activities.

The preparation of the brief may involve relatively little work in the case of the smaller scheme, but for a major work such as the construction of a large bridge, it is likely that a large amount of information will need to be collated before design work can commence or a programme for the construction of the falsework be drawn up.

The information required may need to be obtained from various sources, and these may include data from earlier site operations and from discussions with people having local knowledge. Certain information may be of direct relevance to both the permanent and temporary works, such as site conditions, or where the falsework takes advantage of existing permanent structures. The following list indicates the type of information that may be required for the preparation of the brief and typical (but not exhaustive) examples of information that should be collated for particular types of scheme are given in Annex H:

a) appropriate drawings of the permanent works;

b) programme for the construction of the permanent structure;

c) temporary support arrangements of short and long duration to be provided and safeguarded;

d) the timing for removal of the falsework and the ability of the permanent structure to become self-supporting at that stage;

e) special access requirements on to, under, and about the permanent works;

f) requirements for access to and about the falsework, both for falsework operations and for other site or public activities;

g) equipment and materials available for use in the falsework;

h) proposals for any moving and reuse of partly erected falseworks having regard to avoiding damage in the process;

i) data on rainfall, water-levels, current velocities, etc., that could influence the design of the falsework;

j) site investigation data and reports relating to the areas under and adjacent to the falsework, including information on all underground services;

k) any limitations to the staged construction of the works by positioning of construction joints, sequence of separate pours, rapidity of successive pouring, timing of post-tensioning, and removal of supports;

l) alternative methods for concreting or setting into position the members of the permanent structure;

m) any requirements for pre-cambering or residual cambering;

n) loads that may be induced on to the falsework from permanent works that have been completed including the application of staged post-tensioning, load redistribution and any movements of significance, including any settlements or deflections that can be anticipated from the permanent works as load is progressively increased;

o) the positioning and nature of loads being transferred from the falsework to elements of the permanent structure already constructed, e.g.:

1) proposals to support successive floors of multi-storey construction from lower floors;

2) proposals to use permanent foundations to support falsework;

p) the positioning of loads from falsework over underground services or adjacent to excavations or retaining walls forming part of the permanent works;

q) proposals for the protection of the falsework, including its foundations, against disturbance or damage by such events as impacts from loads or traffic or water-borne debris, or environmental conditions;

r) limitations applied by various authorities in relation to working within or adjacent to railway properties, watercourses, existing road systems and the like;

s) limitations applied with planning permission controlling the working hours, noise levels, etc.

6.2.2 Basic approach

6.2.2.1 *General*

Falsework should be designed in accordance with recognized engineering principles. The design methods given take into account the variability of materials, workmanship and site conditions that are within the recommendations of this code. The maximum allowable construction tolerances (see **7.3.2**) should be taken into account in the final design. Falsework usually comprises a multitude of simply assembled members of proprietary equipment or tubular scaffolding and the method of analysis should be based on an appropriate distribution of load between members (see **6.4**).

Falsework structures should be designed to meet the needs of the permanent structure they support and the actual conditions on site at the time of their use. Falsework systems should be designed with equal regard to ease and safety of dismantling as to the erection. For instance it should be possible to ease formwork away from the faces of the concrete by the operation of adjustable screw heads or the removal of specific striking pieces. Formwork should be readily disassembled to the “eased off” condition for handling. Where the falsework is to be immediately reassembled nearby, it is advantageous if the system can be moved in as large assemblies as possible with the plant economically available. In some cases an almost complete assembly may be rolled or skidded from location to location and in some others it may be crane handled after rolling out from under the newly constructed work. In all such cases the falsework should be designed to resist the forces and stresses resulting from these treatments.

It may be necessary to attach bracings specifically for this handling. Maintenance of the structure while still in position should be considered at the early design stage.

6.2.2.2 *Elemental failure*

Falsework usually comprises a structure with many components, junctions and connections. Careful attention should be paid to the way in which such connections and junctions are detailed, to reduce the dependence on individual workmanship and comprehensive site checking at these points. Detailing of the falsework structure should be such that any local failure of the structure does not lead to the progressive collapse of the whole.

6.2.2.3 ***Choice of falsework solution***

One of the first decisions concerning the falsework for any project is to decide whether it is appropriate to have a specific design or, alternatively, to proceed on the basis of a "standard solution". In this context, a "standard solution" involves the selection of a suitable combination of support components for which the basic design work has already been carried out and presented in a tabular or other easily assimilated form, and for which no further structural calculations are necessary. When selecting a "standard solution", those responsible for making the final choice should ensure that they understand and take full account of the limitations of these designs so that they will only be used in appropriate circumstances. The choice of a "standard solution" may be influenced by such matters as availability of material and the particular experience of the supervisory and construction work force.

A limited range of "standard solution" is given in Section 8 together with essential information that should be taken into account. The design offices of contractors or suppliers or the like may also produce "standard solution" to suit their materials or operations. Such designs should be produced in accordance with the recommendations in this code and should also be accompanied by information similar to that in Section 8, covering layout, loading, limitations, tolerances, etc., together with instructions for safe erection and dismantling.

6.2.2.4 ***Type and arrangement of falsework***

A number of different concepts of support are likely to be economical and merit detailed consideration on each project. These include:

a) a support provided by tubular scaffolding or extending props;

b) a support formed of a multitude of proprietary braced towers or systems;

c) heavy duty vertical support members to reduce the numbers and with stronger distribution members;

d) supporting towers with beams or girders at high level;

e) clear spanning beams or girders between the permanent columns or abutments above or below the level of the permanent works.

Other approaches that may be relevant are:

1) staged construction of the permanent works to control the load applied to the falsework;

2) launching or lifting the permanent works in appropriate form on to the falsework;

3) falsework supported from the permanent works.

Apart from the structural design to deal with applied loads, further consideration may be necessary for moving assembled structures such as table forms and bridging units.

6.2.2.5 ***Selection of materials and components***

It is necessary to identify the properties of the materials and the components that it is proposed to use. These should comply with the recommendations of Section 3.

When designing for the use of proprietary equipment the design should normally utilize the equipment in accordance with the current recommendations of the supplier of the equipment being used. Where it is proposed to use the equipment in situations not specifically intended by the supplier, it is recommended that in the first instance the supplier is consulted for specific advice about the proposed method of use. Where this information is not readily available, or as an alternative means of obtaining the required data, the user should assess the capability of the equipment for the new loading conditions using normal engineering principles and, where appropriate, arrange for testing of components for the purpose of checking the proposed design. Where testing of materials or components needs to be carried out this should be indicated in the information provided with the proposed design.

6.3 Forces applied to falsework

6.3.1 General

6.3.1.1 *Introduction*

The forces applied to falsework should be calculated and assessed from the loads as described in Section 4.

It is not necessary to design for every combination of adverse loading conditions providing the work on site is sufficiently controlled to avoid particular adverse combinations, e.g. extreme wind forces need not be considered in combination with dynamic loads from crane handled units.

6.3.1.2 *Vertical forces*

Vertical forces at the various stages will include the self-weight of falsework, imposed loads, permanent works loads and construction operations loads.

6.3.1.3 *Horizontal forces*

6.3.1.3.1 *Wind forces*

The forces applicable for the extreme wind conditions should be evaluated as described in **4.5.1**. This gives a value for W_m, the maximum wind force during the life of the falsework (i.e. for the extreme condition).

A maximum working wind force during operations, W_w, is assessed as the maximum wind speed during which working operations can take place and is normally limited to that of a wind force, on the Beaufort Scale, of 6. This corresponds to a design wind speed, V_s, of 18 m/s and gives a dynamic wind pressure, q, equal to 200 N/m^2. The values of force coefficient, effective frontal area and shielding factor will be the same as used for the evaluation of the extreme wind condition.

The forces from both of the above conditions should be used to check the stability of the structure at the appropriate phases of construction.

6.3.1.3.2 *Forces resulting from erection tolerances*

The acceptable erection tolerances on nominally vertical members defined in Section 7 result in horizontal reactions in association with the applied vertical forces. Each reaction should be assumed to act at the point of application of the vertical force. The magnitude of each force depends upon the amount of out-of-plumb of the particular member, and the total force on a group of members is the vectorial summation of the individual horizontal forces; these forces are random in direction and consequently the resultant will be less than the arithmetic total.

Provided the maximum permissible erection tolerances recommended for individual tubes, structural steel sections, and proprietary components used as support towers (see **7.3.2.3** to **7.3.2.5**) are not exceeded, provision should be made for a horizontal reaction equal to 1 % of the applied vertical forces at the point of application of the vertical forces. Where these conditions do not apply allowance should be made either by increasing the horizontal reaction provision or by treating as out-of-vertical by design (see **6.3.1.3.3**).

The provision for forces arising in construction using props should be based on the erection tolerances described in **7.3.2.2**.

6.3.1.3.3 *Forces resulting from members out-of-vertical by design*

Falsework members (e.g. beams or supports) may be designed to follow gradients or profiles and the members installed out-of-vertical by design. The vertical forces transmitted by the members will give horizontal components that require to be resisted.

The horizontal force components will be in addition to any associated horizontal force arising from the erection tolerance. Annex J provides further information on forces related to sloping soffits.

6.3.1.3.4 *Concrete pressures*

The pressures of fresh concrete on formwork should be calculated as described in **4.4.2**. The fresh concrete applies modified hydrostatic pressures acting normally to the face of the formwork. Where any of the formwork surfaces are supported by falsework, the pressures on the formwork may be transmitted to that falsework. The horizontal forces on the opposing formwork surfaces may be resisted within the formwork system by tying opposite faces together. The resultant vertical load will be the self-weight of the concrete that has to be borne by the supporting falsework. Where the opposite faces are not adequately tied together, the separate face forces will be transferred to the falsework. It is important that individual formwork panels forming the soffit are also adequately restrained against being separated by horizontal forces.

A similar horizontal force arises when concrete is placed against an existing vertical surface, e.g. a construction joint. This force should be taken into account (see Figure 10).

Where the soffit is not level, the concept is more complex and is discussed in Annex J.

6.3.1.3.5 *Water and wave forces*

Where falsework is subjected to water and wave forces these should be evaluated as outlined in **4.5.2**.

6.3.1.4 ***Dynamic and impact forces***

The effects of dynamic and impact forces on falsework should be evaluated and allowed for in the design. The magnitude of such forces is given in Section 4. Where possible, such impact forces should be minimized or avoided (see **4.4**). It is always preferable to prevent accidental impacts from occurring rather than to strengthen the falsework to resist them.

6.3.1.5 ***Forces from the permanent structure***

Movements of the permanent structure against which the falsework is in contact may be the result of:

a) thermal movement, shrinkage or creep;

b) settlement of the permanent foundation as the load increases;

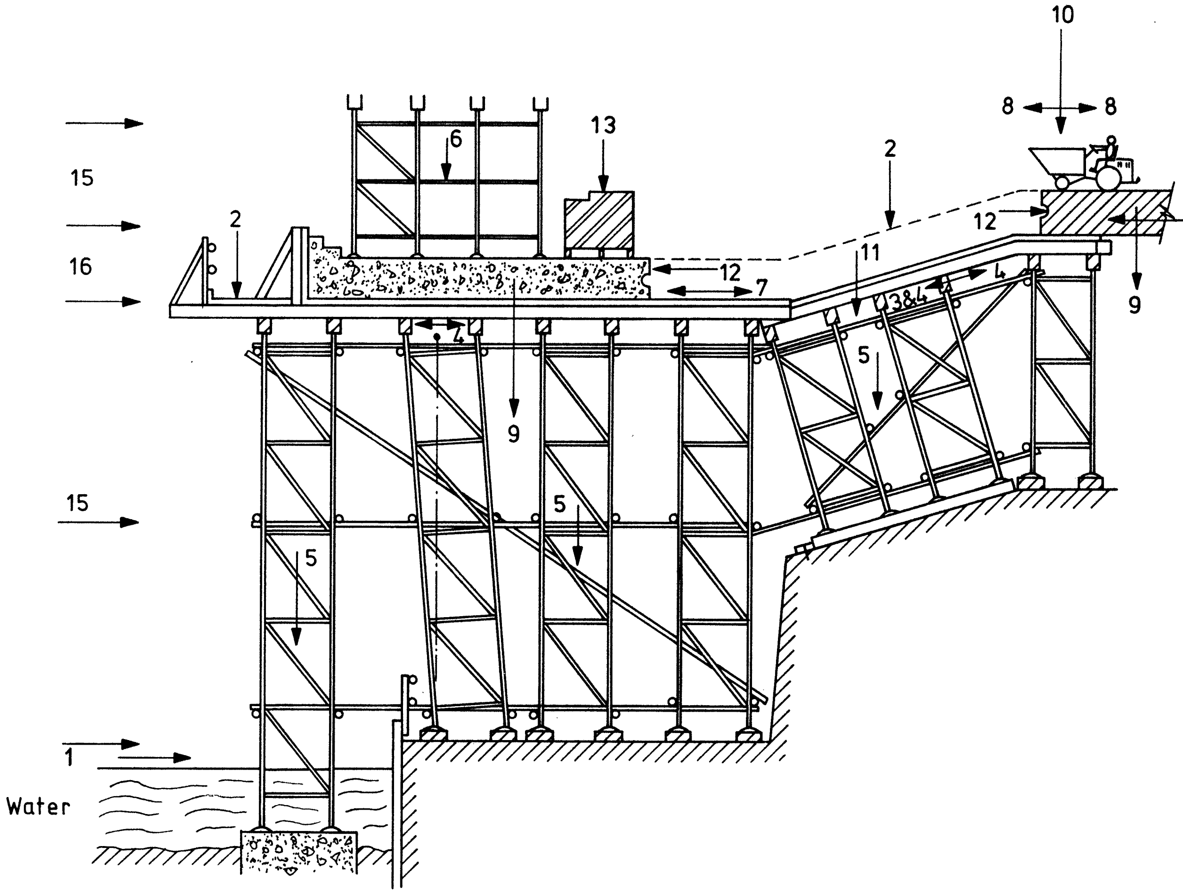
c) post-tensioning of the permanent works;

d) redistribution of loading as the structure acts compositely on completion of a section.

Such movements are exceptional circumstances to which falsework will not usually be subjected (see **6.2.1** and **6.4.5.4**).

6.3.2 Combinations of forces

Account should be taken of the varying combinations of forces that can be applied to the falsework at different phases of construction. A typical arrangement of forces is shown in Figure 10



Key

1. Wave forces
2. Working area loads
3. Out-of-vertical by design
4. Erection tolerance
5. Self-weight of falsework and formwork
6. Self-weight of later stage of falsework
7. Horizontal friction between concrete and soffit formwork
8. Impulse loads
9. Load from cast concrete
10. Mobile plant load
11. Applied permanent load, e.g. from concrete to be cast
12. Reaction from active concrete pressure against completed work
13. Load from stacked materials
14. Force from permanent work, e.g. thermal movement forces
15. Wind forces on falsework (either maximum or working)
16. Wind forces on formwork

Figure 10 — Typical force combinations

6.4 Analysis of falsework structure

6.4.1 General

The falsework solution chosen for a particular structure may be subjected to several of the forces outlined in **6.3**, the magnitude of which may vary at different phases of construction.

It is recommended that for each falsework structure three design checks be carried out:

a) on the structural strength of the individual members and the connections to transmit the applied forces safely;

b) on the lateral stability of both the individual members and the structure as a whole;

c) on overturning of the falsework.

Within these three checks, subsidiary checks may be necessary to cater for the different phases of construction and the varying stability and restraint conditions.

One of the major differences between the design of falsework structures and permanent works structures is that the falsework is not normally held down to the foundations and usually relies upon its own inherent stability and self-weight to remain stable throughout its life. Furthermore, because the loads imposed on a falsework structure are known within close limits, the individual elements of a falsework structure are more likely to be stressed to nearer their permissible working stresses than the permanent structure on which imposed loadings may vary considerably.

6.4.2 Behaviour of structure

A tiered falsework structure made up of a number of vertical supports or frames requires to be analysed with the loads acting at their points of application. It is not correct to assume that the entire structure acts monolithically. It is suggested that in the case of tiered structures each individual tier is considered and the applied forces resolved in the supporting and restraining members. This simple method of analysis will be adequate for the majority of multiple support falsework structures.

Allowance is not normally made for stiffness of the formwork unless it has been designed and constructed as an integral part of the falsework to contribute to its strength and stability.

Where continuous beams are used in falsework, a rigorous analysis is rarely justified because of the uncertainties in the assumptions made.

Movements may be expected when the loads imposed by the permanent construction are applied to the falsework. These will be the sum of consolidation settlements of the underlying soils, the take up at the individual interfaces in the falsework assembly and elastic movements.

The consolidation of the foundations and underlying soils should be predicted by the designer of the falsework foundations. Within the falsework, an allowance of 0.5 mm may be necessary at each non-rigid connection of steel vertical members one above another, and of 1 mm wherever the member receiving load is of timber. Account should be taken of the effects of any movement of the falsework during concreting on the integrity of the newly placed concrete and its final profile.

The locating of falsework on to the foundations of the permanent works or adjacent to them may induce some movement of those foundations. The construction of additional parts of the permanent works may also result in differential settlements of, or adjacent to, the falsework. The application of loads into the permanent works supported by the falsework or adjacent to it can also result in movements and redistribution of loads that will need to be taken into account in the falsework design.

6.4.3 Structural strength

6.4.3.1 *Element design*

The postulated falsework structure should be analysed to find the parameters that need to be taken into account in the selection of the individual elements. These parameters include the relevant end loads, bending moments, shear forces, bearing stresses and deflection characteristics.

In the design of vertical members supporting continuous beams, the reaction used should take into account the effects of continuity. In cases where random lengths of bearers are used, such as in a bridge deck with primary and possibly secondary bearers, a detailed analysis would involve the interaction of many indeterminate factors. It is recommended that in such cases the reactions are analysed for all the bearers as simply supported spans and the resulting reaction at the falsework support increased by 10 % to allow for the variations of the transferred loads.

NOTE Each load should be increased by 10 % once only.

Where specific conditions can be identified, such as beams on three supports, a more detailed analysis may be justified.

Wherever possible, when detailing the individual elements, the maximum tolerances permitted for workmanship on site as described in Section 7 should be adopted and allowed for in the element design. For items listed in **7.3.2**, the allowable safe working stresses used in conformity with the recommendations of this code will be adequate.

Where the eccentricity at the point of transfer of loads between components exceeds the tolerances recommended in **7.3.2**, the connection and the resulting forces should be evaluated and provisions made for them.

Where high concentrations of loading occur on relatively light members, any problems of local instability due to web crushing and buckling become more critical. The methods of analysis are outlined in Annex K.

Consideration needs to be given to the careful control of the effective lengths of members acting as struts and Annex L gives guidance on the effective lengths that should be used.

The allowable stresses recommended in Section 3 may be exceeded by 25 % in cases where an increase in stress is solely due to wind forces, provided that the sections should be not less than those needed if the wind stresses were neglected.

6.4.3.2 ***Factors of safety***

The factors of safety recommended in Section 3 allow for the falsework structure to consist of reusable components that will have a short-term loading in a particular falsework structure, with the resulting lesser risks that apply to such loadings when adequate control is provided.

Falsework designs may require further consideration to avoid excessive deflections, settlements, or distortions that could affect the quality of the permanent work.

Falsework structures do not always have to be designed to withstand the most adverse conditions. In cases where no persons or other property would be exposed to risks from damage occurring under extreme environmental conditions, it may be economic to design for normal loading and environmental conditions instead of the extreme conditions (see also **6.3.1**).

6.4.4 Lateral stability

6.4.4.1 ***General***

To ensure the lateral stability of falsework structures, including beam grillages, they should be designed to be able to resist at each phase of construction the applied vertical loads and the greater of either:

a) horizontal forces equivalent to 2.5 % of the applied vertical loads considered as acting at the points of contact between the vertical loads and the supporting falsework; or

b) the horizontal forces that can result from wind, erection tolerances, non-verticality, concrete pressures, water and waves as described in **6.3.1.3**, dynamic and impact forces as described in **6.3.1.4** and the forces generated by the permanent works as described in **6.3.1.5**.

The horizontal forces need to be resisted. It may be possible for the forces to be transmitted horizontally to a completed part of the permanent structure that has attained sufficient strength to safely resist such forces. Consideration should be given to the ability of the permanent works to resist these loads.

6.4.4.2 ***Bracing to transfer forces***

The transfer of horizontal forces to the ground or other foundations may be by inclined ties and/or struts. The designs of struts should be checked as outlined in **6.4.4.3**. Where such inclined bracing members are transmitting horizontal forces, there will be components of force from the braces transmitted into other members. The forces in the adjoining members will be the vectorial summation of the applied loads and the components of the bracing forces. Should the bracing be transmitting wind forces, the permissible stresses in the members can be increased as stated in **6.4.3.1**.

In a falsework structure composed of a number of props, tubes or frames, the horizontal forces may be collected by horizontal lacing that transmits the forces to the braced bays of falsework. These lacing members should be designed to transmit the forces into the bracing. The application of horizontal forces to tall structures may give rise to tensile forces in vertical members and the connections should be designed accordingly.

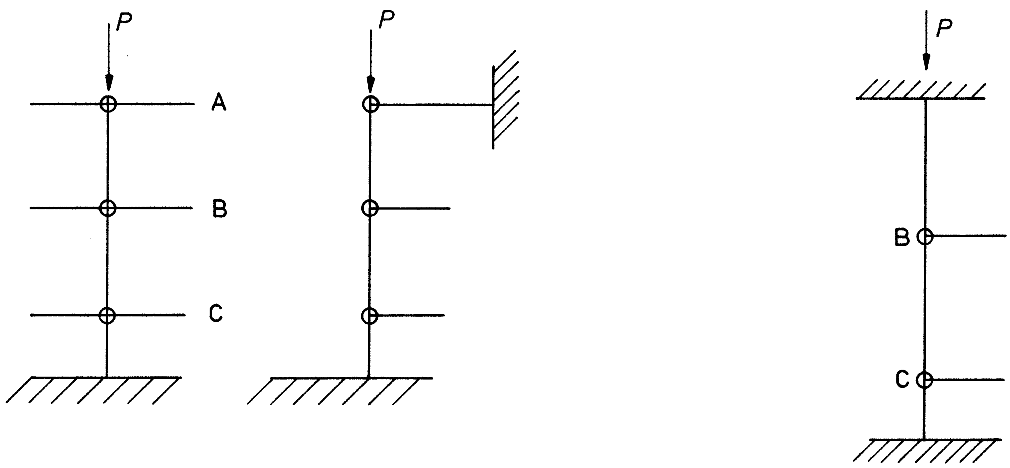
6.4.4.3 ***Bracing to maintain node point positions for struts***

Within falsework structures the effective lengths of struts may be reduced by introducing points of restraint within the length of the strut. A point of restraint will normally be achieved where there is lacing or bracing in two directions to that point, usually called a node point. The lacing or bracing to any node point should resist a transverse shear force equal to 2.5 % of the force in the strut at that point. Typical examples are given in Figure 11.

The transverse shear force is in accordance with **35**a) of BS 449-2:1969 and is a notional force used to check that the lacing and bracing will prevent movement of the node point. Generally, the use of the minimum recommendations for lateral stability of **6.4.4** will give an adequate design to satisfy the recommendation as the transverse shear force is not considered as additional to the forces in the lateral stability lacing and bracing. The exception is where the horizontal forces of **6.3.1.3** are not transmitted through the falsework but are resisted by connection to the permanent works. Bracing may still be necessary to stabilize the strut.

For members carrying bending stresses, the lacing should be proportioned to resist any shear due to the bending in addition to the transverse shear force. Where there are several node points in the length of a strut the transverse shear force will be constant. The lacing to the node points will be similar as the load is notional and not cumulative down the length of the strut.

Where a number of similar struts are restrained at common points with linking members, the effective transverse shear force on the lacing system only is cumulative. The force should be collected and transmitted to an adequate anchorage point, e.g. the foundation and/or the bracing.



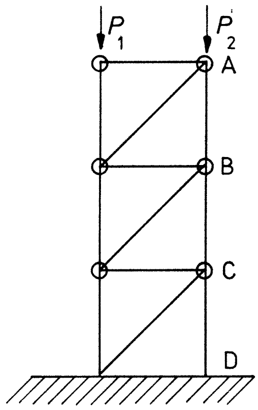
Node points at levels A, B and C are effective if the lacings at A, B and C are capable of transmitting an applied force of 0.025P separately to suitable fixing or bracing.

a) Several nodes in a free standing strut

Node points can be considered at B and C if the lacings at B and C are capable of transmitting an applied force of 0.025P separately to suitable fixings or bracings.

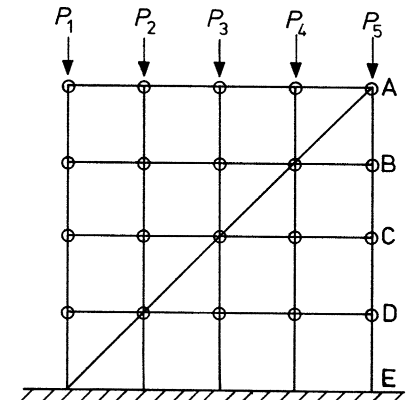
b) Several nodes in struts with fixity at head

Figure 11 — Typical examples of stability of node points



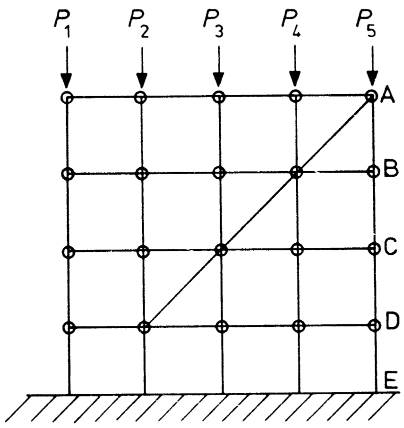
Node points at levels A, B and C are effective if the lacing and bracing from A to D is adequate to transmit a horizontal applied force of 0.025 ($P_1 + P_2$) applied separately at levels A, B or C.

c) Free-standing tower



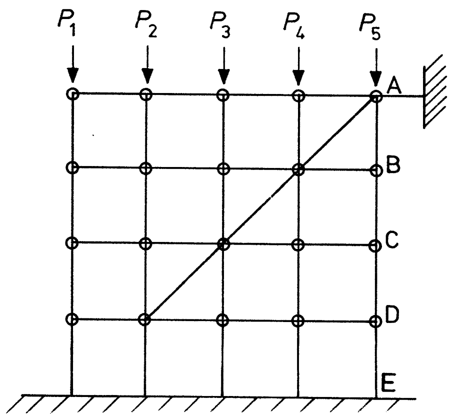
Node points at levels A, B and D are effective if the lacing and bracing from level A to E are adequate to transmit a horizontally applied force of $0.025\ \Sigma\ \frac{1}{5}\ P$ at levels A, B,C or D.

d) Free-standing falsework: Multiple support



Node points at levels A, B, C and D are effective if the lacing, and bracing from level A to D, are adequate to transmit a horizontally applied force of $0.025\ \Sigma\ \frac{1}{5}\ P$ and each strut from D to E is designed for the associated axial force P and a horizontal force of $0.025P$ from D to E in bending.

e) Free-standing falsework: Multiple support with bracing not taken to base



Node points at levels A, B, C and D are effective if the lacing and bracing from levels A to D are designed for a horizontally applied force of $0.025\ \Sigma\ \frac{1}{5}\ P$. In this case the struts D to E are designed for the axial load P only.

f) Falsework fixed at head but standing on struts not braced to base

Figure 11 — Typical examples of stability of node points (*concluded*)

6.4.4.4 *Bracing during erection and dismantling*

Bracing should be included in the falsework as it is erected to ensure the stability of the structure and the safety of people working on or adjacent to it. The loads involved during erection are generally small but the incorporation of bracing as the erection proceeds will assist in the correct positioning of components. Where the final bracing cannot be installed as erection proceeds, careful consideration should be given to the stability at each stage of erection.

When considering the stability of the falsework during erection and dismantling proper attention should be given to the provision of safe access on to and about the falsework at appropriate levels. The effects of wind forces on access ways should not be overlooked. The security of soffit formwork against uplift wind forces should also receive consideration. The design should be checked to ensure that the bracing is adequate when formwork or the like is being assembled on top of the otherwise unloaded falsework and subjected to wind.

Care should be taken to maintain safe working conditions during the dismantling of the falsework. It may be necessary to specify the sequence of dismantling and to indicate any members that are not to be removed until later. Some permanent works are so flexible that as the falsework is removed there is an increase in the loads acting on the remaining falsework and in such cases the sequence of dismantling should be defined. This consideration may apply to horizontal forces and bracing as well as vertical support members.

6.4.4.5 *Bracing to beams and lattice girders*

In clear span falsework systems the spanning members and/or their bracing will be required to transmit all the horizontally applied forces and components of forces to the supports at each end of the members. The calculation of these forces has generally been covered in **6.3**. In addition to these forces, the compression flanges of the spanning members may also need to be braced to ensure stability. For the smaller beams, such as telescopic floor centres, the top compression flange is generally adequately restrained by the friction connection of the formwork above the floor centres. In the case of beam and lattice girders the calculation of the magnitude of the compression flange restraint force should be as given in Annex L. This force is taken as being divided equally between the number of points at which restraint occurs.

Where beams and lattice girders are installed out-of-vertical by design, a component of the vertical load will be induced into the lateral bracing between girders.

The bracing may be required to transmit both the compression flange restraint forces and all the horizontally applied forces and out-of-vertical components at the appropriate phases of construction.

Concrete placing will not be carried out under extreme wind conditions, and immediately after casting of the concrete, the structure will generally have become a stiff horizontal diaphragm connected to suitable restraint. Beams and lattice girders in such a structure should be braced to withstand the greater value of either:

a) the working wind condition with full applied imposed loads giving rise to the maximum compression flange restraint forces and out-of-vertical components; or

b) the extreme wind force in conjunction with the compression flange restraint force calculated using the self-weight of the falsework, formwork and reinforcement, and the loading from construction operations; this corresponds to the condition at the final stage of erection.

Where the part-completed structure is not restrained, e.g. where the deck to a bridge is not continuous and infill bays are being cast at a later stage to connect the deck to the abutment, the bracing will require to withstand the extreme wind force and any out-of-vertical component, together with the maximum compression flange restraint force and any other laterally applied force.

Vertical diaphragm bracing may be required where the beams or lattice girders are erected out-of-vertical by design. It may also be required to transfer wind forces from one flange into the main bracing provided in the plane of the outer flange. Where beams or lattice girders are supported off their lower flanges, the vertical diaphragm bracing at the supports will also transmit the full upper flange bracing forces into the supports.

On long spans an additional flange bracing system may be required to transmit the wind forces to the supports.

6.4.4.6 ***Lateral restraint provided by friction***

Local restraint to lateral forces may be transmitted by static friction between contact surfaces. When two members are in contact and transmitting forces, there is a constant relationship between the value of the normal reaction and the force at which frictional lateral restraint is just overcome so that the members slide one on the other. This relationship, shown in Figure 12, is known as the coefficient of static friction, μ, and is independent of the area of contact when the normal reaction is the same, and also assumes no indentation or mechanical connection of the members. The frictional force always opposes the motion. (See also Annex J.)

The coefficient of static friction is given by the expression:

$$\mu = \frac{F_f}{R} = \frac{W_{\sin}\theta}{W_{\cos}\theta} = \tan\theta$$

where

R is the reaction force normal to surface (in N);

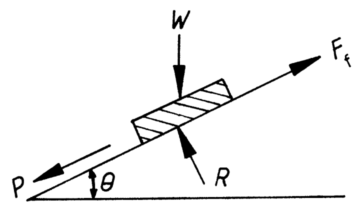
F_f is the limiting value of the frictional force (in N);

W is the vertically applied force (in N);

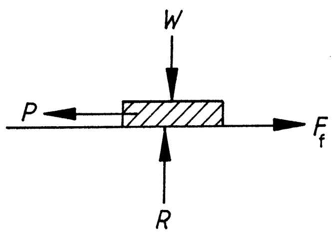
θ is the minimum angle from the horizontal, for a particular pair of materials, at which sliding will commence (in degrees).

It is important to note that the value of the limiting value of the frictional force is that force at which sliding is just about to occur. Thus, when lateral restraint is to be transmitted between the members in contact by friction, a suitable factor of safety to prevent sliding should be used. It is recommended that this should not be less than 2.0.

The value given in Table 19 for the coefficient of static friction, μ, are given for general guidance as a lower bound value and judgement should be exercised in individual cases. Testing may be appropriate.



a)



b)

NOTE P is a force applied to overcome the static friction (in N).

Figure 12 — Lateral restraint provided by friction

Table 19 — Minimum value of coefficient of static friction, μ

Lower load-accepting member	Upper load-bearing member				
	Plain steel	Painted steel	Concrete	Softwood timber	Hardwood
Plain steel	0.15	0.1	0.1	0.2	0.1
Painted or oiled steel	0.1	0.0	0.0	0.2	0.0
Concrete	0.1	0.0	0.4	0.4	0.3
Softwood timber	0.2	0.2	0.4	0.4	0.3
Granular soil	0.3	0.3	0.4	0.3	0.3
Hardwood	0.1	0.0	0.3	0.3	0.1

6.4.5 Overall stability

6.4.5.1 *General*

A falsework structure may overturn as a unit, and safety against this form of instability should be ensured. Such a structure usually relies for its resistance to overturning upon its own self-weight, with or without additional kentledge. In some cases it is possible to tie down to suitable foundations.

The structure should not overturn at any stage during construction when subjected to an overturning moment 1.2 times that calculated. For this calculation only, the working capacity of holding down bolts may be multiplied by 1.2.

Where kentledge is used, care is required to ensure that it is effectively connected to the falsework, and to design the falsework to withstand both the imposed loads and the kentledge loading.

Where falsework is tied to foundations, tension piles or parts of the permanent structure, it should be ensured that the falsework does not cause unacceptable stress in the permanent works.

6.4.5.2 *Stability during erection and dismantling*

The sequence of erection and dismantling should take account of the wind and other forces likely to arise and their consequences (see **6.4.3.2**).

6.4.5.3 *Stability in the working phase*

Working operations applied to or upon the falsework will not usually be possible when wind speeds approach the maximum predicted and as indicated in **6.3.1.3.1** will usually be suspended when the wind speed approaches a design wind speed, V_s, of 18 m/s.

For consideration of the working operations and activity loads, e.g. the placing of fresh concrete, precast units or structural steelwork, the design should take account of the maximum working wind force, W_w, corresponding to a design wind speed, V_s, of 18 m/s.

The falsework should normally be designed to withstand all other combinations of vertical and horizontal loads under maximum exposure wind conditions (but see **6.3.1** and **6.4.3.2**). In all cases the design should provide a factor of 1.2 against overturning.

Account should be taken, under maximum exposure wind conditions, of stored materials on the falsework and portions of the permanent works not yet adequately connected to transfer their loads into the foundations of the permanent works.

Examples of this situation are:

a) steel box girder structure with joints not bolted or welded to transfer the forces acting on it adequately into the permanent supports;

b) precast concrete segmental bridge construction where the structure is not monolithic with abutments or supports until the segments have been stressed and grouted;

c) concrete deck structure with the joints not cast until a later phase of construction, thus allowing decks to “float”.

6.4.5.4 ***Stability during subsequent construction operations***

The stability of the falsework may be affected by additional construction, be it of additional permanent works or additional falsework structures at other levels. The effects of such additional loads may change the loads distributed in the falsework and the method of dismantling should take account of all sources of loading.

A redistribution of vertical loading can result from the post-tensioning of newly constructed permanent work while the falsework is still in position. This may be significant to a span just completed as well as to any connected portions in adjacent spans (see **6.2.1**). Where structures more than 100 m long are to be constructed, the longitudinal strains associated with post-tensioning movements and thermal movements should be examined.

6.5 Foundations to falsework

6.5.1 Purpose of foundations

At the point of contact of the falsework with the ground or with the permanent construction, it is necessary to distribute the loading from the falsework into the ground or works below in a safe manner to confine within acceptable limits any total or differential settlements. The assessment of the site conditions and the determination of safe bearing pressures for use in the design is described in **5.5.2**.

Points of contact between the falsework vertical members and underlying slabs should comprise baseplates attached to the feet of the vertical members, sometimes on distribution members. These distribution members should protect the permanent works from severe point loads (see **6.5.4**).

6.5.2 Falsework supported on permanent works foundation

Where the vertical loads from the falsework are transferred to a ground floor slab a check should be made that this slab can safely receive the loading. In some instances the loads may be collected on to pile caps or foundation bases. In most cases the use of parts of the foundations of the permanent works as a foundation for the falsework should minimize the possibility of settlements at those points. Where part of the falsework is supported on such foundations and the remainder of the falsework distributes its loading directly to the ground, the possibilities of differential settlement and the effect of such loads on the permanent structure should be considered.

Where the loads from falsework are transferred to a permanent foundation, a check should be made that the permanent foundation is at no time eccentrically loaded to an extent that the loading could be detrimental to any support piles or create dangerously uneven ground pressures under the foundation.

In some instances it may not be intended to construct a floor slab at ground level until after the works above have been completed. In such cases, it is preferable to distribute the falsework load on a prepared foundation of hard filling or concrete blinding as required for the permanent construction, and distribute the loads from the falsework safely to these prepared surfaces rather than unprepared ground.

6.5.3 Falsework supported on permanent works above ground level

In multi-storey construction, and in some other instances, the foundations under the falsework will comprise permanent construction. It is necessary to determine any limitations the design of the permanent works imposes on the incidence and distribution of load from the falsework. The strength of the permanent works at the time that they are to be loaded should be established, with particular reference to the stage of construction and the proportion of the ultimate strength that it has gained. This assessment may frequently be based on the expected rate of gain of strength (or maturity) of concrete, and can be checked by testing concrete cubes representing the concrete in the structure in accordance with BS 1881-115 to 120. Shock-loading through the falsework to the structure below should be avoided.

In multi-storey construction it may be necessary to erect falsework to successive floors above, before removing falsework from the lower floors. Advice on propping and repropping procedures for multi-storey buildings is given in Annex M.

6.5.4 Falsework supported on the ground

The loads from the falsework should be applied to the ground through sole plate distribution members, which are most commonly of timber, occasionally of precast concrete, and sometimes of in situ concrete. Where timbers, very often sleepers, or precast concrete are used, care is required to ensure "bedded in" contact with the ground. It is frequently desirable to ensure this contact by the use of in situ concrete. The alignment of these distribution members should be controlled to achieve satisfactory load distribution on the member. Load from a standard may be assumed to spread down through a timber ground distribution member at a slope of 2 horizontal to 1 vertical in a direction along the grain, and 1:1 across the grain and thereafter through the blinding concrete base material at 1:1 both ways (see **3.5.3**). Wherever practical, these members should be continuous under at least three vertical standards of the falsework and the width in contact with the ground should not be less than 250 mm. These ground distribution members will not usually be subject to critical stresses but such a possibility should not be overlooked.

The following precautions should be taken to ensure a satisfactory foundation.

a) Topsoil and weak strata at founding levels should generally be removed. In exceptional circumstances with soils of low bearing capacity it may be desirable to take advantage of a well grassed surface that would provide a better formation.

b) No distribution members should be set or bedded on to frozen ground.

c) No distribution members should be founded over ground that has previously been excavated locally and backfilled without reference to the precautions outlined in Section 5.

d) Edges subject to erosion, e.g. the edges of slopes and terraces, should be protected against eroding forces.

e) Groundwater flows affecting the ground strata or ground surface should be reported to the falsework designer.

f) Any rock outcrops, buried rocks or obstructions that are uncovered and not indicated on the drawings should be compared with the design assumptions as they can result in differential settlements.

g) Where the requirements are such that foundation members need to be set other than level, appropriately shaped packs should be used at the base of the vertical, and the foundation member should be effectively prevented from moving down the slope. (See Figure 13.)

Foundation supports comprising piles or other deep ground insertions should be designed and installed to specific designs and drawings. The settlement characteristics of these measures should be evaluated.

Where there is a likelihood of the foundation area becoming flooded, consideration should be given to directing such flood waters away from the area and to installing foundations that can safely withstand the direct and indirect consequences of such flooding including scour, undermining or weakening of ground strata.

6.5.5 Falsework founded in watercourses

Where supports, which will usually consist of piers or piles, are installed in rivers or other watercourses they should be designed to withstand the horizontal loads arising from flood conditions applied to an area of resistance substantially greater than that offered by the supports alone. This increase should cater for the accumulation of river debris that is likely to occur. To minimize this accumulation and avoid the impact of larger pieces, measures should be specified and installed upstream to divert such debris from the supports or to retain it independently. The measures adopted will depend on the circumstances.

The consequences of impact by heavy floating objects should be taken into account in the design of the protective barriers or the falsework. The use of fenders, floating booms and cutwaters should be considered for this purpose.

Scour is likely to occur in fast running water and is likely to affect the bed of the waterway around and under the falsework and any banks, channels or other existing features of the waterway. Protection should be provided where such scouring forces are likely to occur.

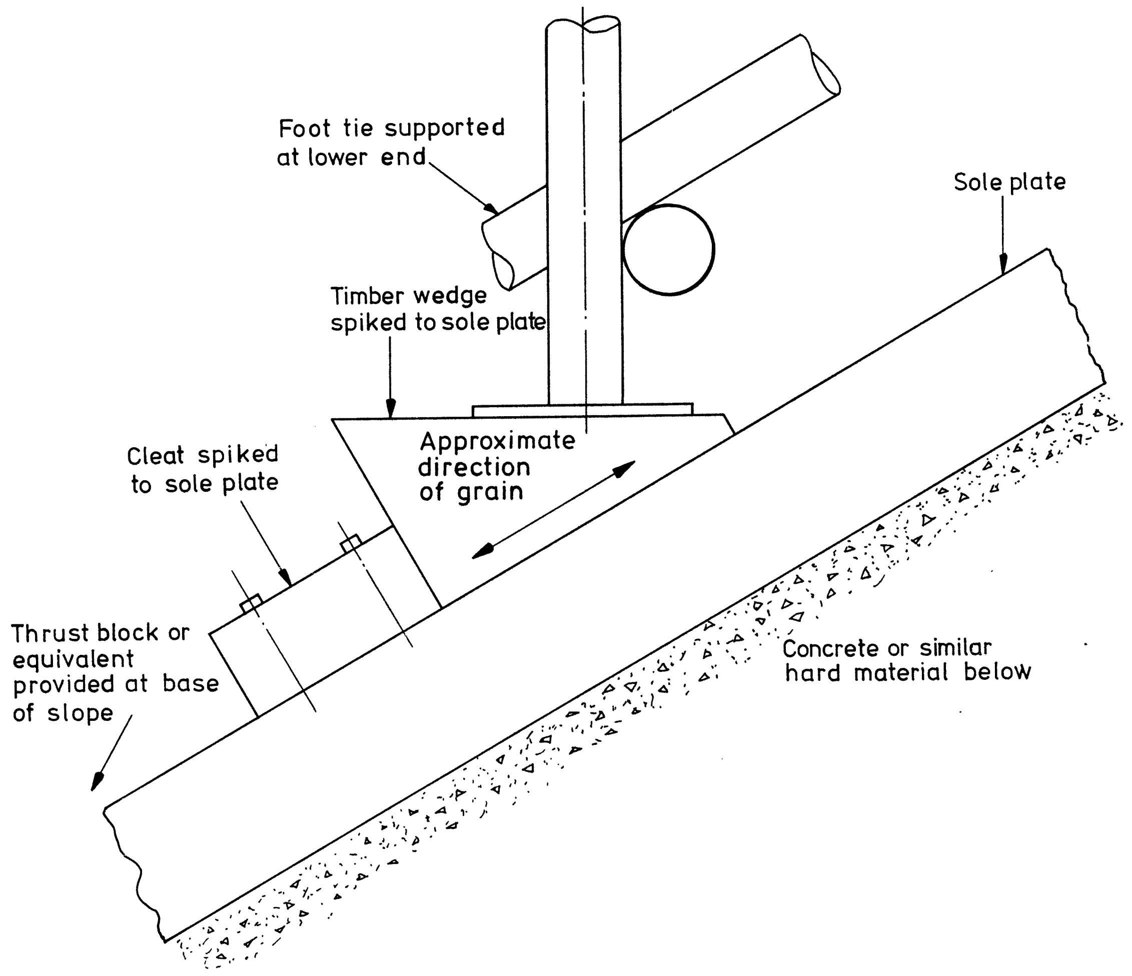


Figure 13 — Base detail on slopes

6.6 Additional considerations affecting certain design solutions

6.6.1 Clear span falsework systems

Where it is necessary to provide clear areas below the permanent structure, a spanning support system will be needed. These areas may be required when crossing site access roads, existing roads, railways, waterways and existing buildings. These spanning units or girders should be taken into account in the design of the permanent works so that they can be supported off the permanent piers or abutments. Alternatively, it will be necessary to erect temporary support towers. This may require the installation of extra foundations.

The beam/girder end supports should cater for horizontal movements arising from elastic deformation, rotation and geometry changes of the beams. The resulting eccentricities of loading on the supports should be defined and allowance made for them.

Where staged construction is carried out on clear span falseworks, the effects on both the falsework and the permanent work should be considered. After the first stage of construction of the span, the subsequent construction stages will induce stresses on the former stages due to the further deflections of the composite structure. Before a design involving composite action of the falsework and the permanent structure is adopted, a careful examination should be carried out to ensure that there will be no unacceptable effects on the permanent structure.

6.6.2 Independent towers in groups

Independent towers usually have sufficient stiffness and internal bracing to ensure stability during erection and reference should be made to manufacturers' data. With many proprietary framed tower systems the bracing is an integral part of the system, thus ensuring that the equipment is braced as it is erected. Once erected, additional lacing and bracing may be required to tie the separate towers into stable groups or to the permanent structure in order to cater for the horizontal components of the imposed loads and increase the stiffness of the assembly. Owing to individual erection and levelling, adjacent towers may not be at the same level and this should be taken into account in making horizontal lacing connections.

6.6.3 Falsework (or centring) for arches

Falsework for arches has traditionally been known as centring, although this term is more usually used when the falsework takes the form of near radial props from a few levels below.

An arch is a particular case of a sloping slab. If the support is a complete birdcage, forces are straightforward to calculate. Care should be taken to ensure that no portion of the partly completed structure can slide down a sloping surface (see **6.4.4.6**). If the arch is of in situ concrete, the pressures from it will also create horizontal forces (see Annex J).

Where the falsework is of arched form, care is needed when considering the varying stresses in the falsework structure as loading proceeds. Loading normally starts at the haunches, and with steep-sided arches this may initially give significant lifting of the crown, owing to the geometry of the arrangement.

Wherever possible, the construction of arches should be balanced and any out-of-balance limited to a maximum out-of-balance, measured along the arch, of 1 m. Resistance should be provided against out-of-balances forces.

The design of falsework to clear span arches can be as 2-pin or 3-pin arches. A gently curved arch with or without a lower tie-rod may be designed as a simply supported beam with a variable distributed load. A check is necessary on the end shear and any tie-rod connection of the tension member into the falsework at the springings to the arch.

Where the centring itself has any members not vertical, it is important that the foundations are designed to cater for the horizontal loads so created. It may be desirable to base the design on balancing the loads from the two sides of the arch, but great care should be taken to ensure that construction follows the design assumptions.

During the erection and dismantling of centring to arches, the stability of the centring should be considered with adequate ties, struts or braces provided at critical stages. Provision should be made to ease the formwork away from the completed arch; this is particularly relevant for arches with steep sides.

6.6.4 Horizontal or raking falsework

In certain circumstances, falsework is used to brace vertical permanent works and the falsework is therefore erected in a horizontal or raking position. An example of this is strutting to vertical diaphragm walls until such time as a permanent slab is in position to restrain the diaphragm walls.

When falsework is used to carry essentially horizontal forces, the design should follow the recommendations of this code but particular care should be taken to account for the self-weight of the falsework in the design. It is recommended that wherever falsework is used horizontally, or raking, an allowance for access loading should be included in the design of the members during the erection and dismantling stages. Care should always be taken in erecting horizontal and raking falsework to ensure that such members can be safely positioned prior to being loaded, and also that they can be de-stressed after use when the loads are transferred to the permanent works.

6.6.5 Mobile falseworks

Whenever falsework structures are moved from one use to another without dismantling, the design of the individual items of the structure should be as recommended in this code. Care should be taken to allow for any additional forces that could be applied to the structure during the movement. These forces may arise from uneven movement, differential settlement and misalignment of trackways, etc. The magnitude of such forces is not covered by this code.

6.7 Design using scaffold tube and fittings

6.7.1 General

Information on the equipment is given in **3.8**, and on erection tolerances in **7.3.2**. The use of tube and fittings is also described in BS 5973.

Falsework erected with tube and fittings will normally comprise standards, horizontal lacing tubes and diagonal bracing tubes. Generally, the vertical load-bearing tubes should be connected by end-to-end couplers, preferably sleeve couplers. All other connections should be made with couplers giving maximum eccentricities from member to member of not more than 55 mm. At node points, several tubes may be intersecting at the same theoretical point (see **6.4.4.3**). Generally, provided that the distance between the centrelines of the load paths does not exceed 150 mm, the effects of torsion and eccentricity may be ignored.

However, if couplers are being used to transfer loads of greater than 6.3 kN, the effects of eccentricity may have to be allowed for (see Figure 14).

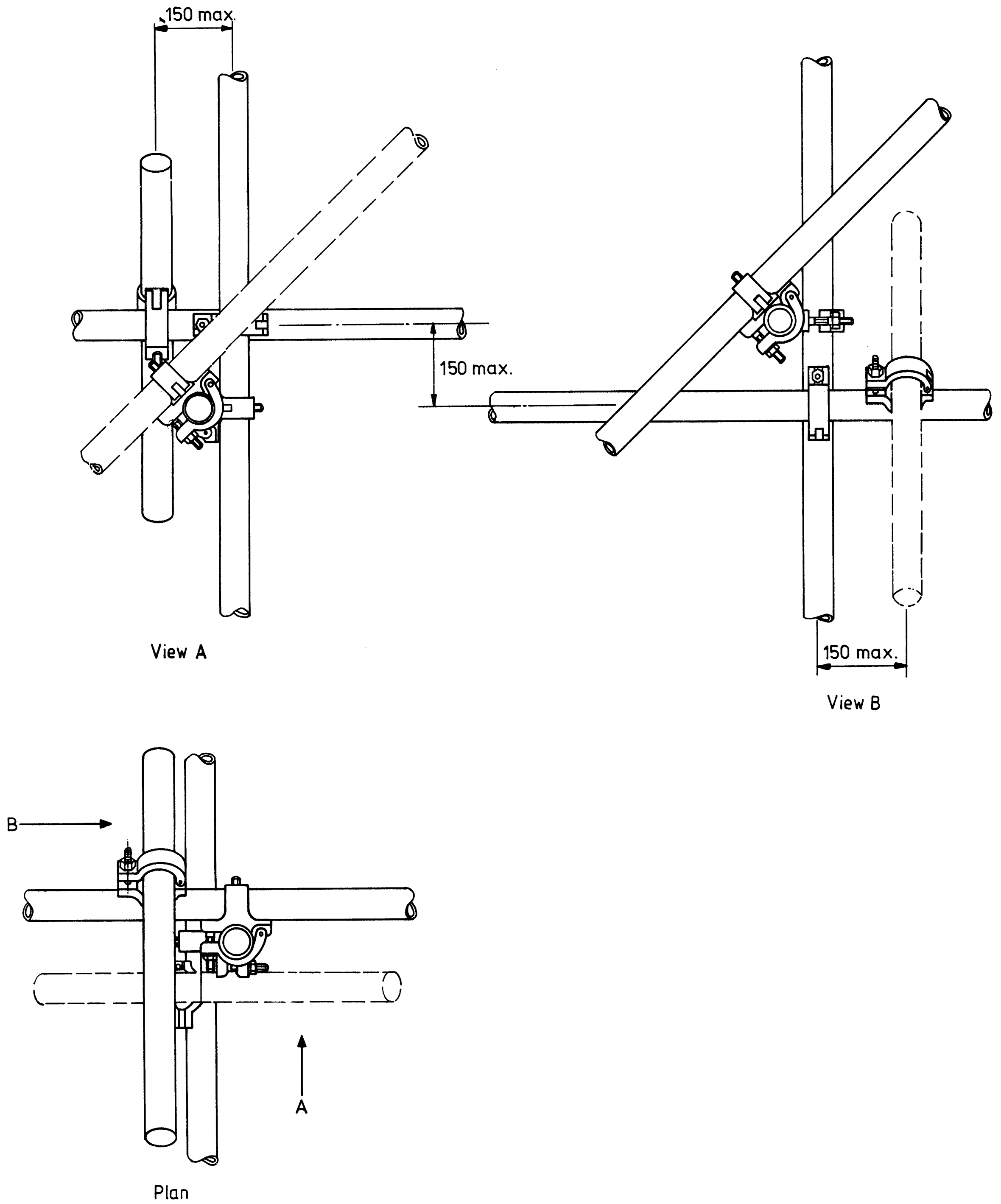
Where a loaded horizontal tube is connected to and supported from two vertical tubes alongside and there are no others, the full eccentricity is developed, and the design should be carried out accordingly (see tube A in Figure 15). However, if two horizontal tubes B at right angles to tube A, also connected to other standards, are used to counteract this eccentric loading, it will be unnecessary to assess the eccentric loading so created. To prevent eccentricity elsewhere, further tubes such as C should be added.

Annex B recommends safe working loads for right angle couplers when correctly erected and tightened. When it is intended to increase the safe working capacity of a joint by the use of supplementary couplers to back up the main coupler, the information regarding the strength of the proposed arrangement should be obtained from the manufacturer or supplier. The supplementary coupler may be the other half of a right angle coupler, or a single coupler may be used. Attention is drawn to the fact that when using supplementary couplers the mode of failure may be changed.

BS 1139 has no requirement for rotational slip so a coupler may slip around the tube to which it is attached at relatively low loads. Consequently, loads should not be applied in this manner.

A point of articulation exists where a tube and an adjustable base or headjack meet. Careful consideration should be given to the bracing, to prevent any movement, particularly where there is a large extension of the adjustment (see **7.3.2.3**).

Dimensions in millimetres.



NOTE One diagonal is shown dotted for clarity.

Figure 14 — Maximum deviation of load path

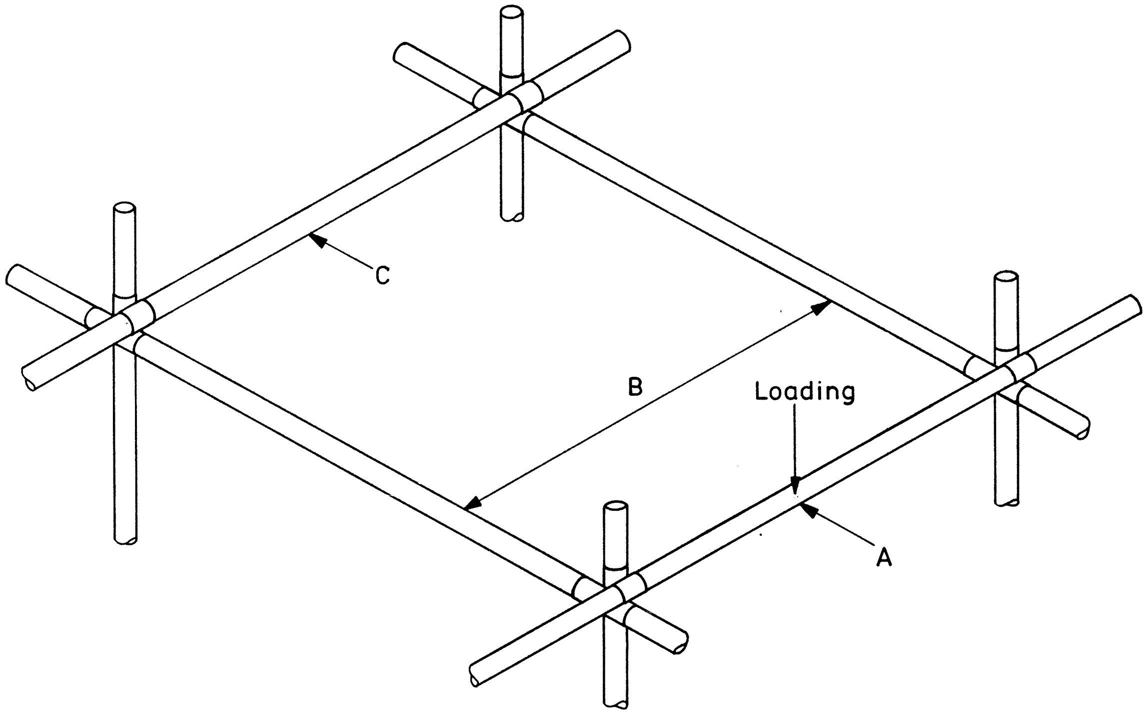


Figure 15 — Eccentricity of loading

6.7.2 Effective lengths of scaffold tube struts

In general, the degree of joint restraint exhibited at a node point at which cross-connections are by means of scaffold couplers can be assumed as being negligible. This, together with the lack of restraint against rotation provided by such couplers means that when designing structures using scaffold tube and fittings, effective lengths of less than $1.0L$ should not normally be used (see L_3 in Figure 16). Effective joint restraint is only likely to exist in riveted or bolted structural connections, or with welded joints combined with reasonable continuity of members. However, if it can be shown that a particular type of coupler provides joint restraint, this may be taken into account when calculating the effective lengths.

Where a structure includes a strut with a free cantilever projection (m_1L_1 in Figure 16), this cantilever plus the portion immediately adjacent to it (L_1 in Figure 16) should be considered as having one effective length.

The effective length, l, of such a strut should be determined by the expression:

$$l = L + 2mL$$

where

L is the length between supports;

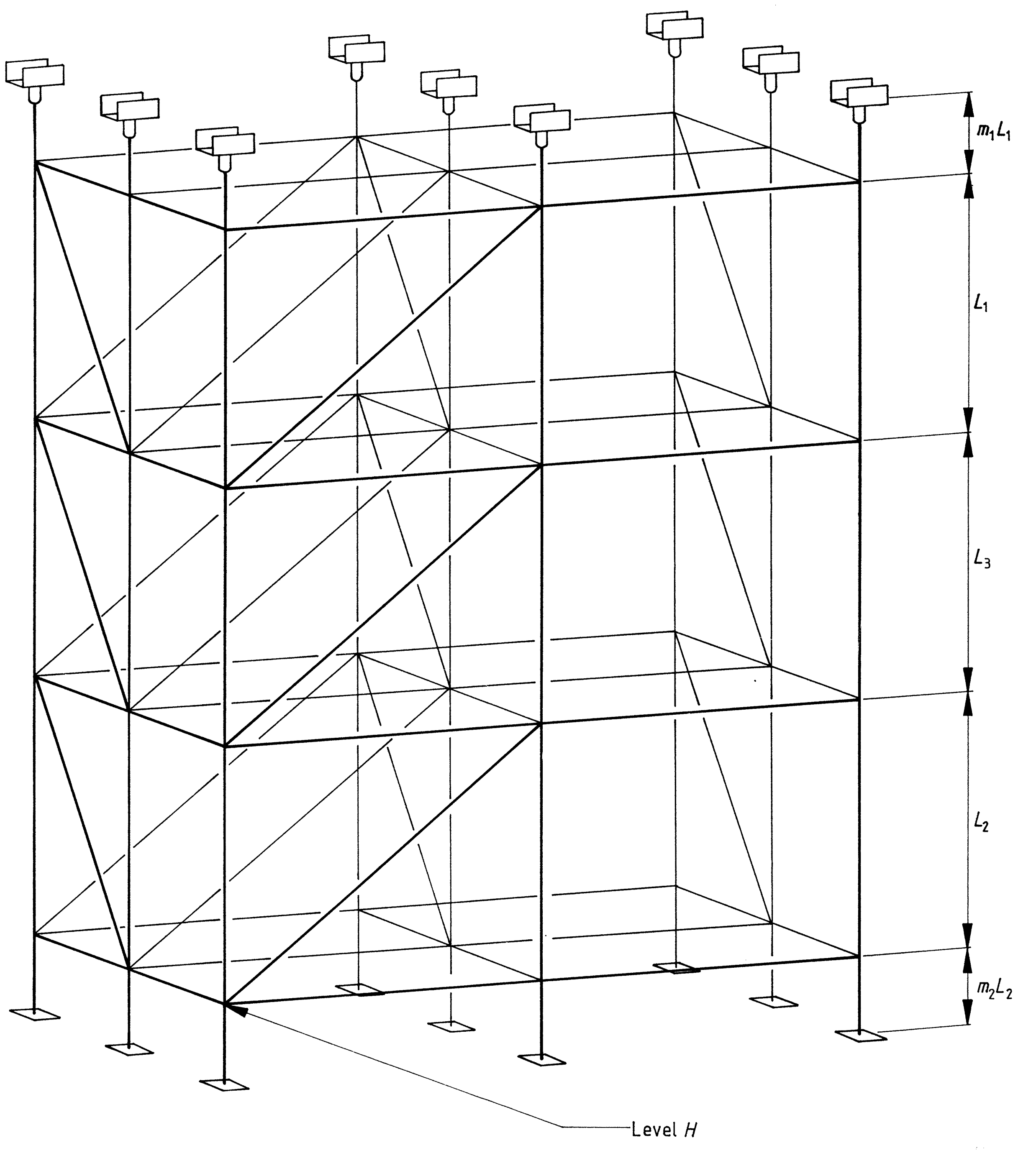
mL is the length of the cantilever projection.

In a free-standing structure the lower portion (m_2L_2 in Figure 16) should also be considered as a free cantilever and should be designed in accordance with the above expression.

Where both ends of a projecting member are held in position relative to one another, the member is not a free cantilever and should be treated as a normal strut with an effective length of $1.0L$. For example, such a condition can be achieved for the base projection, either by a system of external ties preventing movement relative to the ground, preferably at level H (see Figure 16) or by the continuation of the diagonal bracing system to the ground. It can normally be assumed that friction between a baseplate and the ground is sufficient to prevent movement of that end of the projection.

Figure 16 indicates how these effective lengths should normally be applied to a structure in steel scaffold tube. It assumes axial loading on the columns and does not take into account any horizontal loadings. In falsework and similar structures, e.g. access birdcages, the unrestrained members may be subjected to horizontal as well as vertical loads and this should be taken into account in the design. Annex L provides further information on the general philosophy of effective lengths.

NOTE Configuration of scaffold is diagrammatic only.



NOTE Configuration of scaffold is diagrammatic only.

Figure 16 — Effective lengths in tube and coupler scaffolding

Section 7. Work on site

7.1 Introduction

This section gives guidance for work on site. When work is of a traditional nature it may come within the scope and limitations of Section 8. In all other major and non-standard situations the recommendations of this section should be followed.

7.2 Specific design instructions

As previously recommended, as part of the design process, drawings, sketches or similar details should be prepared in a form that can facilitate their use as directions for work on site. They should define such points as requirements for foundations, positions of components, the nature of connections to other components, and limitations for loading and sequence of operations. These written instructions should be followed, but those responsible for work on site should bear in mind the need to compare conditions experienced on site during construction with those assumed in the proposed design in order that appropriate action may be taken to modify the design if it becomes apparent that this may be advisable.

7.3 General workmanship

7.3.1 Critical factors of workmanship

The following main factors concern work on site.

a) The foundations should be satisfactory.

b) The falsework should be in accordance with the design, in particular as regards quality and quantity of components, and setting out.

c) Tolerances should be in accordance with **7.3.2**.

d) All connections should be properly constructed.

e) There should be adequate safe access and working places.

7.3.2 Accuracy of falsework

7.3.2.1 ***General***

Unless otherwise specified, the limiting criteria recommended in **7.3.2.2**, **7.3.2.3** and **7.3.2.4** should not be exceeded on site.

7.3.2.2 ***Adjustable steel props and forkheads***

The following limiting factors are appropriate to adjustable steel props.

a) Props should be undamaged and not visibly bent.

b) Props should be plumb within 1.5° of vertical (i.e. not exceeding 25 mm out-of-vertical over a height of 1 m).

c) Props should be placed centrally under the member to be supported and over any member supporting the prop, with no eccentricity in excess of 25 mm.

d) Props should be located in accordance with **7.3.4**.

7.3.2.3 ***Tube and coupler falsework***

In the case of tube and coupler falsework the following factors should apply.

a) The tubes used in falsework should be undamaged, not visibly bent or creased and have smooth square cut ends. Other components should also be undamaged.

b) Verticals should be plumb within 15 mm over 2 m of height, subject to a maximum displacement from the vertical of 25 mm.

c) Vertical members should be placed centrally under the members to be supported and over the member supporting them with no eccentricity exceeding 25 mm.

d) Adjustable forkheads and baseplates should be adequately laced or braced where their extension exceeds 300 mm, unless an alternative figure is specified. The bracing tubes should be attached close to the fork or baseplate and to an adjacent vertical member, close to the lacing.

e) Tubes should have end-to-end joints in adjacent tubes staggered. Sleeve couplers should be used in preference to joint pins for axial connections.

f) The centrelines of tubes at a node point should be as close together as possible, and never more than 150 mm apart (see Figure 14).

g) Sole plates used to distribute falsework loads on to foundation soils should normally be set horizontally within a tolerance not exceeding 25 mm in a length of 1 m (but see **6.5.4**).

h) The tubes and couplers should be located in accordance with **7.3.4**.

7.3.2.4 *Purposely fabricated steelwork*

The following tolerances should be adopted for purposely fabricated steelwork when designed using the permissible stresses given in Annex A.

a) *Inclination of a column from vertical (see Figure 17a)*

1) for columns of length $L_s < 1\,450$ mm, Δ_v should not exceed 5 mm;

2) for columns of length $L_s \geqslant 1\,450$ mm, Δ_v should not exceed 0.0035 L_s or 25 mm, whichever is the lesser;

where

L_s is the clear length of the strut or column (in mm);

Δ_v is the inclination from vertical (in mm).

b) *Out-of-straightness of a strut or column (see Figure 17b))*

1) for a column or strut of length $L_s < 3\,350$ mm, Δ_s should not exceed 5 mm;

2) for a column or strut of length $L_s \geqslant 3\,350$ mm, Δ_s should not exceed 0.0015 L_s or 25 mm, whichever is the lesser;

where

L_s is the clear length of the strut or column (in mm);

Δ_s is the out-of-straightness of the column or strut (in mm).

c) *Out-of-straightness of a beam (see Figure 17c))*

1) for a beam of length $L_b < 3\,350$ mm, Δ_b should not exceed 5 mm;

2) for a beam of length $L_b \geqslant 3\,350$ mm, Δ_b should not exceed 0.0015 L_b or 40 mm, whichever is the lesser;

where

L_b is the clear length of the beam (in mm);

Δ_b is the out-of-straightness of the beam (in mm).

d) *Eccentricity of a beam bearing (see Figure 17d))*

The eccentricity of any beam, e_o, should not exceed 5 mm.

7.3.2.5 *Other types of construction*

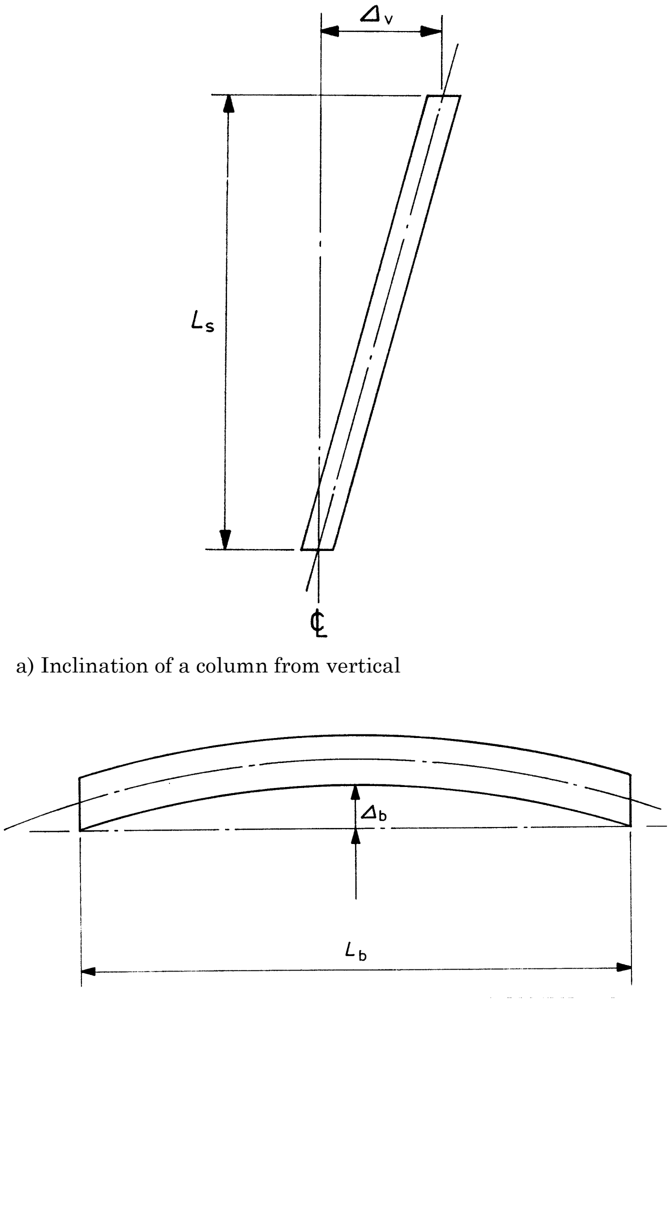
For special designs, manufactured components or other types of construction, the appropriate tolerance requirements should be ascertained from the manufacturer or designer.

7.3.3 Vertical movement

It is particularly important that all the feet of one tower are supported on similar foundations, to prevent any tendency to settle sideways. The details at the foot of each leg should be so arranged as to spread the load sufficiently.

There will always be some take-up in the joints of a falsework structure that will vary with the type of joint and the type of materials used; allowance should be made for these movements (see **6.4.2**).

Dimension in millimetres.



a) Inclination of a column from vertical

b) Out-of-straightness of a vertical strut or column

c) Out-of-straightness of a beam

d) Eccentricity of beam bearing

Figure 17 — Points of measurement of tolerances for purposely fabricated steelwork

7.3.4 Top arrangements

Forkheads, where used, should be rotated to centralize the bearer they support. Where beams terminate in a forkhead they should extend past the centre point of the forkhead by at least 50 mm. Alternatively, where timbers butt in a forkhead, the joint should be within 15 mm of the centre of the forkhead.

Soffit formwork should be nailed to the upper timber. The timbers should themselves be nailed at the intersections, and to the top plate or forkhead.

7.3.5 Wedging

Load-bearing wedges should be selected to be of uniform sound quality, used in matched pairs and secured by nailing. Single wedges should not be used for vertical adjustment except in conjunction with an inclined surface. For folding wedges used for vertical adjustment, the slope of the inclined face should not exceed 1 in 5 and they should be as wide as the member above to develop full bearing of the members in contact. They should not be used with their contact area reduced by more than half the area of the face. Wedging should be applied to avoid eccentricity of loading on the member being wedged, or distortion of formwork. The material (see **3.4.1.3**) from which the wedges are formed should be appropriate for the stresses to which they will be subjected.

7.3.6 Lacing and bracing

Lacing and bracing is specified to provide essential restraint in a horizontal direction and should therefore be located where indicated by the design and secured to a firm support or it could prove to be ineffective (see **6.4.4**).

When erecting props, tubular lacing should normally be used to connect the upper parts. This lacing should remain until after concreting and be so arranged as to connect all the props above the jacking device, at a position one-third of the distance up the prop extension.

7.3.7 Importance of details

Constant emphasis is laid on the importance of careful attention to details. Falsework usually consists of a comparatively large number of members to produce a supporting structure or arrangement with a multitude of connections and junction conditions. The integrity and stability of the whole should not be jeopardized by misassembly of one of the many connections or junctions.

The omission of a bolt or a wedge, or the failure to tighten up an item properly, could lead to local instability that might endanger the structure. It is therefore important that sufficient care and attention is paid to the initial erection.

When equipment such as a proprietary framing system is used, all the recommended components should be used; no changes should be made without further consideration of the original design.

7.4 Checking falsework

7.4.1 When to check

There are a number of clearly defined stages in the construction of certain types of falsework when formal checks are desirable. They may not be necessary for simpler arrangements, such as those described in Section 8 of this code when a formal check immediately prior to concreting may be all that is required.

It is recommended that the need for a formal check should be considered:

a) when the proposed founding level for the falsework is in preparation;

b) when the falsework has attained a height of 10 m or a height equal to 1.5 times the minimum of its plan dimensions;

c) when the falsework reaches its support level;

d) at intermediate stages, when the strength or stability of the falsework may have been adversely affected by environmental or other loading conditions or unauthorized interference;

e) where equipment is being continually reused and periodic checks are appropriate;

f) immediately prior to loads being applied.

Should the structure be subjected to load redistribution due to post-tensioning, a constant observation should be maintained for any movements or sounds of distress in the supporting falsework. It may in such instances be necessary to carry out a check before stressing continues.

Critical factors of workmanship to which attention should be given are listed in **7.3.1** and **7.4.2**.

7.4.2 Items to be checked

At the stages indicated in **7.4.1**, thorough inspection of the falsework is necessary to ensure that the completed structure will function as intended.

Whilst the following list is not exhaustive, it will give guidance on what to look for in a systematic manner.

The inspections should be undertaken with direct reference to any drawing or specification that has been issued, and checks subsequent to the first should inspect every feature that could have altered in the intervening period.

THE RESULT OF CHECKS OR INSPECTIONS SHOULD BE RECORDED IN WRITING AND ACTION TAKEN TO CORRECT ANY FAULTS.

It should be checked that:

a) *general*:

1) all the drawings and written instructions have been strictly complied with;

2) only the correct materials in serviceable condition have been employed, especially if specific types or qualities were required as will normally be the case with structural steel or timber;

b) *at founding level*:

1) the setting out is correct;

2) the ground has been adequately prepared and is at a satisfactory level (foundations appearing sound in dry or freezing conditions can be quite inadequate following rain or thaw);

3) suitable sole plates or other bases have been provided and have been properly levelled;

4) sole plates or other bases have not settled;

5) sole plates have been properly bedded down (no cavities underneath), and steps taken to prevent erosion;

6) sole plates and other load-distributing members laid on the slope are adequately prevented from movement down the slope;

7) any chocks or other supports are the correct shape, and are adequately secured;

8) baseplates have been used and are properly spaced and centred on the sole plates;

9) the extension of each screw or adjustable base is within the permitted limits, and braced if necessary;

c) *above founding level:*

1) ties and/or rakers have been fitted, linking all uprights in two directions roughly normal to each other, or at a specified skew angle;

2) upright members are plumb (to do this, a few upright members should be checked with suitable instruments and marked; the remainder can be checked by eye) (see **7.3.2** and **8.4.2** for tolerances);

3) joints in vertical members are properly butted and aligned, and reinforced if required;

4) the spacing and level of each lift of lacing members are correct;

5) the number and position of all bracing members (longitudinal, lateral and plan) are correct with connections close to node points;

6) the restraints are effective where falsework is stabilized by butting, wedging or tying of lacing members, instead of bracing;

7) forkheads are properly aligned, and any extension is within the permitted limits and braced where necessary;

8) bearers are correctly spliced, centralized in forkheads, and if required, wedged and nailed in the fork;

9) beams, including floor centres, have adequate settings and are secured against movement;

10) any necessary web stiffeners and lateral restraint have been provided;

11) all pins, bolts, clips and the like, have been fitted, are of the correct type and are secure;

12) scaffold couplers are properly tightened (it will usually be sufficient to physically check some at random, including the less accessible regions);

13) where access is required by workmen, ladders, platforms, guardrails and toeboards are fixed and comply with the requirements of the Construction Regulations (see **1.5** and BS 5973).

7.5 Application of loads to falsework

During erection of the falsework, the lower work is subjected to a steady increase of dead load plus live load effects from the erection process together with wind and impact forces. It is when the structure or component becomes of appreciable height by comparison with its plan dimensions that wind loading and other lateral loads become of consequence to the design. When substantial loads can be anticipated, provision should be made to ensure safety and stability, e.g. the installation of bracing. The installation of such provision at the right time should be assured.

The next critical stage of construction is likely to be reached when the installation of formwork commences. This will usually entail the loading of formwork, closely followed by reinforcement on to the falsework. The falsework will then be subjected to the additional loads of the labour force and stacked materials, and to wind forces, particularly at the upper level. The combination of loads that these circumstances entail may need to be controlled to avoid both unreasonable risks and unreasonable additional bracing costs.

By the time the falsework is ready to receive concrete or units, changes may be envisaged for the method of concreting or application of other loads. The effect of such changes should always be considered in relation to the original method proposed.

Control of the sequence and rate of placing of in situ concrete is necessary so that pressures are not allowed to build up undesirably. Whilst it is generally desirable to load the falsework system as uniformly as possible, it is equally desirable to limit the area of concrete face to which fresh concrete is to be added in order to reduce the formation of 'cold joints' on which poor adhesion occurs. Thus the formulation of a concreting method has to recognize several sometimes conflicting requirements and so dictate the rate and method of placing.

The method of raising the concrete on to the formwork, its distribution and placing can impose impact or surge effects on formwork and falsework and such forces should be avoided or minimized. It is important that wedges and struts are properly nailed or otherwise restrained so that they do not work loose owing to impact or vibration. When concreting under inclined formwork, uplift forces can be minimized by careful compaction, thus aiding an accurate cast. Any uplift forces should be absorbed within the formwork. The concrete discharged on to the formwork should not be allowed to accumulate to cause local overloading (see **4.4.3.1**). Measures may be necessary to reduce cracking of the concrete and these should be taken as soon as possible.

Where precast concrete, structural steel, or other such components are applied as load to the falsework, care should be taken to minimize impact forces in both the vertical and horizontal directions. Dragging of sections into final position should be avoided unless specific provisions have been incorporated into the design.

Attention should be paid to any restrictions on the loading of the falsework so as to avoid concreting or placing other loads when high winds, heavy rains, snow, or swollen rivers occur.

Consideration should be given to the possibility of loads being built up by rainwater, snow or concrete spillage on to the permanent works immediately after their construction. It would be an unusual circumstance that would make such an oversight critical in itself, but in combination with other loading conditions it could be of serious import.

7.6 Dismantling

7.6.1 General

The process of dismantling tall falsework should be undertaken with care so that tall unbraced sections are not formed that are susceptible to wind or other unstabilizing forces (see also **6.4.4.4**).

When a falsework system is dismantled, the individual components should be examined for damage and damaged pieces removed for attention.

7.6.2 Supports required after general dismantling

During the increase in strength of newly cast permanent works above the falsework it may be possible at an early date to authorize the safe stripping and removal of falsework and formwork to the general area of the work. It may be necessary to retain support to specific points of the permanent works for a longer period until the concrete is stronger (see Annex M), post-tensioning has occurred, or subsequent construction has taken place. The arrangements for dismantling the falsework should preferably allow for these specific supports to be retained as adequately braced towers while the general falsework is removed from around them. It is essential that such members to be retained should be clearly indicated to the dismantling team.

7.7 Maintenance, inspection and identification of materials

The equipment and materials used in falsework should always be examined when they are dismantled before being used again. When they arrive fresh on to a site and it is not known that they have been properly inspected prior to despatch, they should be inspected before use. Such inspections should only be undertaken by competent personnel.

When equipment and materials have been dismantled and any damaged or suspect items set aside, the remainder should be collected together. Before they are used again they should be cleaned of deposits of soil, concrete, or other such unwanted materials. Screwed members should have threaded portions oiled after cleaning. Loose fittings should be collected together in boxes or bags between uses to facilitate their storage and handling.

Not only can their loss prevent the use of the main equipment of which they form a part but may increase the risk of dangerous improvization.

Components should be properly stacked on a suitable surface to enable easy movement and reduce the dirtying of the materials. Tubes and long members should be laid horizontally and clear of the ground. Similar types should be kept together. Visual identification of length will be facilitated by stacking all components with one end on a datum mark.

Structural steel sections should be identified by having their grade and structural thickness marked on them.

Where steel or stress graded timber is cut to length, the identification marking should be repeated on all pieces.

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Section 8. Standard solutions

8.1 Introduction

This section is concerned solely with "standard solutions" for the simpler type of falsework structures, and describes arrangements for the supporting structure for three thicknesses of slabs and three ranges of size of beams. The solutions described cover the use of telescopic props and standard scaffold tube with timber and plywood soffit formwork to provide support to newly cast in situ concrete. When utilizing these designs it is important that all the various limitations described in **8.4** are observed.

If the limited number of "standard solutions" described in this section do not suit the requirements of the work to be carried out, and if no other suitable "standard solutions" are available from an alternative source (see **6.2.2.3**), the structure should be the subject of a specific design prepared in accordance with the recommendations of this code.

8.2 Procedure for the use of standard designs

It is strongly recommended that the required arrangement of vertical supports, lacings and bracings, together with the supporting foundation preparation and method for erection of the formwork above is drawn or sketched out so that all concerned know exactly what is to be erected.

Thorough inspections should be carried out prior to loading, and in accordance with **7.4.2**.

8.3 Criteria assumed in preparing the standard solutions

8.3.1 Loadings

In the Table 20 and Table 21, allowance has been made for:

a) a density of concrete of 2 500 kg/m^3;

b) a self-weight of 50 kg/m^2 for formwork material shown in Figure 19 and Figure 20;

c) the additional effects of limited heaping of concrete and impact, together with those of the construction labour and plant involved in spreading, levelling and compacting concrete, taken as 150 kg/m^2 (see also **8.4.7**).

8.3.2 Factor of safety

Providing that the falsework components are used as described in this section, the falsework structure will have a minimum factor of safety against collapse of 2.0.

8.3.3 Timber quality

For the purposes of this section, stresses appropriate to class SC3 have been used. The timber is assumed to have been planed all round in accordance with the reductions and tolerances in BS 4471 and to be without wane.

8.3.4 Continuity

Allowance has been made for joints in the timber support members to be made at any support point. The support reactions below the primary members have been increased by 10 % to allow for the effects of continuity (see **6.4.3.1**).

8.3.5 Plywood

Calculations are based on the use of standard sanded Douglas fir exterior plywood, 7-ply 19 mm thick, using values from BS 5268-2:1991, Table 44. The spans given are the maximum permissible that allow uniform spacing behind a 2 440 mm long plywood panel with the face grain parallel to the span and with a maximum deflection of 1/270 of that span.

Other plywoods are similar, but if there are particular requirements for flatness of the concrete surfaces a check should be made.

8.4 Limitations

8.4.1 Foundations

The foundation should be level and comprise one of the following:

a) the permanent works foundations or a ground slab;

b) a slab supported by continuous propping down to a base as described in item a);

c) a suspended slab not supported continuously from ground level but for which the application of falsework loads has been approved;

d) ground or fill capable of sustaining a pressure of 100 kN/m^2.

Arrangements other than those given above should be the subject of specific design.

8.4.2 Support equipment

8.4.2.1 *Adjustable steel props, and forkheads*

The props to be used should comply with the following.

a) The props should comply with the requirements of BS 4074.

b) The props should be undamaged and not visibly bent.

c) The props should have the high tensile steel pins provided by the manufacturer and only these should be used.

d) The props should be placed centrally under the member to be supported and over the member supporting them, with no eccentricity in excess of 25 mm.

e) The props should be plumb within 1.5° of vertical, checked by the use of a suitable gauge (i.e. not exceeding 25 mm out-of-vertical over a height of 1 m).

f) The props should have the top and base effectively located and stabilized (see **8.4.5** and **8.4.6**).

g) The props should incorporate suitable means for ensuring that, where employed, forkheads can be centralized.

The forkheads used should be at least 100 mm long (measured along the axis of the timber).

8.4.2.2 *Tube and coupler falsework*

When using tube and coupler falsework for standard solutions the falsework should not exceed 6.0 m in height and the tubes used should comply with the following.

a) The tubes and couplers employed should comply with the requirements of BS 1139. Although the requirements for tube specified in BS 1139-1.1:1990 is marginally higher than specified in BS 1139-1:1982, and because scaffold tube has a long life, the advice given for "standard solutions" in this code is applicable to both types of tube in an assumed "used" condition.

b) The tubes should be undamaged, not visibly bent or creased and have smooth square-cut ends.

c) The tubes should, when vertical, be placed centrally under the member to be supported and over the member supporting them with no eccentricity exceeding 25 mm.

d) The tubes should, when vertical, be plumb within 15 mm over a height of 2 m and checked by means of a suitable gauge. In addition, the total horizontal displacement should not exceed 25 mm.

e) The tubes should, when vertical, be provided with plain or adjustable baseplates and forkheads or capping plates, laced and braced where their extension exceeds 300 mm. The bracing tubes should be attached close to the fork or baseplate and to an adjacent vertical member, close to the lacing.

f) The tubes should be laced at head and foot and intermediate levels so that the vertical distances between levels of lacing do not exceed 2 m.

g) The tubes should be connected by right angle couplers, except for diagonal bracing, where swivel couplers may be used.

h) The tubes should have end-to-end joints in adjacent tubes staggered and made with sleeve couplers.

i) The tubes should be effectively located and stabilized (see **8.4.5** and **8.4.6**).

j) The tubes should be connected so that the centrelines of the tubes at node points are as close together as possible, and never more than 150 mm apart (see Figure 14). The forkheads used should be at least 100 mm long, measured along the axis of the timber.

8.4.3 Timber

Timber should comply with the requirements of strength class SC3 and may be sawn or planed.

8.4.4 Plywood

Plywood should be exterior grade quality Douglas fir or equivalent with appropriate strength characteristics to suit the spans given in **8.5** (see also **8.3.5**).

NOTE For Douglas fir plywood, a minimum thickness of 19 mm should be used.

8.4.5 Construction

8.4.5.1 ***Base arrangements***

Sole plates, used on compacted ground, if of timber, should have a minimum cross-section of 250 mm × 125 mm. No upright should be within 300 mm of the end of the sole plate.

8.4.5.2 ***Lacing of props***

When erecting props, tubular lacing should be used to connect the upper parts. This lacing should remain until after connecting and be so arranged to connect all the props above the jacking device, at a position one-third of the distance up the prop extension. This use of lacing always applies where the extended prop length exceeds 2.75 m.

8.4.5.3 ***Top arrangements***

Forkheads, where used, should be rotated to centralize the bearer they support, and any wedge used should be secured by nailing.

Where beams terminate in a forkhead, either:

a) they should extend past the centre joint of the forkhead by at least 50 mm; or

b) where timbers butt in a forkhead, the joint should be within 15 mm of the centre of the forkhead.

Where upper timbers butt on a primary timber, both ends should be within 15 mm of the centre line.

Soffit formwork should be nailed to the upper timber and the timbers should be nailed at the intersections and to the top plate or forkhead.

8.4.6 Stability

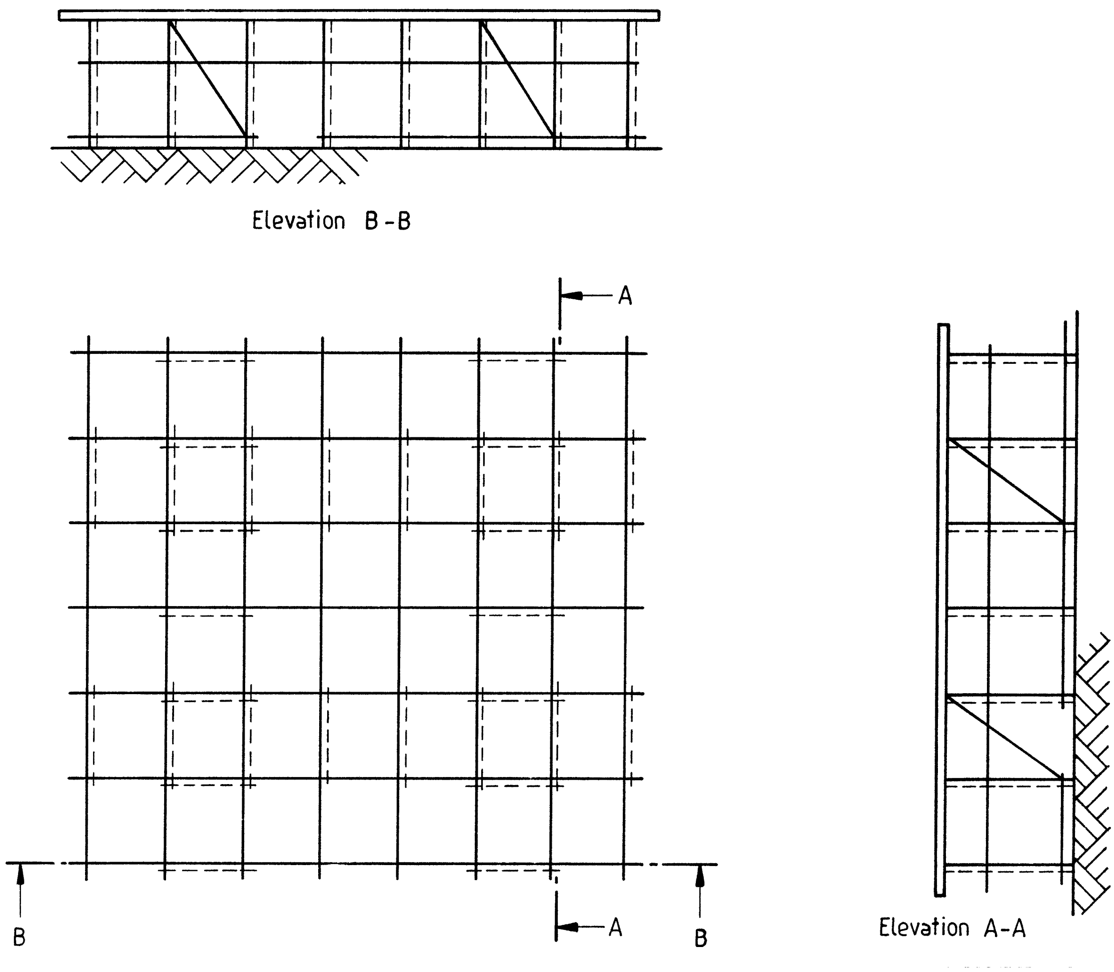
8.4.6.1 ***General***

To withstand windloading, to control any inherent tendency of the structure to move or fall over, and to ensure that each level of lacing is effective, one or more of the methods of providing stability described in **8.4.6.2**, **8.4.6.3** and **8.4.6.4** should be adopted.

8.4.6.2 ***Stability by bracing***

Bracing may be used as a means of obtaining stability in falsework. When bracing falsework utilizing props as the vertical load-bearing members, at least every sixth prop in a row, a vertically orientated diagonal brace should be fixed in the direction of the row. Similar bracing should be provided at right angles to the first direction of bracing (see Figure 18 and **8.4.6.4**). These braces should be fixed to lacing at the base of the prop as near to the prop as possible and, at the other end, to the next prop as near to its top as possible. The angle of inclination of a brace should be not greater than two vertical to one horizontal. If a brace connected between two adjacent props would otherwise exceed this limit, it should be connected to the next but one prop. The lacing at the base of the prop should consist of a horizontal tube fixed as close to the base of the prop as possible, and should connect adjacent rows of props.

In the case of tube and coupler falsework, diagonal braces should be provided at a minimum frequency of one brace every sixth standard, in each line of standards. These braces should be attached as near to the tops and bottoms of the standards as is possible, and should be so inserted that they cross, and are attached to, every level of lacing. The angle of inclination of the bracing to the horizontal should be not more than two vertical to one horizontal.



NOTE A foot tie may be omitted from a line of bays between each pair of braces to facilitate access.

Figure 18 — Basic arrangement of bracing to provide stability

8.4.6.3 ***Stability from existing structure***

Stability may be obtained by connection to an adjacent stable structure such as an adequately strong portion of the permanent works. Columns are frequently useful. Connections may be made by packing or blocking at both ends, or providing a connection with tension and compression capability. No vertical member should be more than four members away from such a strong point unless otherwise stabilized.

8.4.6.4 ***Stability by mixed methods***

Stability may also be ensured by means of a combination of the methods described in **8.4.6.2** and **8.4.6.3**. Where the falsework for a beam consists of up to five pairs of vertical supports and in addition is effectively stabilized against such features as walls or columns, provision of further longitudinal stability (i.e. along the length of the beam) is not necessary. The provision of lateral stability is still a requirement and if it is proposed that this be by means of diagonal bracing this should be provided for, at a minimum, each alternate pair of vertical supports. Where the ends of the falsework are not supported by external means, in addition to the provision of the necessary longitudinal stability, each end pair of supports should be braced diagonally.

8.4.7 Concrete placing

Placing of concrete may be by means of a tower crane and skip, by direct discharge from a mixer truck, by pumping or by barrows. In no case should the free fall of the concrete exceed 500 mm.

Where the concrete is pumped into position, on no account should the pump or the pipeline be connected to or laid upon the falsework or formwork so that loads or surge forces are transmitted to the falsework.

The designs in **8.5** do not allow for the use of mechanized vehicles on the falsework.

8.4.8 Tolerances

All tolerances and workmanship should be in accordance with **7.3**.

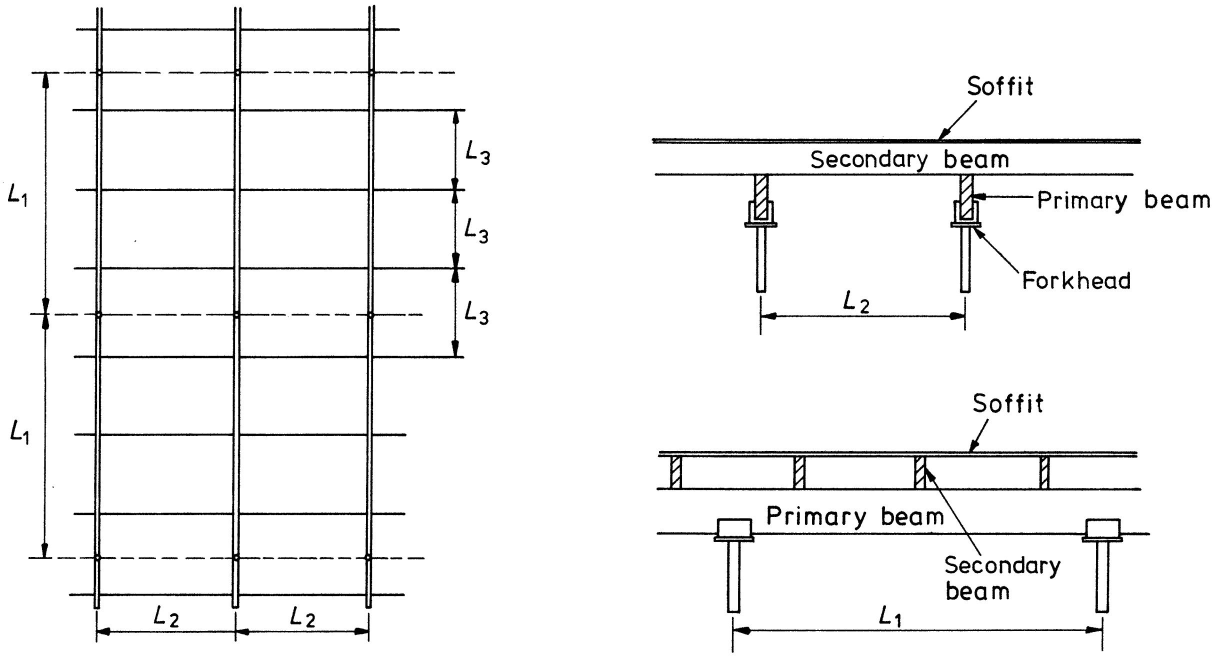
8.5 Dimensional information

8.5.1 Support of slabs

Arrangements of members are shown in Figure 19 and appropriate dimensions are given in Table 20 for falsework erected within the limitations of **8.4** for the construction of concrete slabs.

8.5.2 Support of individual beams

The support of formwork in which concrete will be cast to from a beam is a specific and limited example of the “standard solution” for slabs. Figure 20 is provided to indicate suitable methods of supporting in situ concrete beams and lintels and Table 21 gives recommended dimensions for use with Figure 20.



L_1 is the span of primary members between centres of verticals (e.g. props).

L_2 is the span of secondary members between centres of primary members and verticals.

L_3 is the span between centres of secondary members.

Figure 19 — Arrangement of members for construction of concrete slabs

Table 20 — Dimensions for standard solutions for slab support arrangements

Equivalent concrete thickness of slab (mm)	Primary members 150 × 75 nominal, single or twin	Max. span of primaries between centres of verticals (e.g. props) L_1 (m)	Secondary members nominal size (mm)	Max. span of secondaries between centres of primaries L_2 (m)	Max. span of ply sheets between centres of secondaries L_3 (mm)	Area of slab on each vertical support $L_1 \times L_2$ (m^2)	Falsework using props, Max. free height according to prop size (m) No. 1	No. 2	No. 3	No. 4	Falsework using tubes and couplers. Max. heights (m) Lift	Overall
150	SINGLE	1.38	100 × 50	1.15	610	1.587	3.12	3.25	3.75	4.00	2	6
	TWIN	1.96	100 × 50	1.15		2.254	3.10	3.10	3.10	3.60		
	SINGLE	1.27	100 × 75	1.43		1.816	3.12	3.35	3.50	3.85		
	TWIN	1.74	100 × 75	1.43		2.488	2.90	2.90	2.90	3.40		
	SINGLE	1.19	150 × 50	1.69		2.011	3.12	3.35	3.35	3.75		
	TWIN	1.59	150 × 50	1.69		2.687	2.75	2.75	2.75	3.25		
300	SINGLE	1.14	100 × 50	1.00	488	1.140	3.12	3.35	3.40	3.67	2	6
	TWIN	1.64	100 × 50	1.00		1.640	*	*	*	*		
	SINGLE	1.05	100 × 75	1.24		1.302	3.12	3.18	3.18	3.53		
	TWIN	1.45	100 × 75	1.24		1.798	*	*	*	*		
	SINGLE	0.99	150 × 50	1.47		1.455	3.00	3.00	3.00	3.41		
	TWIN	1.33	150 × 50	1.47		1.955	*	*	*	*		
450	SINGLE	1.07	100 × 50	0.85	488	0.910	3.12	3.35	3.40	3.67	2	6
	TWIN	1.49	100 × 50	0.85		1.266	*	*	*	*		
	SINGLE	0.99	100 × 75	1.05		1.040	3.12	3.18	3.18	3.53		
	TWIN	1.33	100 × 75	1.05		1.400	*	*	*	*		
	SINGLE	0.97	150 × 50	1.24		1.203	3.00	3.00	3.00	3.41		
	TWIN	1.22	150 × 50	1.24		1.513	*	*	*	*		

* Props cannot be used in these cases as they are not strong enough.

NOTE 1 This table to be read in conjunction with Figure 19.

NOTE 2 Larger timber sizes may be substituted for those given.

NOTE 3 Maximum load at prop base = 17 kN. Maximum load at scaffold tube base = 23.7 kN.

Dimensions in millimetres.

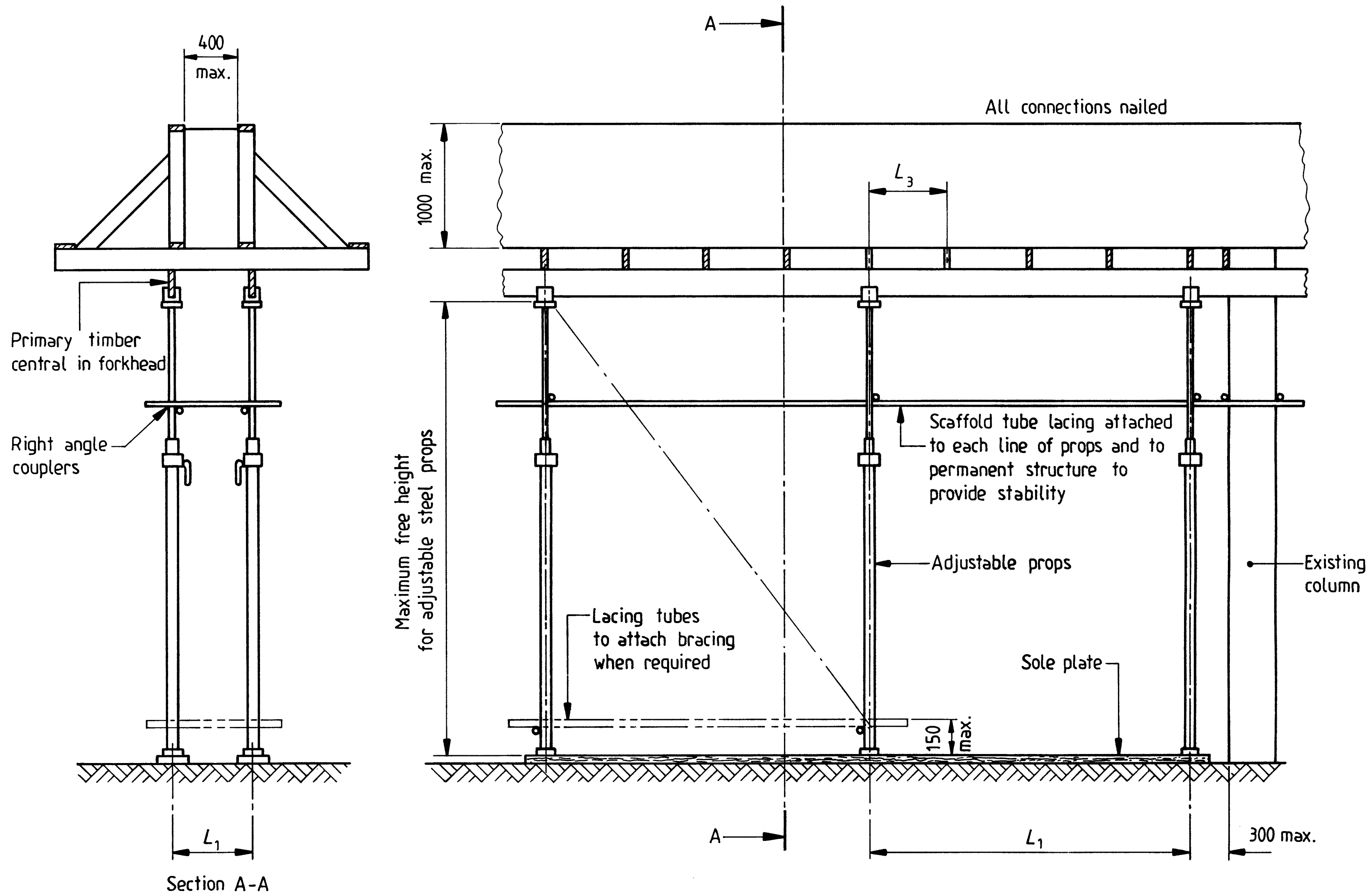


Figure 20 — Arrangement of members for typical beam support

Table 21 — Dimensions for standard solutions for beam support arrangements

Depth of concrete beam	Primary members nominal size	Max. span of primaries between centres of verticals (e.g. props) L_1	Secondary members nominal size	Max. span of secondaries between centres of primaries[a] L_2	Max. span of ply sheets between centres of secondaries L_3	Falsework using props, max. free height according to prop size m				Falsework using tubes and couplers, Max. heights m	
mm	mm	m	mm	mm	mm	No. 1	No. 2	No. 3	No. 4	Lift	Overall
up to 450	100 × 75	2.16	100 × 50	600	488	3.12	3.35	3.96	4.75	2	6
450 to 700	150 × 75	1.7	100 × 50	600	406	3.12	3.35	3.96	4.25	2	6
450 to 740	150 × 75	1.7	100 × 75	600	406	3.12	3.35	3.96	4.25	2	6
700 to 1 000	150 × 75	1.5	100 × 75	600	348	3.12	3.35	3.96	4.00	2	6

NOTE 1 This table should be read in conjunction with Figure 20.

NOTE 2 Larger timber sizes may be substituted for those given.

[a] This span should not normally be less than 400 mm.

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Annex A (informative) Permissible stresses and modulus of elasticity for steel grades generally used in falsework

A.1 *Permissible stresses*

For grade S275JR steel of BS EN 10025 up to and including 40 mm thick, the permissible stresses are as follows.

a) *Stresses due to bending*. For steel in good condition:

1) for parts in tension, the permissible tensile bending stress, p_{bt}, should not exceed 180 N/mm^2.

2) for parts in compression, the permissible bending stress, p_{bc}, should not exceed the value given in Table A.1 for the appropriate values of l/r and D/T;

where

l is the effective length of the compression flange (see Annex L);

r is the radius of gyration of the beam about its axis lying in the plane of bending (in mm);

D is the overall depth of beam (in mm) (see Figure A.1);

T is the mean thickness of the flange (in mm), i.e. area of the horizontal portion of the flange divided by the width (see Figure A.1).

b) *Direct tension*. The permissible axial tensile stress p_t on the net area of the section should not exceed 170 N/mm^2.

The thickness of an outstanding leg of any member in tension, unless the leg is effectively stiffened, should be not less than one-twentieth of the outstand.

c) *Direct compression*. The average axial compressive stress, F_c, calculated on the gross sectional area of axially loaded struts should not exceed the permissible axial compressive stress p_c given in Table A.2 for the greatest l/r ratio of the member concerned.

The thickness of an outstanding leg of any member in compression, other than load-bearing stiffeners (see Annex K), and unless the leg is stiffened, should be not less than one-sixteenth of the outstand.

d) *Combined stresses*. For combined bending and axial compression, the following expression should be satisfied:

$$\frac{F_c}{p_c} + \frac{F_{bc}}{p_{bc}} \leq 1$$

where

F_c is the maximum applied compressive axial stress (in N/mm^2);

F_{bc} is the maximum applied compressive bending stress (in N/mm^2);

p_c is the permissible axial compressive stress (in N/mm^2);

p_{bc} is the permissible bending stress in compressive members (in N/mm^2).

For loads due to wind see **6.4.3.1**.

e) *Shear*. For plates, flats and hot-rolled sections with a d_f/t_w ratio not greater than 85 (where d_f is the clear distance between flanges (in mm) and t_w is the web thickness (in mm) (see Figure A.1)):

1) the maximum shear stress, having regard to the distribution of stresses in conformity with the elastic behaviour of the members in flexure, should not exceed 125 N/mm^2;

2) the average shear stress in unstiffened webs of I beams should not exceed 110 N/mm^2;

3) the gross section of the web of rolled I beams and channels should be taken as the depth of the beam multiplied by the web thickness;

4) in calculating the resistance of tubes to shear, the total shear force at any section divided by half the gross sectional area of the tube should not exceed 125 N/mm^2.

f) *Bearing stress*. The bearing stress should not exceed 210 N/mm^2.

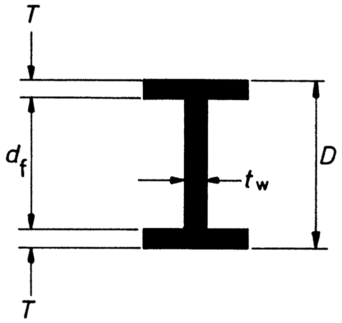


Figure A.1 — I-beam dimensions

A.2 Young's modulus

For all grades of structural steel the modulus of elasticity, E, should be taken as 210 kN/mm^2.

Table A.1 — Permissible bending stress in compressive members, p_{bc}, for beams

l/r_y	D/T									
	5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
40	180	180	180	180	180	180	180	180	180	180
45	180	180	180	180	180	180	180	180	180	180
50	180	180	180	180	180	180	180	180	180	180
55	180	180	180	178	176	175	174	174	173	173
60	180	180	176	172	170	169	168	167	167	166
65	180	180	172	167	164	163	162	161	160	160
70	180	177	167	162	159	157	156	155	154	154
75	180	174	163	157	154	151	150	149	148	147
80	180	171	159	153	148	146	144	143	142	141
85	180	168	156	148	143	140	138	137	136	135
90	180	165	152	144	139	135	133	131	130	129
95	180	162	148	140	134	130	127	125	124	123
100	180	160	145	136	129	125	122	119	118	117
105	180	157	142	132	125	120	116	114	112	111
110	180	155	139	128	120	115	111	108	106	105
115	178	152	136	124	116	110	106	103	101	99
120	177	150	133	120	112	106	101	98	96	95
130	174	146	127	113	104	97	94	91	89	88
140	171	142	121	107	97	92	88	85	83	81
150	168	138	116	100	92	87	82	79	77	75
160	166	134	111	96	88	82	77	74	72	70
170	163	130	106	92	84	77	73	69	67	65
180	161	126	102	89	80	73	69	65	63	60
190	158	123	97	85	76	70	65	61	59	56
200	156	119	95	82	73	66	62	58	55	53
210	154	116	92	79	70	63	58	55	52	50
220	151	113	90	77	67	61	56	52	49	47
230	149	110	87	74	65	58	53	49	47	44
240	147	107	85	72	62	56	51	47	44	42
250	145	104	83	69	60	53	48	45	42	40
260	143	101	80	67	58	51	46	43	40	38
270	141	98	78	65	56	49	45	41	38	36
280	139	96	76	63	54	48	43	39	37	35
290	137	94	75	61	52	46	41	38	35	33
300	135	93	73	60	51	44	40	36	34	32

Table A.2 — Permissible axial compressive stress, p_c, on cross-section

l/r	p_c (N/mm²) for grade 43 steel									
	0	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
0	170	169	169	168	168	167	167	166	166	165
10	165	164	164	163	163	162	162	161	160	160
20	159	159	158	158	157	157	156	156	155	155
30	154	154	153	153	153	152	152	151	151	150
40	150	149	149	148	148	147	146	146	145	144
50	144	143	142	141	140	139	139	138	137	136
60	135	134	133	131	130	129	128	127	126	124
70	123	122	120	119	118	116	115	114	112	111
80	109	108	107	105	104	102	101	100	98	97
90	95	94	93	91	90	89	87	86	85	84
100	82	81	80	79	78	77	75	74	73	72
110	71	70	69	68	67	66	65	64	63	62
120	62	61	60	59	58	57	57	56	55	54
130	54	53	52	51	51	50	49	49	48	47
140	47	46	46	45	45	44	43	43	42	42
150	41	41	40	40	39	39	38	38	38	37
160	37	36	36	35	35	35	34	34	33	33
170	33	32	32	32	31	31	31	30	30	30
180	29	29	29	28	28	28	28	27	27	27
190	26	26	26	26	25	25	25	25	24	24
200	24	24	24	23	23	23	23	22	22	22
210	22	22	21	21	21	21	21	20	20	20
220	20	20	20	19	19	19	19	19	19	18
230	18	18	18	18	18	18	17	17	17	17
240	17	17	17	16	16	16	16	16	16	16
250	16	15	15	15	15	15	15	15	15	15
300	11	11	11	11	11	11	10	10	10	10
350	8	8	8	8	8	8	8	8	8	8

Annex B (informative)
Properties of components in tube and coupler falsework

B.1 Properties of steel scaffold tube complying with the requirements of BS 1139

See Table B.1.

Table B.1 — Section properties of steel scaffold

Type of tube	Outer diameter mm	Nominal wall thickness mm	Mass per linear m kg/m	Cross-sectional area (A) cm^2	Moment of inertia (I) cm^4	Modulus of elasticity E N/mm^2	Elastic modulus z cm^3	Radius of gyration r cm	Minimum yield strength N/mm^2	Maximum allowable stress in bending p_{bc} N/mm^2	Maximum allowable stress in axial compression p_c N/mm^2	Maximum allowable shear stress N/mm^2	Maximum allowable stress in axial tension p_t N/mm^2	Stiffness EI N/mm^2
Steel tube complying with the requirements of BS 1139														
BS 1139-1:1982	48.3 ± 0.5	4.0	4.37[b]	5.57	13.8	210 000	5.70	1.57	210	139[a]	See Table B.3	93	127	$2\,898 \times 10^7$
BS 1139-1.1:1990	48.3 ± 0.5	4.0	4.37[b]	5.57	13.8	210 000	5.70	1.57	235	155	See Table B.2	104	142	$2\,899 \times 10^7$

[a] In the cases where tube may require the application of an allowance for corrosion, p_{bc} should be limited to 125 N/mm^2 for tubes having a yield stress of 210 N/mm^2 and 139 N/mm^2 for tubes having a yield stress of 235 N/mm^2.

[b] Tolerances in accordance with BS 1139-1.1:1990 are $\frac{+12.0}{-8.0}$ % on single tubes and ± 7.5 % on batches (10 tonnes or more).

B.2 Axial compressive stress

Values for axial compressive stress, p_c, are given in Table B.2 and Table B.3 and are based on the following Perry-Robertson formula appearing in Annex B of BS 449-2:1969:

$$K_2 p_c = Y_s + (\sigma + 1)C_o/2 - \sqrt{\{[Y_s + (\sigma + 1)C_o/2]^2 - Y_s C_o\}}$$

where

p_c is the axial compressive stress (in N/mm^2);

K_2 is the load factor from BS 449-2, for Table B.2 and Table B.3 the value of this load factor has been increased to 2.0 instead of 1.7 as used in BS 449-2;

Y_s is the minimum yield stress (in N/mm^2);

σ is the factor for slenderness ratio and is given by the expression:

$$\sigma = 0.3\left(\frac{l}{100r}\right)^2$$

C_o is the Euler critical stress (in N/mm^2) and is given by the expression:

$$C_o = \frac{\pi^2 E}{(l/r)^2} = \frac{\pi^2 210\ 000}{(l/r)^2}$$

where

l is the effective length (in mm);

r is the radius of gyration (in mm);

E is Youngs modulus of elasticity [taken as 210 000 N/mm^2 in the above expression (see **A.2**)].

B.3 Safe working loads for scaffold fittings complying with the requirements of BS 1139

Fittings complying with the requirements of BS 1139 will have the minimum capacities indicated in Table B.4 providing that they are in reasonable condition and properly fastened. They may be used on steel or aluminium tubes complying with the requirements of BS 1139 unless the supplier states that they are suitable for use only with certain types of tube.

Table B.2 — Maximum permissible axial stresses and loads in steel scaffold tubes manufactured in accordance with BS 1139-1.1:1990 with a yield stress of 235 N/mm²

Effective length	Slenderness ratio	"As new" tubes		"Used" tubes	
		Permissible axial compressive stress	Permissible axial load	Permissible axial compressive stress	Permissible axial load
l	l/r	p_c		p_c	
mm		N/mm²	kN	N/mm²	kN
0					
250	15.9	137.2	76.4	116.6	70.0
500	31.8	133.7	74.5	113.6	63.3
750	47.8	126.9	70.7	107.8	60.1
1 000	63.7	115.4	64.3	98.1	54.7
1 250	79.6	99.2	55.3	84.4	47.0
1 500	95.5	81.3	45.3	69.1	38.5
1 750	111.5	65.3	36.4	55.5	30.9
2 000	127.4	52.6	29.3	44.7	24.9
2 250	143.3	42.9	23.9	36.5	20.3
2 500	159.2	35.5	19.8	30.1	16.8
2 750	175.2	29.7	16.6	25.3	14.1
3 000	191.1	25.2	14.1	21.4	11.9
3 250	207.0	21.7	12.1	18.4	10.3
3 500	222.9	18.8	10.5	16.0	8.9
3 750	238.8	16.4	9.2	14.0	7.8
4 000	254.8	14.5	8.1	12.3	6.9
4 250	270.7	12.9	7.2	11.0	6.1
4 500	286.6	11.5	6.4	9.8	5.5
4 750	302.5	10.4	5.8	8.8	4.9
5 000	318.5	9.4	5.2	8.0	4.4
5 250	334.4	8.5	4.7	7.2	4.0
5 500	350.3	7.8	4.3	6.6	3.7
5 750	366.2	7.1	4.0	6.0	3.4
6 000	382.2	6.5	3.6	5.6	3.1

NOTE 1 It is recommended that, for columns carrying dead and imposed loads, $l/r < 207$.

NOTE 2 For struts and braces intended to carry wind loads and lateral forces, $l/r < 271$.

NOTE 3 For members designed as ties but which may suffer reversals of loading $l/r < 383$.

NOTE 4 Where there is combined bending and axial compression, treat as for combined stresses in BS 449, using the appropriate value of l/r and stress from Table B.2.

NOTE 5 For tube complying with BS 1139-1:1982, Table B.3 gives equivalent data.

Table B.3 — Maximum permissible axial stresses and loads in steel scaffold tubes manufactured in accordance with BS 1139-1:1982 (see Table B.2)

Effective length	Slenderness ratio	"As new" tubes		"Used" tubes	
		Permissible axial compressive stress	Permissible axial load	Permissible axial compressive stress	Permissible axial load
l	l/r	p_c		p_c	
mm		N/mm²	kN	N/mm²	kN
0		127	70.7	108	60.1
250	15.9	123	68.5	105	58.2
500	31.8	119	66.2	101	56.3
750	47.8	113	63.0	96.2	53.6
1 000	63.7	104	57.7	88.1	49.1
1 250	79.6	90.3	50.3	76.8	42.8
1 500	95.5	75.4	42.0	64.1	35.7
1 750	111.5	61.4	34.2	52.2	29.1
2 000	127.4	50.0	27.9	42.5	23.7
2 250	143.3	40.9	22.8	34.8	19.4
2 500	159.2	34.0	18.9	28.9	16.1
2 750	175.2	28.7	16.0	24.4	13.6
3 000	191.1	24.2	13.5	20.6	11.5
3 250	207.0	20.9	11.6	17.8	9.9
3 500	222.9	18.1	10.1	15.4	8.6
3 750	238.8	15.9	8.8	13.5	7.5
4 000	254.8	14.1	7.9	12.0	6.7
4 250	270.7	12.5	6.9	10.6	5.9
4 500	286.6	11.2	6.2	9.5	5.3
4 750	302.5	10.1	5.6	8.6	4.8
5 000	318.5	9.1	5.1	7.7	4.3
5 250	334.4	8.2	4.6	7.0	3.9
5 500	350.3	7.5	4.2	6.4	3.6
5 750	366.2	6.9	3.9	5.9	3.3
6 000	382.2	6.4	3.5	5.4	3.0

NOTE 1 It is recommended that, for columns carrying dead and imposed loads, $l/r < 207$.

NOTE 2 For struts and braces intended to carry wind loads and lateral forces, $l/r < 271$.

NOTE 3 For members designed as ties but which may suffer reversals of loading, $l/r < 383$.

NOTE 4 Where there is combined bending and axial compression, treat as for combined stresses in BS 449, using the appropriate value of l/r and stress from Table E.1.

Table B.4 — Safe working loads for individual couplers and fittings

Part or Section of BS 1139	Class	Type of coupler or fitting	Type of load	Safe working load
Part 2:1982	—	Right angle coupler	Slip along a tube	6.3 kN
Section 2.1:1991	A	Right angle coupler	Slip along a tube	6.3 kN
Section 2.1:1991	B	Right angle coupler	Slip along a tube	9.4 kN
Section 2.2:1991	—	Putlog ends	Shear strength	1.12 kN
Part 2:1982	—	Swivel coupler	Slip along a tube	6.3 kN
Section 2.1:1991	A	Swivel coupler	Slip along a tube	5.3 kN
Section 2.1:1991	A	Parallel coupler	Tension	4.7 kN
Part 2:1982	—	Sleeve coupler — frictional	Tension	3.0 kN
Part 2:1982	—	Sleeve coupler — frictional	Bending	0.79 kN·m[a]
Section 2.1:1991	A	Sleeve coupler — frictional	Tension	1.5 kN
Section 2.1:1991	A	Sleeve coupler — frictional	Bending	—
Section 2.1:1991	B	Sleeve coupler — frictional	Tension	3.0 kN
Section 2.1:1991	B	Sleeve coupler — frictional	Bending	0.59 kN·m
Part 2:1982	—	Putlog coupler	Force to pull the tube axially out of the coupler	0.53 kN
Section 2.2:1991	—	Putlog coupler	Force to pull the tube axially out of the coupler	0.53 kN
Part 2:1982	—	Joint pin (expanding spigot coupler)	Shear strength	21.0 kN
Section 2.2:1991	—	Joint pin (expanding spigot coupler)	Shear strength	21.0 kN
Part 2:1982	—	Putlog ends	Shear strength	1.12 kN
Part 2:1982	—	Adjustable base plate	Axial compression	30.0 kN
Section 2.2:1991	—	Adjustable base plate	Axial compression	30.0 kN

NOTE 1 In 1982, all couplers and fittings were specified in BS 1139 : Part 2.

NOTE 2 Couplers in accordance with BS 1139-2.1: 1991 are marked with EN 74 and where appropriate “A” or “B”.

NOTE 3 In some cases fittings can be subjected to more than one type of loading. In such cases consideration should be given to the combined effect.

[a] Based on steel tube in accordance with BS 1139-1:1982 (see foreword and Clause **3.8**).

Annex C (informative) Initial testing, quality control and inspection of falsework equipment

C.1 Introduction

Much of the falsework equipment in use is of a proprietary design that has been purchased or hired. Detailed information, such as that provided by the supplier, is of great importance in inspecting such equipment if the inspection is to be carried out by those not fully experienced in its use.

C.2 Tests on falsework equipment

Very often, technical information relating to the performance of such material has been compiled from tests carried out during the development of the equipment. It is desirable that test procedures for similar systems or components should be standardized in such a way as to make the critical properties comparable. Work has commenced on the production of suitable test procedures (BS 5507) for certain items of equipment and where they exist should be used by suppliers in compiling the necessary design data. If no standard test procedure exists the following points should be considered when establishing a test method.

a) The test should, as near as is practicable, simulate the conditions and manner in which the equipment is used on site.

b) Test methods and conditions should be easily repeatable and should be as simple as possible.

c) Testing should be carried out or supervised by an independent test house using suitable test machinery and recording equipment.

d) The test report should show

1) descriptions of components to be tested with drawings of items and relevant dimensions;

2) the arrangement of an item or system to be tested, with details of test rig, loading points, etc.;

3) a description of the test method or reference to a standard test procedure;

4) tabulation of test results, with test measurements;

5) a summary of the test and conclusions;

6) tensile or compression test results of samples cut from the tested items together with the strength range of the material from which the tested items will be made.

C.3 Prototype and initial testing

Where the strength of a manufactured component cannot be ascertained by applying design criteria recommended in this code, testing should be carried out at the prototype stage of development in order to obtain results, including ultimate behaviour, on which design data for the component or system can be based.

Because of the variability in the sizes and forms of falsework equipment, this code does not attempt to provide any data on test procedures. The reader should refer to recognized methods of test and assessment for further information when the testing of equipment is required, including the BS 5507 series of standards.

C.4 Quality control of manufacture

Good quality control and inspection during manufacture should generally make further destructive testing unnecessary, although it may be advisable to carry out cheek tests at further intervals.

Where necessary, test certificates should be obtained to ensure the raw material supplied for manufacture is in accordance with requirements. Where any doubt may exist, sampling and testing of material supplies should be undertaken.

Annex D (informative) Fatigue in Bailey Bridge sections

NOTE See **3.9.8**.

This annex is the complete text of the Department of Transport Technical Memorandum (Bridges) No. BE 13, which includes an MEXE report on fatigue in Bailey Bridges. It is reproduced by permission of the Controller of Her Majesty's Stationery Office; this memorandum is Crown Copyright.

Technical Memorandum (Bridges) No. BE 13

Fatigue risk in Bailey Bridges

1) The Military Engineering Experimental Establishment has drawn the Department's attention to the possibility of fatigue failure occurring in the main girder panels of Bailey Bridging when these are used in service for long periods. This arises because fatigue as a criteria was discounted in the original design on the grounds of the short life required from the bridge in its military role. A copy of the MEXE report is attached.

2) The most probable position for a fatigue crack to develop is at points of maximum stress in the tension chord at the sway brace slot as shown in the attacked Sketch at "A". Occasionally the cracks shown at "B" have been found under conditions of heavy shear.

3) The danger of a complete collapse as a result of fatigue cracking will depend on the make-up of the bridge. One crack in a "single-single" girder construction is much more dangerous than in a multiple girder construction where a measure of "fail-safe" is provided.

4) All highway authorities owning Bailey Bridging are therefore advised to observe the following precautions:

4.1) To arrange for an immediate inspection to be made of bridges in service and thereafter at regular intervals.

4.2) If the bridge has been recently painted then the paint should be removed from the critical zones to ensure detection of cracks. Cracks are more readily discernible when the structure is under load and only those visible to the naked eye need cause concern.

4.3) Whenever visible cracks are detected the panel should be taken out of service immediately. No attempt should be made to repair cracks by welding.

4.4) Where the risk of complete collapse is revealed emergency action should be taken to relieve the bridge of load by road closure and by shoring.

4.5) When any structure is dismantled the bridge panels should be carefully examined for cracks before repainting and stock piling for further use. Stored panels should be carefully examined before building into a bridge structure.

5) The foregoing precautions should also be observed when Bailey Bridging panels are used by a Contractor for Temporary Works.

6) Further information on fatigue in Bailey Bridges may be obtained by reference to a paper published by the British Welding Journal, April 1960, pages 272-280 ("Programmed fatigue testing of full-sized welded steel structural assemblies" by J.G.Whitman and J.F.Alder).

7) This memorandum is also being circulated to the British Railways, London Transport and British Waterways Boards and Highway Authorities are requested to bring it to the attention of any other private bridge owner.

(L.R.Greenaway) Assistant Chief Engineer, Bridges Engineering Division, Ministry of Transport.

Military Engineering Experimental Establishment Report

Fatigue in Bailey Bridges

It is known that Bailey Bridges are being used in service for very long periods and the purpose of this note is to sound a warning on the possible dangers of fatigue failure of the main girder panels which may result if the original design of any particular Bailey Bridge did not take such long life into account.

The Bailey Bridge was designed to meet a military requirement where the lightness and transportability of the equipment was of paramount importance. The military load classification, therefore, represents the greatest loads that the structure can safely carry on short term considerations. Fatigue as a criterion in design was discounted on the grounds of the short life required from the bridge in its military role.

As a general statement it can be said that the danger of a fatigue failure will arise if the make-up of a Bailey Bridge was decided on the basis of the military load classification, and if the subsequent civilian loading approached this value for long periods. Even under such conditions, however, ten years or more have been known to elapse before fatigue cracks develop because the normal traffic spectrum includes a majority of light vehicles.

The danger becomes more acute when local circumstances give rise to a larger proportion of vehicles which load the bridges to capacity, and this danger becomes particularly acute if the make-up of the bridge is designed for a low military class (e.g. Class 9), where common civil vehicles such as 3-ton lorries stress the bridge members to their "short life" limit.

Fatigue life in a bridge can be controlled through the level of live load stress and so any new design should be based on a stress level which will ensure an adequate life to meet the particular requirement. It is, therefore, recommended that for civilian use the make-up of the main girders of the Bailey Bridge are designed so that the stresses do not exceed the values laid down in British Standard 153:1966 revision (Steel girder bridges) as given for British Standard 968[2)] Steel (Case G).

If any existing bridge is found to be carrying loads in excess of such recommendations, a very careful search should be made for the existence of fatigue cracks. The most probable position for a fatigue crack to develop is at points of maximum stress in the tension chord at the sway brace slot as shown in the attached Sketch at "A". Very occasionally the cracks shown at "B" have been found under conditions of heavy shear. Examination for such cracks should be carried out some time after the application of any fresh coats of paint and are more readily discernible when the structure is under load.

Only cracks visible to the naked eye need cause any concern – cracks less than ¼ in. (6 mm in length are, in fact, very fine, and not easy to see, but if they are found there may remain many years of life to the bridge. The rate of growth of such cracks however, should be carefully watched and it is recommended that they are not allowed to exceed ½ in. (12 mm) in length — particularly if more than one crack is occurring as illustrated — before the component is taken out of use.

As the crack grows in length it becomes more easy to detect, and when it exceeds ½ in. (12 mm) in length and begins to open up, it is readily discernible. At this stage, however the component will be getting dangerously near to the end of its fatigue life and immediate actions should be taken.

Naturally, the danger of complete collapse as a result of such cracks will depend on the make-up of the bridge. One crack in a single-single construction is much more dangerous than in a multiple construction where a fair measure of "fail safe" is provided. No attempt should be made to repair such cracks by welding.

Finally, it should be realized that fatigue is cumulative and no significant recovery takes place while the equipment is stored in an unstressed condition. When any structure is dismantled, therefore, the bridge panels should be carefully examined for this type of crack before repainting and stock-piling for further use. Also, existing stocks should be carefully examined before building into a structure where fatigue conditions would apply.

[2)] BS 968 is now superseded. See BS EN 10025, BS EN 10113, BS EN 10137, BS EN 10155, BS EN 10210-1, BS 7613 and BS 7688

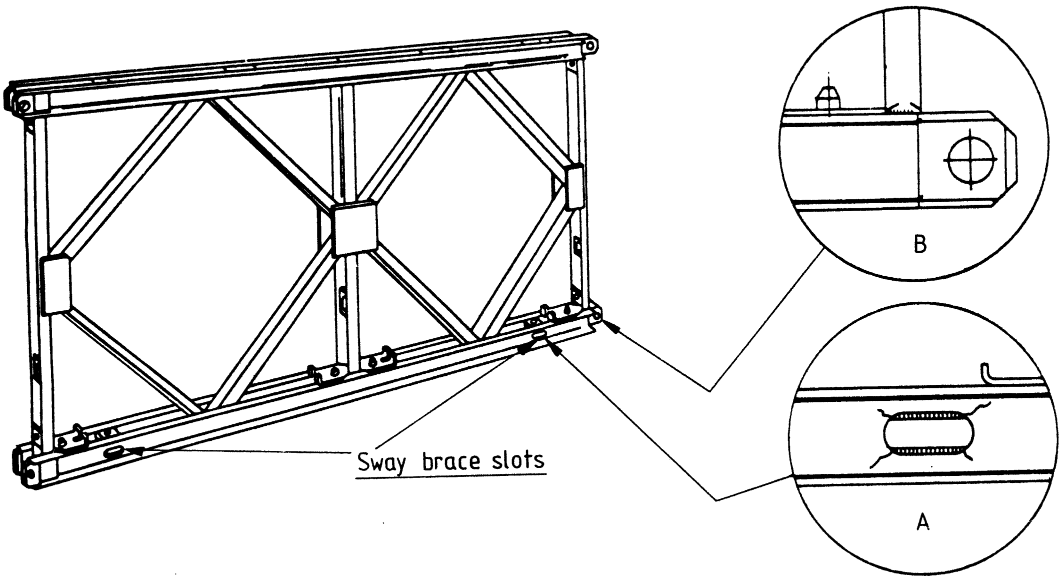


Figure D.1 — Bailey panel: possible locations of fatigue cracks

Annex E (informative)
Additional data on material properties

NOTE See Annex A and Annex B for steel and steel scaffold tube respectively.

E.1 Concrete shrinkage and creep

Concrete shrinkage: 250×10^{-6} progressively up to 28 days.

Concrete creep: $\dfrac{2\ 000 \times 10^{-6}}{U}$ per N/mm^2

where U is the concrete cube strength (in N/mm^2) at time of loading, the order of strain due to creep being estimated as being equivalent to twice that for concrete shrinkage.

E.2 Coefficient of linear expansion/contraction

Steel: 10×10^{-6}/°C
Concrete:
limestone or granite aggregate: 5×10^{-6}/°C
flint or quartzite gravel aggregate: 10×10^{-6}/°C

E.3 Modulus of elasticity[3)]

The modulus of elasticity for concrete is given in Table E.1.

Table E.1 — Modulus of elasticity for concrete

Concrete cube strength, *U* N/mm^2	Modulus of elasticity, *E* kN/mm^2
20	25.0
25	26.5
30	28.0
35	29.5
40	31.0
50	34.0

E.4 Density of reinforced concrete

E.4.1 If the quality of steel by volume exceeds 2 % and the aggregates are of normal density, the density should be taken as given in Table E.2 (the values on this table are based on a density of unreinforced concrete of 2 400 kg/m^3).

Table E.2 — Density of reinforced concrete

Steel quantity (by volume) %	Density kg/m^3
3	2 550
4	2 610
5	2 660
6	2 720

E.4.2 If the relative density of any of the aggregates exceeds 2.75, the density of the concrete should be determined by calculation or density test.

E.4.3 The density of light-weight concretes should be determined from trial mixes. As a guide, the likely ranges of densities for certain types of light-weight concretes are given in Table E.3.

Table E.3 — Density ranges for light-weight concretes

Concrete type	Density range (without reinforcement) kg/m^3
No-fines, with normal density aggregates	1 600 to 2 000
Lightweight coarse aggregate and normal density fine aggregate	1 600 to 2 000
Lightweight coarse and fine aggregates	1 575 to 1 800

E.5 Masses and densities of materials

The mass of various falsework components is given in Table E.4. Table E.5 gives masses and densities of men and various materials.

E.6 Masses of corrugated steel sheeting

Table E.6 gives the mass of sheeting to be allowed for t n the design of temporary structures. Allowance should also be made for the amount of overlap of the sheets and the additional mass of the fittings.

E.7 Some unusual loads that frequently require consideration

a) *Accumulation of debris* on boarded decks from demolition and flue cleaning/boiler scaling should be assumed to have a density of 1 600 kg/m^3.

b) *Ice formation at high altitudes*: a covering of ice 2 mm thick on a scaffold tube adds approximately 0.3 kg/m to the mass of the tube.

c) *Concrete spillage* on the top of lacing adds approximately 1.5 kg/m to the mass of the tube.

[3)] Further guidance may be obtained from BS 8110

Table E.4 — Masses of scaffolding materials

Scaffolding materials	Mass
Steel scaffold tube,	
48.3 mm diameter	4.37 kg/m
Aluminium scaffold tube,	
48.3 mm diameter	1.66 kg/m
Steel couplers and fittings	1.00 kg to 2.25 kg
Boards,	
38 mm thick	6 kg/m or 25 kg/m^2
50 mm thick 225 mm wide	8 kg/m or 33 kg/m^2
63 mm thick	10 kg/m or 41 kg/m^2
Steel props (approximate individual masses) to BS 4074:	
size no. 0	15 kg
size no. 1	22.7 kg
size no. 2	23.6 kg
size no. 3	26.3 kg
size no. 4	35.6 kg

Table E.5 — Masses and densities of men and materials

Item	Mass/density
Man (average)	80 kg
Man with small tools (average)	90 kg
Spot board and mortar	30 kg
Wheelbarrow full of mortar	150 kg
Tarpaulins and fixings	1 kg/m^2
Ladders and fixings	8 kg/m
100 bricks	275 kg
Timber (softwood)	500 kg/m^3 to 650 kg/m^3
180 litres of water or liquids in containers	200 kg
Packaged flooring tiles, ceramic tiles, roofing tiles, slates	1 600 kg/m^3

Table E.6 — Masses of corrugated steel sheeting

Standard thickness	Approximate equivalent standard wire gauge	Number of corrugations (nominal cover width is given in brackets)									
		8/3 (610 mm)		10/3 (762 mm)		10 1/2/3 (800 mm)		12/3 (914 mm)		12 1/2/3 (952 mm)	
mm		kg/m	m/tonne	kg/m	m/tonne	kg/m	m/tonne	kg/m	m/tonne	kg/m	m/tonne
0.425		2.50	400	3.05	328	3.18	315	3.59	278	3.80	263
0.50	26	2.94	341	3.59	279	3.74	267	4.23	236	4.47	224
0.60	24	3.52	284	4.31	232	4.49	223	5.07	197	5.36	187
0.70	22	4.11	243	5.02	199	5.23	191	5.92	169	6.25	160
0.80		4.70	213	5.74	174	5.98	167	6.77	148	7.15	140
0.90		5.28	192	6.45	156	6.73	151	7.61	133	8.04	126
1.00	20	5.87	170	7.17	139	7.48	134	8.46	118	8.93	112
1.20	18	7.05	142	8.61	116	8.97	111	10.15	99	10.72	93
1.60		9.40	106	11.48	87	11.96	84	13.53	74	14.29	70
2.00		11.75	85	14.35	70	14.95	67	16.92	59	17.87	56

[a] The nominal cover width is the width of the sheeting after corrugation, and is subject to manufacturing tolerances.

Annex F (informative)
Wave forces

F.1 Definitions

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

F.1.1 *bore*

A very rapid rise of water level in which the advancing water presents an abrupt front of considerable height; usually associated with shallow estuaries.

F.1.2 *clapotis*

The standing wave phenomenon caused by the interaction of an incident non-breaking wave train and its reflection from a vertical or near-vertical face.

F.1.3 *fetch*

An area in which waves are generated by wind.

F.1.4 *still water depth* (d)

The depth of the water at the structure if all wave action was absent.

F.1.5 *still water level*

The elevation that the sea surface would assume if all wave action was absent.

F.1.6 *wave height* (H)

The vertical distance from crest level to the preceding trough level.

F.1.7 *wave length* (L)

The horizontal distance between similar points on two successive waves.

F.2 Symbols

For the purposes of this annex only, the following symbols apply.

Symbol	Quantity	Unit of measurement
d	still water depth	m
H	wave height	m
L	wave length	m
P_c	maximum water pressure at level C	kN/m^2
P_1	clapotis pressure	kN/m^2
w	force applied per unit volume of water	kN/m^3

F.3 General

Where falsework is erected in or adjacent to water, it may be subjected to wave forces. In marine locations this is a probability, but elsewhere is a possibility that should be considered.

There are three main sources of waves:

a) wind generated waves;

b) oscillations set up by moving objects, e.g. passing boats;

c) tidal surges or bores in certain estuaries

The prediction of wind generated waves on a particular site can either be by observation over a representative period of time in relation to the exposure of the falsework or by calculations based on exposure, sea fetch and other relevant factors, or by reference to the Marine Information and Advisory Service[4]. Wind generated waves will build up slowly and last for correspondingly longer periods. The oscillations, waves and washes set up by boats and floating craft passing near by will be of comparatively short duration and un likely to equal the magnitude of wind generated waves. It is possible, but unlikely, that oscillations leading to substantial waves will be generated by pumps, paddles or similar machinery. Tidal bores are more accurately known than the foregoing and can be predicted as single waves. It is suggested that the full hydrostatic pressure force be used for the maximum height of a single wave impacting the falsework with no allowance for any dynamic force in the unlikely event of the single wave breaking on to the falsework.

The analysis of wave forces depends on the types of wave at the structure.

There are three distinct types of waves:

1) non-breaking waves (see **F.4**);

2) breaking waves;

3) broken waves.

Breaking waves will result in higher dynamic loads being applied to the falsework than from non-breaking or broken waves. Generally, non-breaking wave conditions will prevail when the wave height at a wall is less than 1.7 times the still water depth at the wall.

The location of certain structures, e.g. protective structures, will be such that waves will break before striking them. The forces are complex and no formulae have been developed in detail.

As a guide only, the methods of analysis given in **F.4** based on vertical walls in front of waves, may be considered, but it is recommended that, wherever possible, the advice of an engineer experienced in marine work should be obtained, and preferably that protective measures are installed to prevent waves breaking on to the falsework or formwork.

Three detailed references for wave forces are:

i) BS 6349 *Code of practice for maritime structures*;

ii) Shore Protection Manual Volume II Chapter 7-3 Wave forces [11];

iii) Indian Standard IS 4651-III.

F.4 Non-breaking waves

Generally, where the incident wave height at a wall is less than 1.7 times the still water depth at the wall, the wave may be assumed to be non-breaking and to form a clapotis or standing wave. The forces are essentially hydrostatic but the wave is larger near to the wall owing to the rebound phenomenon or clapotis being formed.

The incident wave of length L and height H, on impacting with the vertical wall, forms a larger wave called a clapotis. The clapotis pressure, p_1, of the clapotis, adds to or reduces the still water hydrostatic pressure (see Figure F.1).

p_c, (in kN/m^2), the maximum water pressure at level C, has a pressure diagram of triangular shape ABC (see Figure F.1) and is given by the expression:

$$p_c = wd + \frac{wH}{\cosh\left(\frac{2\pi d}{L}\right)}$$

where

w is the force applied per unit volume of water, i.e. 9.81 kN/m^3;

d is the still water depth (in m);

H is the wave height of original free wave (in m);

L is the wave length (in m).

[4] Marine Information and Advisory Service, Brook Road, Worrnley, Godalming, Surrey, GUX 5UB

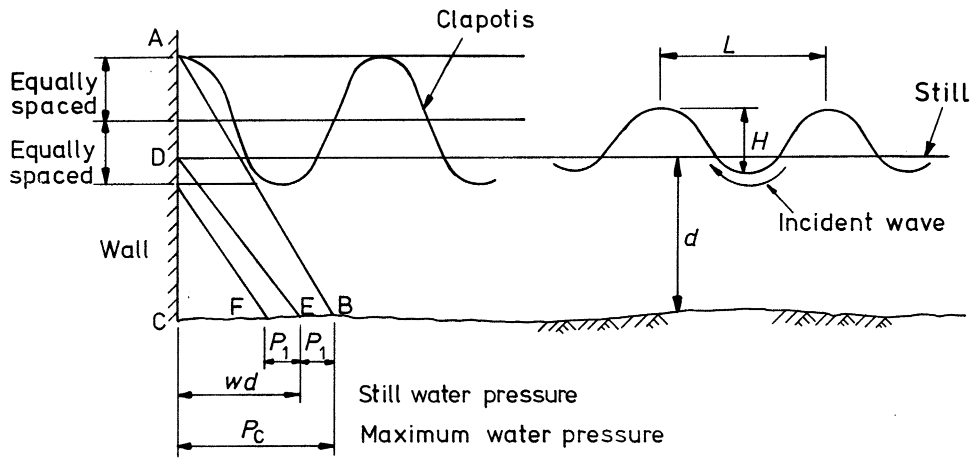


Figure F.1 — Non-breaking waves: section diagram

Annex G (informative)
Site investigations for foundations for falseworks

G.1 Introduction

The principles for site investigations for foundations for falseworks are the same as those for site investigations for the foundations of permanent works. However, the foundations for falseworks are often set at shallow depths and hence the investigation should pay special attention to the character of the ground at these depths. Further information on site investigation practice should be obtained from BS 5930, BS 8004 and BS 1377.

G.2 General

A site investigation should normally investigate the following features:

a) the character of the ground;

b) the ground water conditions;

c) the engineering properties of those strata relevant to the design of the foundations.

G.3 Preliminary appraisal

A preliminary appraisal consists of a review of information concerning the available site, e.g. geological maps, and a walk-over inspection of the site paying particular attention to the geology, previous uses of the site and the type and performance of any existing foundations. On the results of this, the site investigation is planned.

G.4 Depths for investigation and sampling

The following gives an outline of the methods used for routine investigations, but other methods may be required for special ground conditions.

a) *Gaining access to the ground*

1) Shallow machine-dug trial pits (maximum depth approximately 5 m).

2) Hand auger boring (suitable only for cohesive soil).

3) Cable percussion boring (i.e. the "shell and auger" method, suitable for any soil or weak rock).

4) Rotary core drill (used for proving rock).

b) *Determining ground water conditions*

1) Standing water levels can be measured in pits and bore holes provided these have been left open long enough for the water level to reach equilibrium.

2) Piezometers (stand pipes) can be installed as an alternative to leaving open the pits and bore holes. Grout seals may be required if there is more than one water-bearing stratum.

c) *Samples*

1) Disturbed samples taken from the drill tools or excavator bucket.

2) Thick-wall samples taken in connection with the standard penetration test (see **3.3** (test 19) of BS 1377-9:1990), and suitable only for visual examination or laboratory classification tests.

3) 100 mm general purpose samples (see BS 5930), nominally undisturbed and suitable for most laboratory tests.

d) *Bore hole tests*

1) Standard penetration test (see **3.3** (test 19) of BS 1377-9:1990) gives an indication of the consistency of soils and weak rocks and is widely used in granular soils.

2) Vane test (see **4.4** (test 18) of BS 1377-9:1990) measures the shear strength of cohesive soils and is particularly useful in soft clays.

e) *Frequency of samples and bore hole tests*

The following general procedures will be found suitable for most investigations.

1) A disturbed sample should be taken at the top of each new stratum and thereafter at 1 m intervals.

2) In cohesive soils, a 100 mm general purpose sample should be taken at the top of each new stratum, at proposed foundation depth, and at 1 m intervals through each stratum.

3) In granular soils, standard penetration tests should be taken at similar intervals to those for the 100 mm samples

4) In mixed granular and cohesive soils, 100 mm samples and standard penetration tests should be carried out alternately.

G.5 Laboratory tests

In the laboratory, all the samples should be examined and described and on the basis of this, together with the reports of the site work, samples should be selected for testing to determine the classification (see Clause **3** (test 1), Clause **4** (test 2), Clause **5** (test 3), Clause **5** (test 4) and Clause **9** (test 7) of BS 1377-2:1990) and strength characteristics (see Clause **3** (test 17) of BS 1377-5:1990, Clause **7** (test 20) and Clause **8** (test 21) of BS 1377-7:1990) as appropriate to the design of the foundations.

G.6 Final report and recommendations

The report should include a description of:

a) The character of the ground assessed from the preliminary appraisal and from the logs of the individual pits and boreholes, and the results of the laboratory examination of the samples;

b) details of the ground water conditions assessed from the locations of water-bearing strata and the ground water observations;

c) the design criteria of the strata derived from items a) and b).

Annex H (informative)
Examples of design brief contents

H.1 Design brief for a motorway bridge

a) Schedule of drawings of the permanent works.

b) Programme for the construction of the permanent structure.

c) Plan of ground and foundation areas beneath and adjacent to the area to be supported, showing:

1) contours and/or cross-sections showing ground level variations;

2) delineation of any areas to be left clear for access or other reasons, both below and adjacent to the falsework.

d) Site investigation and other soils data relating to the ground on which the falsework may be founded.

e) Investigation of any water courses or sources that might affect the site by altering soil properties, scour, flooding or creation of loads.

f) The limit of loading that may be applied to any permanent works foundations from the falsework.

g) Positions of all services, including overhead power lines, adjacent to the works and clearances required by statutory authorities from roads, railways, power lines, etc., including any special requirements of these bodies.

h) Construction method for the permanent works, defining:

1) sequence of timing of load application;

2) positions of construction joints;

3) positions of any infill or shrinkage bays and timing of their concreting;

4) limitations of absolute and differential settlements during construction;

5) timing and sequence of any post-tensioning.

i) Any loads that will be applied to the permanent works before the falsework can be removed.

j) Any significant redistribution of load in the permanent works before the falsework can be removed, including those due to prestressing, creep, shrinkage and temperature.

k) Loads that will be applied to the falsework by the method of construction from plant, stacking and handling of materials, from access and attachments.

l) Exposure to wind for assessment of wind loads.

m) General specification of the falsework components to be used.

n) Criteria that will determine the stripping times of the formwork and dismantling times of the falsework, including any phasing of the removal of falsework to limit stresses in the permanent works during the removal operation.

H.2 Design brief for a suspended floor in a building

a) Schedule of drawings of the permanent works.

b) Programme for the construction of the permanent structure.

c) Plan of area below showing type of construction of any permanent works already in place.

d) Data on the ability of the floor or ground below to accept the loads transmitted through the falsework.

e) Any loads that will be applied to the permanent works before the falsework can be removed.

f) Construction method for the permanent works, defining:

1) sequence of timing of load applications;

2) positions of construction joints.

g) Exposure to wind for assessment of wind loads.

h) Any loads that will be applied to the falsework not covered by general allowances, such as those caused by a particular method of concreting, stacking of materials, access across slab or by attachments to the falsework

i) General specification of the falsework components to be used.

j) Criteria that will determine the stripping times of the formwork and dismantling times of the falsework.

Annex J (informative)
Forces from concrete on sloping soffits

J.1 General

In typical formwork with only horizontal and vertical faces, lateral pressures on the formwork are generally resisted by each opposite side, and there is no horizontal resultant of force external to the formwork. This assumes that there is adequate structural connection between the various parts of the formwork resisting the concrete pressure so that the basic formwork is not pulled apart. If the concrete is to be poured against previously placed concrete, the latter will form part of the formwork system, and therefore needs to be effectively connected to the other formwork elements. The only force exerted by the concrete that requires to be resisted by the falsework is that resulting from the self-weight of the concrete and its formwork. This load has to be resisted externally to the formwork itself, by the falsework and the ground or some other "immovable" object. The forces associated with concrete with a top or bottom surface that is not level require more careful consideration.

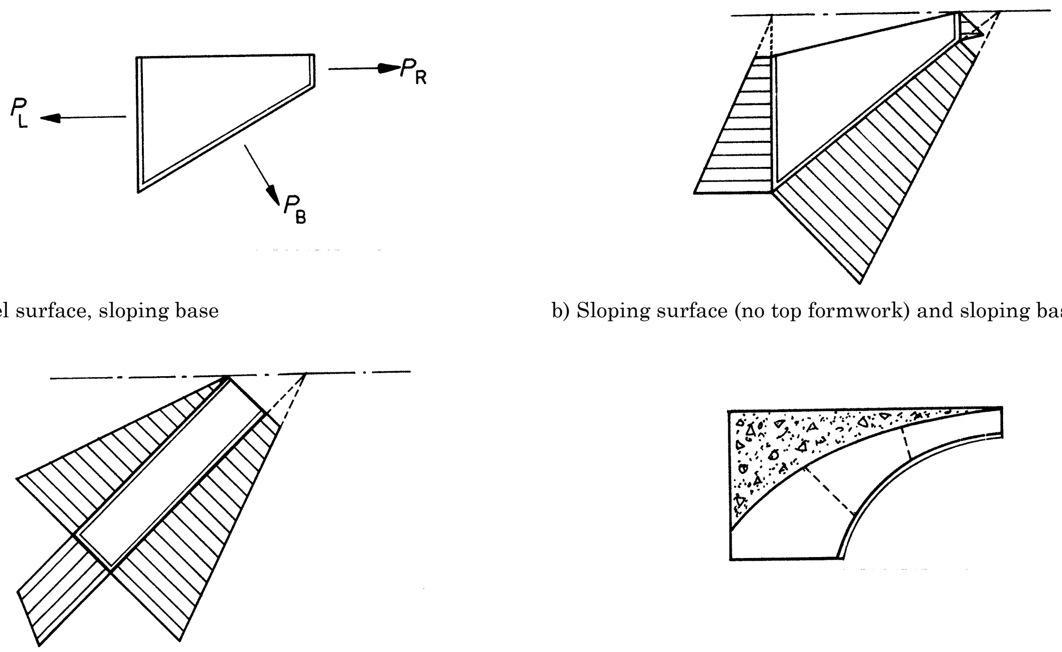
In **J.2**, **J.4** and **J.5** wet concrete is assumed to be a simple liquid, which is not quite true. However, it simplifies understanding of the concept, enabling pressures, and hence loads, to be calculated simply.

J.2 Forces associated with a level upper surface and a sloping base (See Figure J.1a))

The pressure on the formwork will be normal to the face under consideration and so, on a vertical face, the pressure will be horizontal, and on an inclined soffit, normal to the slope. Using the appropriate areas, the total forces can be calculated, P_L or P_R and P_B respectively. P_B can be resolved into horizontal and vertical components, P_h and P_v. The algebraic total of the horizontal forces P_L, P_R and P_h will be zero and P_v will equal the mass of the concrete. Thus it is only the vertical force due to the mass of the wet concrete plus the mass of the formwork that needs to be resisted by the falsework.

Usually the formwork system will comprise the sheeting and its immediate framing. When retaining the wet concrete, the soffit or the formwork connecting two opposite sides will be in tension because one side is restraining the other. Once the concrete stiffens to become self-supporting and the side forms are removed, this tension force will disappear, and the situation is almost as if the concrete had been precast elsewhere and then placed in this position. Such a precast unit can be set down on a sloping support, provided there is enough friction to stop it sliding. There are no lateral pressures, so the only force it applies to the support is due to gravity, i.e. vertical. This force can be resolved into two components, one normal to the surface and one tangential. Should the frictional force generated from the mass of the concrete on the formwork be less than the resolved tangential force, the block is likely to start to slide down the slope (see **6.4.4.6**). Once this happens, the change of momentum will induce a horizontal force in the formwork, this in turn tending to make the falsework move the other way. This horizontal force is a consequence of the change of momentum so it does not arise until movement of the concrete has commenced. The falsework and formwork therefore need to be designed to prevent the commencement of such movement if collapse is to be avoided.

Concrete cast in situ will tend to form a very intimate contact with the formwork (as is found on stripping) and thus the adoption of the basic coefficient of friction will result in underestimation of the actual lateral restraint being provided by the formwork. However, if the load is from an object not cast in situ, there will be no additional safeguard, and the limiting value of frictional force calculated in accordance with **6.4.4.6** will apply. Any sloping interface requires consideration wherever it may be in the falsework. Tapered packs may be provided to enable sloping formwork to be supported by forkheads and there will be no horizontal force on the horizontal face of such a pack, but the sloping surface should be considered. Where supports are not vertical, horizontal forces are generated.



a) Level surface, sloping base

b) Sloping surface (no top formwork) and sloping base

c) All surfaces sloping and with top formwork

d) Arched surfaces

Figure J.1 — Distribution of forces on sloping soffits

J.3 Forces when the top and base of concrete are sloping, but there is no top formwork
(See Figure J.1b))

Where the required slope is sufficiently gentle to be retained by the friction of concrete, it is possible to omit top formwork. In such a case the hydrostatic basis assumed in **J.1** is no longer applicable; no simple theory enabling an accurate evaluation of the resultant forces has been put forward so calculations have to be based on assumptions about the way the concrete acts. When assessing the forces involved, where there is a choice of possible solutions the most conservative alternative should be taken. Assessment of the forces at interfaces below the soffit may be carried out in the same manner as described in **J.2**, being related only to the mass of the concrete.

J.4 Forces when the top and base of the concrete are sloping and top formwork is in use
(See Figure J.1c))

Where there is a top form, there will be an upward force exerted by the upper surface of the concrete on this form; the fixing of such a top form will mean a reversion to hydrostatic behaviour and the loads will approach the theoretical. As in **J.3** there is a small uncertainty due to the open end of the formwork, but it can be ignored for all practical purposes. The vertical force calculated by multiplying the vertical component of the pressure in the soffit by the projected area of the soffit is greater than the mass of the concrete. However, if the corresponding force due to the pressure against the top formwork is deducted, the difference will be equal to the mass of the concrete. Provided that the formwork is adequately designed, so that the pressures from opposing faces of the formwork are resisted by each other, the load to be carried by the supporting falsework will be the self-weight of the concrete plus that of the formwork.

J.5 Arches (See Figure J.1d))

In some cases, the considerations in **J.1**, **J.2**, **J.3** and **J.4** will apply directly to concrete arches cast in situ. Where arches are large, constructed in several parts with top formwork, and supported on vertical falsework, the forces associated with each part will be similar to those described in **J.4**. Provided the overall capacity of the falsework is adequate for the whole arch, only a few special checks will be needed, including the effect of casting a block adjacent to an existing one. The pressure at such an interface will tend to push the new concrete away from the earlier concrete. The formwork system for each block should take this force into consideration as well as the force on the forms themselves.

The forces involved may be contained within the formwork itself, or use may be made of strutting from the top form to other parts of the permanent structure or to the ground behind the springing.
The forces are those that would have been resisted by traditional formwork ties. Below they have to be taken via the falsework to solid ground. Above they should be taken by struts to ground at the back or side, thus completing the structural system. Such forces will have a large horizontal component.

Another way to assess an in situ concrete arch with a sloping top form is to consider the forces that would result if the top form were omitted and the side form were continued upwards, so that the top of the concrete would be level. This is shown hatched in Figure J.1d). In this case, the force on the lower surface is that caused by this larger volume of concrete, and can be seen to be directly related to the hydrostatic pressure. Reversion to the actual situation, by removing the extra concrete and putting in a top shutter, does not affect the force on the lower formwork, as this is due to hydrostatic pressure. However, there is now a force on the top formwork in the opposite direction, and the net force transmitted to the falsework is the vertical sum of the forces on the top and bottom formwork, which is equal to the mass of the concrete. Thus the pressures can be assessed by considering the total concrete that would be there, if it found its own level at a free surface, whereas the force on the falsework is the mass of the concrete actually there. In practice, the theoretical hydrostatic pressure will often not be reached and where procedures to limit concrete pressure are adopted, design figures may be reduced accordingly.

a) Level surface, sloping base

b) Sloping surface (no top formwork) and sloping base.

c) All surfaces sloping and with top formwork.

d) Arched surfaces.

Annex K (informative)
Design of steel beams at points of reaction or concentrated load

K.1 General

In falsework structures, where high concentrations of loading may often occur on relatively light members and there is generally less joint rigidity and restraint than in permanent works, the problem of local instability due to web crushing and buckling becomes more critical. For structural steelwork, web stiffeners should be provided at all loading transfer points including supports, unless calculations are provided to show that such stiffeners are not required for the actual conditions applying and materials to be used on site.

The eccentricities of loading that may arise through workmanship factors (see **7.3.2**) should be considered in design.

K.2 Beams without bearing stiffeners

Unless the following criteria are satisfied, load-bearing web stiffeners will be required (see Annex A for permissible stresses for grade S 275 steel of BS EN 10025).

a) *Web buckling*. The reaction or concentrated load, R (in N), should not be greater than the allowable buckling load, which is given by the expression:

$$R = (b + n_1)\ t_w p_{cw}$$

where

b is the stiff length of the bearing (in mm);

n_1 is the length obtained by dispersion at 45° from the extreme of the stiff bearing, through flange plates if any to the mid-height of the web (in mm) (see Figure K.1);

t_w is the web thickness (in mm);

p_{cw} is the permissible compressive stress in the web, determined assuming that the web acts as a strut and taken as equal to the value of the permissible axial stress p_c for a compression member whose effective length is given in Table K.1 and with a radius of gyration r_g is given by:

$$r_g = \frac{t_w}{\sqrt{12}}$$

The length of stiff bearing, b, should be determined by dispersion of load at a slope of 1:1 through solid steel material which is fixed, see Figure K.1 No dispersion should be taken through packs unless they are secured by wilds or clamping capable of transmitting shear forces.

b) *Web crushing*. The reaction or concentrated load, R (in N), should not be greater than the allowable buckling load, which is given by the expression:

$$R = (b + n_2)\ t_w\, p_b$$

where

b and t_w are as defined in item a);

n_2 is the length obtained by dispersion at 30° from the extreme end of the stiff bearing through the flange plates, if any, to the web (see Figure K.2);

p_b is the allowable bearing stress (in N/mm^2).

c) *Combination of longitudinal and buckling stresses*. Where the reaction or concentrated load is applied to a compression flange that has a longitudinal stress of more than 60 % of the allowable longitudinal stress, the following relationship should be satisfied:

$$\frac{F_{Rc}}{p_{cw}} + \frac{F_{bc}}{p_{bc}} + \frac{F_c}{p_c} \leq 1.6$$

where

F_{bc} is the maximum applied longitudinal compressive bending stress in the web (in N/mm^2);

F_c is the maximum applied longitudinal compressive axial stress in the web (in N/mm^2);

p_{bc} is the permissible compressive stress due to bending (in N/mm^2);

p_{cw} is as defined in item a);

p_c is the permissible axial stress for struts (in N/mm^2);

F_{Rc} is given (in N/mm^2) by the expression:

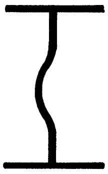
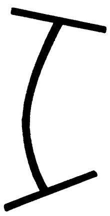
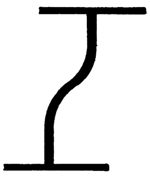
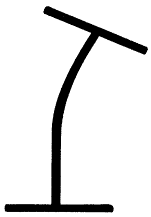
$$F_{Rc} = \frac{R}{(b + n_1)t_w}$$

where

R is the applied reaction or concentrated load and b, n_1, and t_w are defined as in item a).

Where the load or reaction can act eccentrically to the plane of the web, account should be taken of the induced bending stresses.

Table K.1 — Effective lengths and slenderness ratios of an unstiffened web acting as a column

Diagrammatic representation of deformation	Restraint conditions	Effective length, l	Slenderness ratio, l/r
	Web ends restrained against both rotation and relative lateral movement	$0.7D$	$2.4D/t_w$
	Web ends restrained against relative lateral movement but not against rotation	$1.0D$ (but see Note 2)	$3.5D/t_w$
	Web ends restrained against rotation but not against relative lateral movement	$1.2D$	$4.2D/t_w$
	One web end not restrained against rotation nor against relative lateral movement and the other web end restrained against both rotation and relative lateral movement	$2.0D$ (but see Note 2)	$7.0D/t_W$

NOTE 1

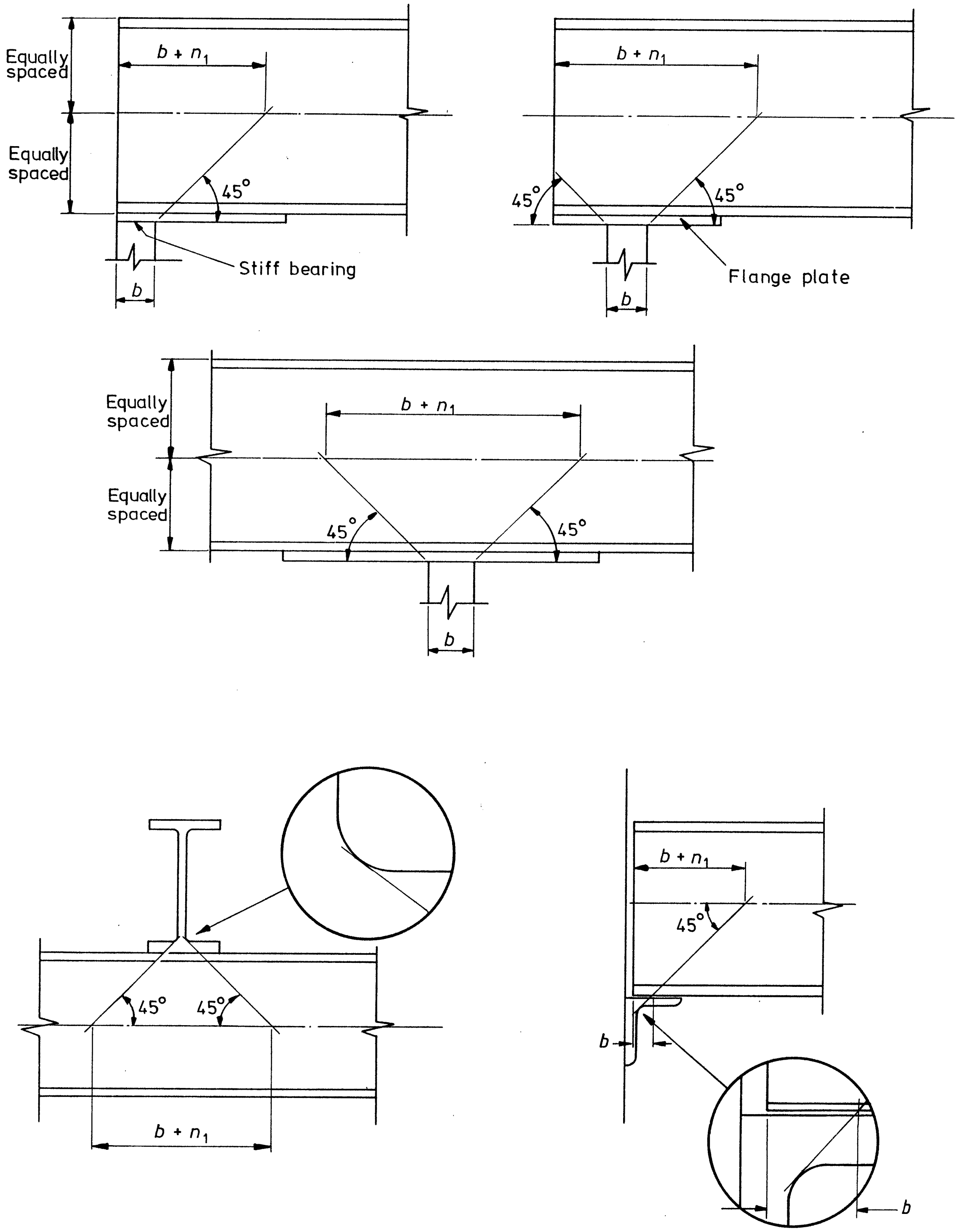
l the effective length (in mm);

D is the overall depth of the section (in mm);

r is the radius of gyration (in mm);

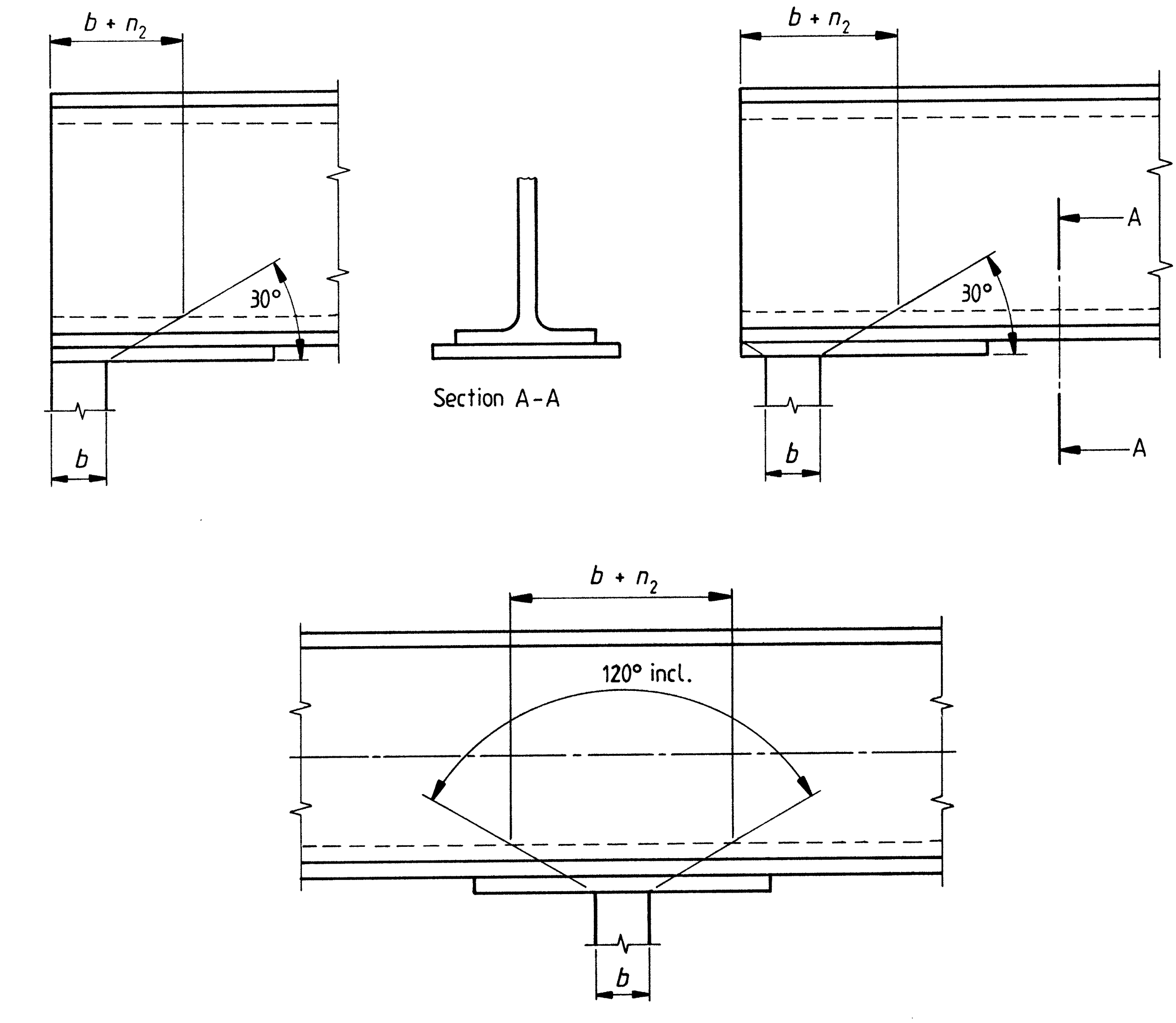
t_w is the web thickness (in mm).

NOTE 2 Where the ends are not restrained against rotation, l/r should be based on the distance between the effective centres of rotation, which may necessitate taking effective lengths greater than $1.0D$ or $2.0D$.



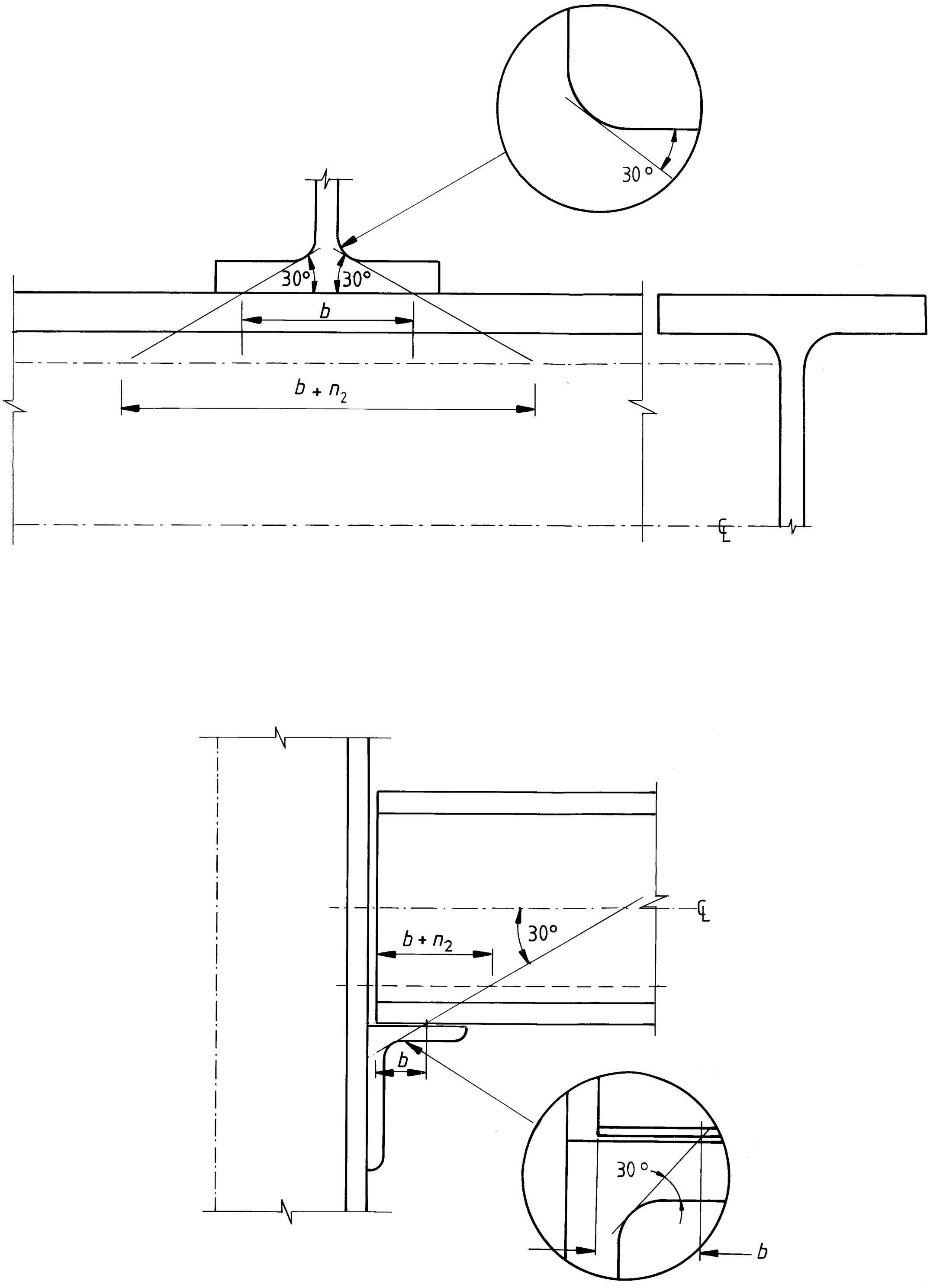
NOTE The dimension n_1 is the additional length obtained by dispersion at 45° in both directions (if available).

Figure K.1 — Stress dispersion: buckling



NOTE The dimension n_2 is the additional length obtained by dispersion at 30° in both directions (if available).

Figure K.2 — Stress dispersion: bearing



NOTE The dimension n_2 is the additional length obtained by dispersion at 30° in both directions (if available).

Figure K.2 — Stress dispersion: bearing (*concluded*)

K.3 Design of bearing stiffeners

K.3.1 Load-bearing stiffeners should, wherever possible, be symmetrical about the web. Where the concentrated load causes compression in the load-bearing stiffeners, they should be designed as struts, and for design purposes the bearing section should be assumed to consist of the pair of stiffeners and a length of web on each side of the centreline of the stiffeners equal, where possible, to twenty times the web thickness. The radius of gyration should be taken about the axis parallel to the web of the beam or girder, and the working stress should be in accordance with the appropriate allowable value for a strut given in Table A.2.

K.3.2 The effective length of the equivalent strut, l, should be taken as given in Table K.2.

K.3.3 The outstanding legs of each pair of load-bearing stiffeners should be so proportioned that the permissible bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds is not exceeded. Unless its outer edge is continuously stiffened, the outstand of any web stiffener from the face of the web should not be more than $14t_s$, where t_s is the thickness of the web stiffener.

K.3.4 Load-bearing stiffeners should be fitted to provide a tight and uniform bearing upon the flange transmitting the load or reaction unless welds or friction grip fasteners (used in accordance with BS 4604, or similar) are provided between the flange and stiffener for this purpose. At points of support, this recommendation should apply at both flanges. Stiffeners that resist loads or reaction applied through a flange should have sufficient welds or fasteners to transmit the design force.

K.3.5 Where the load or reaction can act eccentrically only to the web, or where the centroid of the stiffener does not lie on the centreline of the web, the resulting eccentricity of loading should be allowed for in design.

K.4 Hollow sections

Where concentrated loads or reactions are applied to a hollow section, consideration should be given to the local stresses and the member reinforced where necessary to maintain the required capacity.

Table K.2 — Effective lengths of load-bearings

Restraint conditions	Effective length, l
Loaded flange restrained against both lateral movement relative to other flange, and against rotation in the plane of the stiffener	0.7 L
Loaded flange restrained against lateral movement relative to other flange, but not restrained against rotation in the plane of the stiffener	1.0 L
Loaded flange restrained against rotation in the plane of the stiffener, but not restrained against lateral movement relative to other flange	1.2 L
Loaded flange not restrained against rotation in the plane of the stiffener nor against lateral movement relative to the other flange	2.0 L
NOTE L is the length of the stiffener.	

Annex L (informative)
Effective lengths of steel members in compression

L.1 General

The rotational or positional restraints of members in compression are dependent upon adjacent members but the full theoretical restraint may not be achieved. The values given in this annex are a practical compromise based on experience of conventional steelwork construction. For prefabricated systems, appropriate values should be established by testing.

L.2 Effective length for axial compression

The effective length, l, of a compression member should be derived from its actual centre-to-centre, between the intersections with supporting members, L, in accordance with the conditions of restraint in the appropriate plane.

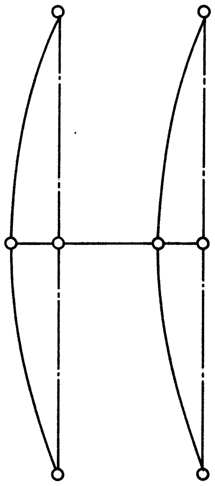
In assessing rotational and positional restraint conditions, connecting members that are more than 90 % fully stressed under simultaneous loading conditions should be considered as providing positional restraint only.

Parallel struts that can buckle simultaneously in the same plane will not be restrained by simple intermediate connecting members (see Figure L.1a)). Effective restraint can be provided by two alternative means (see Figure L.1b)):

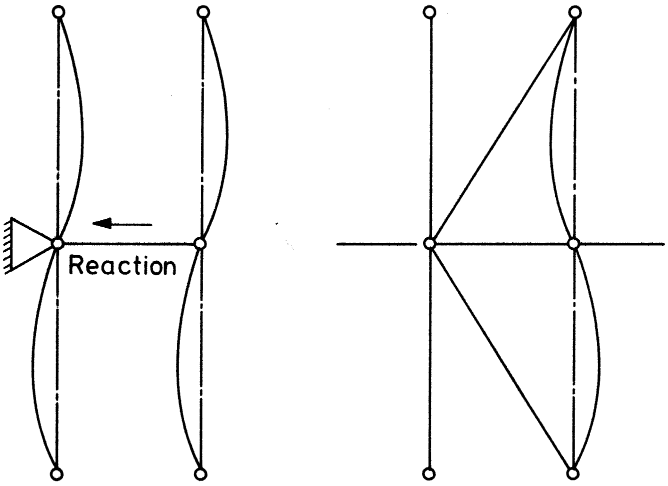
a) by providing external reactions to the buckling forces;

b) by fully triangulating the system in the plane of buckling, thus providing a self-reacting system.

For the purposes of calculating *l*/*r* for struts, the effective length, *l*, given in Table L.1 (which is based on data given in BS 449- 2) should be used. Dimension *L* is the length of strut centre-to-centre between the intersections of members providing lateral support (in mm) and *r* is the radius of gyration (in mm).



a) Ineffective rotational and positional restraint

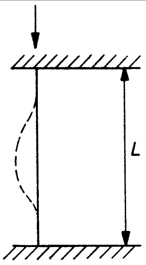
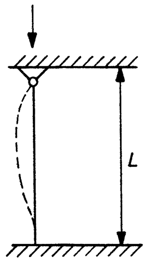
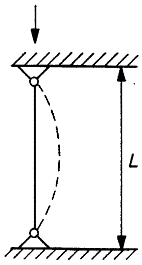
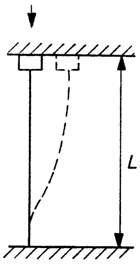
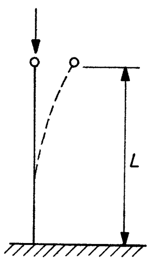


b) Effective positional restraint

NOTE Original position of members shown dashed.

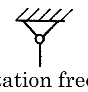
Figure L.1 — Positional restraint of steel members in axial compression

Table L.1 — Effective lengths of struts

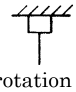
Diagrammatic representation of deformation (see Note)	Restraint conditions	Effective length, l mm
	Effectively held in position and restrained in direction at both ends	0.7 L[a]
	Effectively held in position at both ends and restrained in direction at one end	0.85 L[a]
	Effectively held in position at both ends but not restrained in direction	1.0 L
	Effectively held in position and restrained in direction at one end and partially restrained in direction but not held in position at the other end	1.5 L
	Effectively held in position and restrained in direction at one end but not held in position or restrained at the other end	2.0 L

NOTE Key to end condition code:

rotation fixed and position fixed;



rotation free and position fixed;



rotation fixed and position free;

rotation free and position free.

[a] Not normally appropriate for use in tube and coupler structures. See **6.7.2**.

L.3 Effective length for simply supported or continuous beams

L.3.1 *Beams with intermediate lateral restraints*

Where the support sections of a beam are effectively restrained against torsion as recommended in **L.5** and the compression flange has effective lateral restraints as recommended in **L.6.1** at intervals along its length, the effective length, l, should be taken as the distance between the restraints.

L.3.2 *Beams without intermediate lateral restraints*

Where a beam other than a cantilever beam has no intermediate lateral restraint and its support sections are restrained against twisting about its own longitudinal axis by effective torsional restraints as recommended in **L.5**, the effective length, l, should be taken as given in Table L.2.

Where torsional restraint at support sections is given only by positive connection of the bottom flange to the support, the effective length should be taken as $1.2\ L + 2\ D$. Where torsional restraint at support sections is given only by dead bearing of the bottom flange on the support, the effective length, l, should be taken as $1.4\ L + 2\ D$.

L.4 Effective length for cantilever beams

L.4.1 *Cantilevers with intermediate restraints*

Where the support sections are effectively restrained against torsion as recommended in **L.5** and the compression flange has effective lateral restraint as recommended in **L.6** at intervals along its length, the effective length, l, should be taken as the distance between the restraints.

Table L.2 — Effective lengths for beams without intermediate lateral restraint

Restraint condition	Effective length, l
Compression flange fully restrained against rotation in plan at the supports	$0.85\ L$
Compression flange partially restrained against rotation in plan at the supports e.g. securely cleated connections	$1.05\ L$
Compression flange not restrained against rotation in plan at the supports	$1.2\ L$
NOTE 1 Where neither the top flange nor the load is free to move laterally, the values given in Table L.2 may be divided by a factor of 1. 2.	
NOTE 2 L is the span between supports	

L.4.2 *Cantilevers without intermediate restraints*

Where a cantilever of projecting length, L, carries distributed or concentrated loads and has no intermediate lateral restraint, its effective length, l, should be as given in Table L.3.

The effective length of a cantilever subjected to applied moment at the tip may in this particular case be obtained from **L.3.2** provided that the cross-sections at the tip and the support are held against torsion.

L.5 Effective torsional end restraint

The end of a beam may be taken as effectively restrained against torsion if both flanges are fixed in position laterally at the supports.

Torsional end restraint may be provided by:

a) cleats or end plates attaching the web and/or flanges to a column or similar part of a structural frame which is itself fixed in space (connections may be bolted or welded); or

b) load-bearing stiffener(s) acting as a cantilever from the lower flange and extending the full height of the web (see **K.3**), where the moment of inertia of the stiffeners, J (in mm^4), should not be less than:

$$\frac{D^3 T_{max} R}{250 W}$$

where

D is the depth of the beam (in mm);

T_{max} is the maximum thickness of the compression flange (in mm);

R is the reaction on the bearing (in kN);

W is the total load on the girder (in kN);

or

c) lateral end frames or other external supports to the ends of the flanges, for example connecting adjacent top and bottom flanges. Whatever type of torsional end restraint is adopted, the restraint at each end should be designed to resist, in addition to the effects of externally applied loads, a lateral force at the level of the unsupported flange not less than 2.5 % of the maximum force occurring in the flange

Whatever type of torsional end restraint is adopted, the restraint at each end should be designed to resist, in addition to the effects of externally applied loads, a lateral force at the level of the unsupported flange not less than 2.5 % of the maximum force occurring on the flange.

L.6 Effective lateral restraint

NOTE For the purposes of this subclause, the term girder is considered to include beams, lattice girders, plate girders and trusses.

L.6.1 *General*

Any restraint system providing lateral stability to a girder should effectively limit lateral movement of restrained points relative to the supports.

To ensure that the restraint provided is adequate, a compression flange restraint force, H_c, acting normal to the girder, is considered in the design. Restraint to a group of girders can be provided by bracing one or more pairs of girders to provide the flange restraint force. Alternatively, the groups or a single girder can be linked at restraint points to an external support that is sufficiently rigid to resist the flange restraint force.

The magnitude of the compression flange restraint force is as follows:

a) for up to three girders linked together: $H_c = 0.025 \times$ sum of maximum force in the compression flanges;

b) for four or more girders linked together: $H_c = 0.025 \times$ sum of the three largest maximum forces in the compression flanges.

The force H_c should be divided equally between the number of points where the girders are restrained. Where a group of girders are braced internally, the restraint force is considered not to give rise to any resultant reaction on the supports.

Where, however, the restraint is by an external support, a part of the total flange restraint force is resisted by the girder supports. Combinations of the two methods are possible, but require individual consideration.

L.6.2 *Girder restraint bracing design*

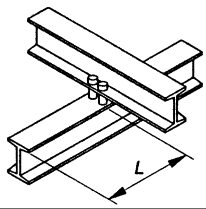
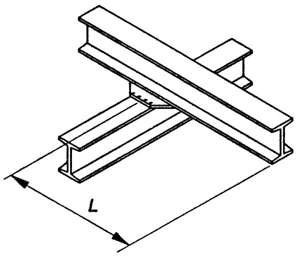
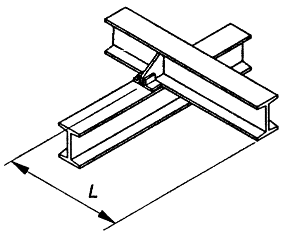
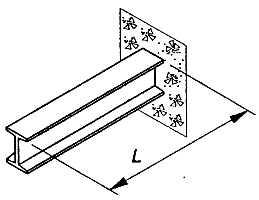
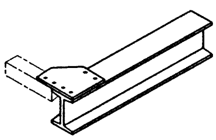
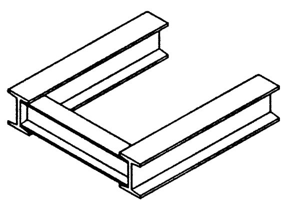
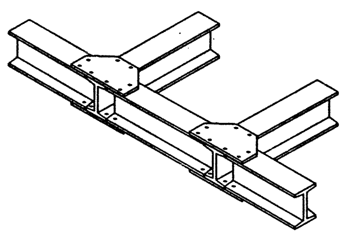
Where two or more girders are effectively connected together at a line of node points with bracing, the bracing should be designed to provide the compression flange restraint force, H_c, distributed as shown in Figure L.2, in addition to any other forces that may be applied. The application of the restraint force is shown in Figure L.2 with the force shown in the form of applied external forces to permit the design of the bracing. This gives a notional end reaction in the linking member AD and WX of H_c/n, where n is the number of node points, but because the bracing system is a closed circuit, this reaction is not transmitted to the supports. The two braced girders AW and DX may provide lateral restraints to other linked girders in the group effectively connected at the node points as shown in Figure L.2.

L.6.3 *Girder restraint from external points*

Where a girder, or group of girders, is linked to adequate external restraint points, the links and their connections should be designed to provide the relevant value of the compression flange restraint force, H_c, distributed as shown in Figure L.3, in addition to any other external forces that may be applied. The application of the restraint forces is shown in Figure L.3.

The flange force, H_c/n, should be restrained by the girder supports. The girder supports at each will have to provide this restraint force as an applied external force in addition to other applied and stability forces at the supports.

Table L.3 — Effective length for cantilever beams without intermediate lateral restraint

Restraint conditions		Loading conditions	
At support	**At tip**	**Normal**	**Destablizing**[a]
Continuous with lateral restraint only 	Free Laterally restrained on top flange only Torsionally restrained only Laterally and torsionally restrained	3.0 L 2.7 L 2.4 L 2.1 L	7.5 L 7.5 L 2.4 L 3.6 L
Continuous with partial torsional restraint 	Free Laterally restrained on top flange only Torsionally restrained only Laterally and torsionally restrained	2.0 L 1.8 L 1.6 L 1.4 L	5.0 L 5.0 L 3.0 L 2.4 L
Continuous with lateral and torsional restraint 	Free Laterally restrained on top flange only Torsionally restrained only Laterally and torsionally restrained	1.0 L 0.9 L 0.8 L 0.7 L	2.5 L 2.5 L 1.5 L 1.2 L
Built-in laterally and torsionally 	Free Laterally restrained on top flange only Torsionally restrained only Laterally and torsionally restrained	0.8 L 0.7 L 0.6 L 0.5 L	1.4 L 1.4 L 0.6 L 0.5 L
Braced laterally in at least one bay  Top flange restraint	 Torsional restraint	Braced laterally in at least one bay  Lateral and torsional restraint	

NOTE L is the projecting length of the cantilever.

[a] The destabilizing loading condition exists when the load is applied on the top flange, and both the load and the flange are free to move laterally

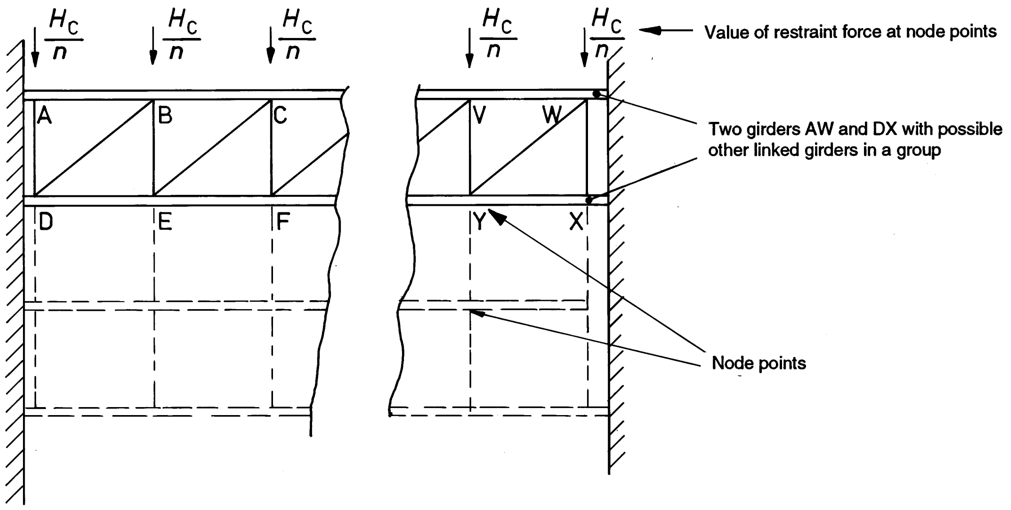


Figure L.2 — Girder restraint (1)

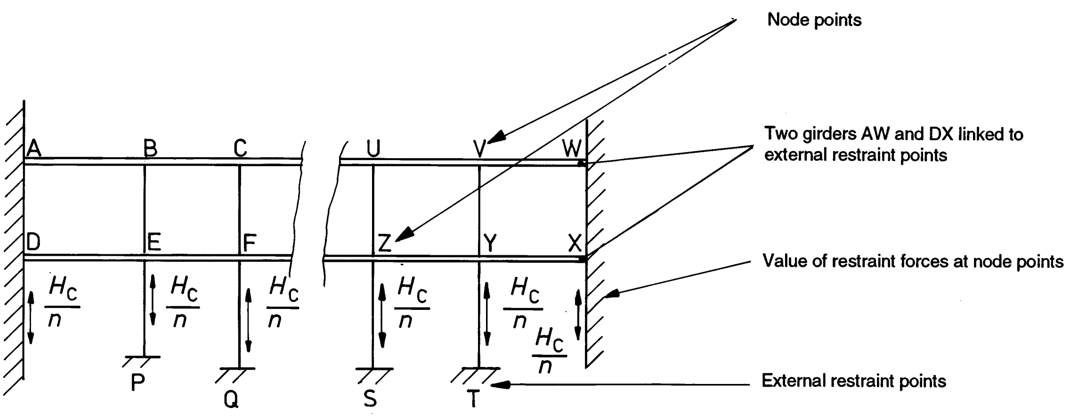


Figure L.3 — Girder restraint (2)

Annex M (informative) Selection of propping and repropping procedures for multi-storey buildings

M.1 General

In multi-storey reinforced concrete construction, it is usually necessary for several slabs to contribute to the support of the last one to be cast. The supporting levels will generally not have reached their twenty-eight day design concrete strengths, and will be of differing ages (and therefore stiffness). Furthermore, the deflections of maturing reinforced concrete members under a sustained load at an early age are not easy to predict.

With the tendency towards both faster construction cycles and the use of larger spans (in which the design live load is decreased relative to the dead load), the importance of the selection and control of propping procedures increases.

In the case of post-tensioned prestressed concrete construction, the tensioning operations will affect load distribution, and special consideration needs to be given to the propping procedure.

In simple cases where a single floor does not have sufficient strength to support a further slab of fresh concrete and progress is not very rapid, stripping and repropping may be undertaken so that two floors support the new floor. Table M.1 shows a method of analysing this approach. For cases where props are left undisturbed Table M.2 gives an analysis. Table M.3 is appropriate when a further level of repropping is in use. Where it is possible to strip props one at a time and replace them without allowing any deflection of the slab above to occur, they may be considered as undisturbed (see Table M.2 and Table M.3). These analyses are based on the following simplified assumptions:

a) creep and shrinkage of the concrete can be disregarded;

b) the footings beneath the props to the first suspended level are taken as rigid;

c) all props are rigid (i.e all levels connected by props are subject to the same deflections);

d) all suspended slabs are equally stiff.

Some confirmation of figures calculated in this way has been obtained from site measurements, but both assumptions b) and c) tend to underestimate the load carried by upper slabs.

M.2 Procedures

Three possible site procedures, with their structural implications, can be identified as follows.

a) *Removal of props or other supports*

1) where slabs are spanning between beams, the slabs both sides of a beam should be released before props under the beam are disturbed;

2) the removal of support to any single slab or beam should be undertaken in the following two stages, to avoid the risk of overloading any props:

i) ease all props by about the same amount (e.g. one or two turns on an adjustable prop);

ii) starting at mid span (or cantilever tip) and working towards the supports, remove the props.

b) R*epropping*. After all the props and formwork at one level have been removed, an appropriate number of props should be reintroduced and adjusted to give a solid contact with the concrete above and below; they should not, however, be deliberately extended enough to put a significant load into them.

Provided the props are carefully erected, and the end plates are in direct contact with the slabs, it may be possible for them to carry higher loads than those given in **3.9.6**. However, it is recommended that the loads imposed on props complying with the requirements of BS 4074 should not exceed 35 kN.

The load reduction due to the removal of formwork is normally disregarded in propping calculations.

Repropping should, where possible, be in alignment with the props above. Where a prop at the level above is not directly over a reprop, the effect on the slab should be considered.

c) "*Quick-strip method*". This method may be offered as an optional facility with proprietary support equipment

With this method, the structural props need to remain undisturbed (and hence loaded) while the forms and their immediate supports are removed.

It is necessary to cheek that the slab can span between the structural props before the formwork is removed.

M.3 Examples

Three examples of the calculation of propping loads are given in Table M.1, Table M.2 and Table M.3. In each case, the load *W* represents the weight of the reinforced concrete plus the formwork and falsework, which is sometimes taken as 10 % of the concrete weight.

An additional allowance may be made for any known imposed loads (see **4.4**) but for simplicity, the calculation of these is not shown in these examples.

Two useful arithmetic checks that can be made on calculations in these and similar examples are:

a) the total slab loads (in the right hand columns) at each stage should equal the total weight of slabs being considered;

b) at each slab level, the prop load acting from above plus slab weight should equal the load transmitted by the slab to the support plus the prop support load from beneath.

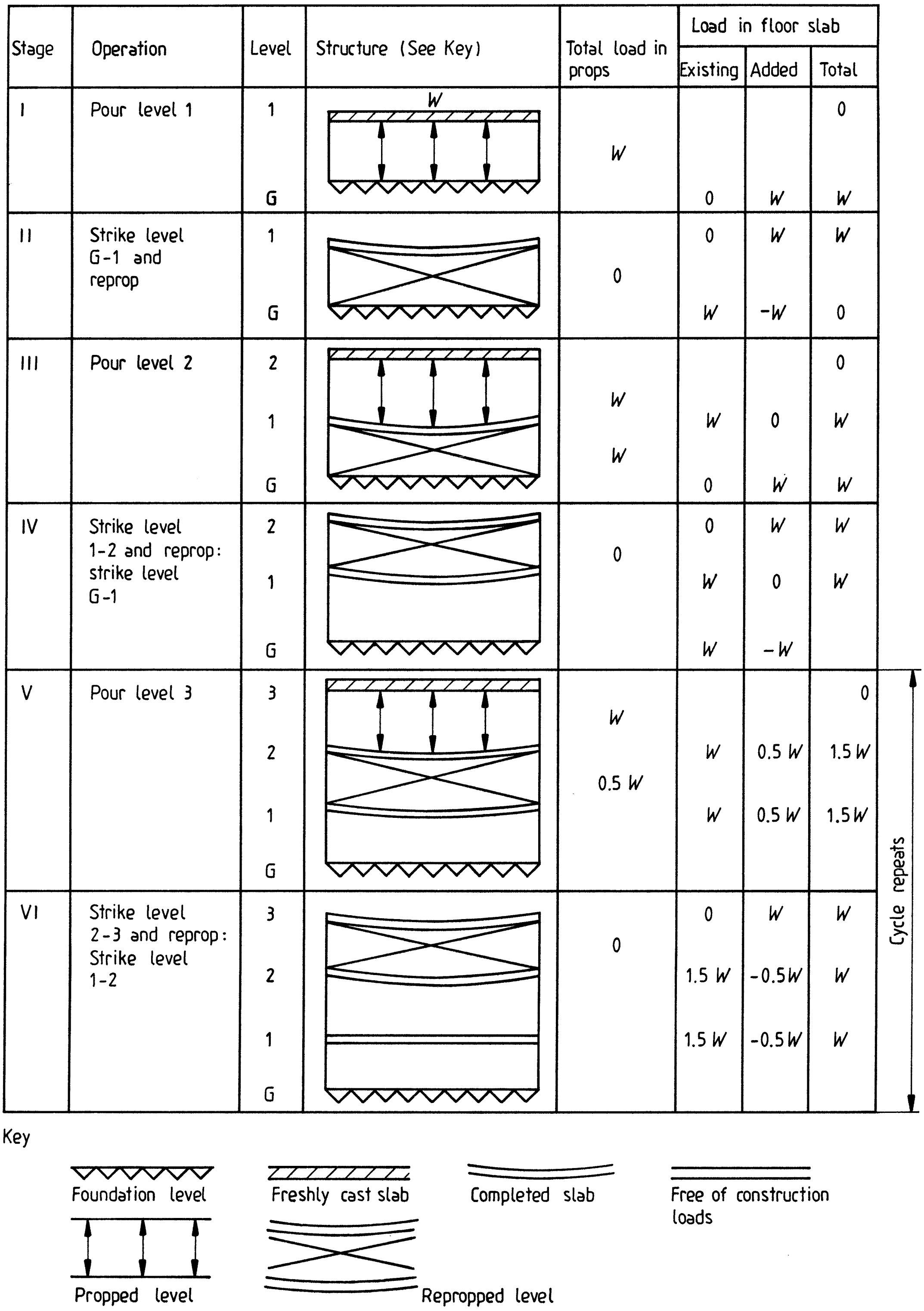
It is important that the first lift of props on a non-suspended slab is not left in too long, as the total weight of all the slabs constructed will act on them. Once they are removed, slabs can deflect and hence contribute to the support of loads.

Where a particular system of propping and repropping is considered for the construction of a large number of similar slabs, the loads on the slabs converge on specific values, and some general conclusions may be drawn.

1) The most heavily loaded slab is the one under the lowest set of props (not reprops) and its load will not exceed 2.5 *W*.

2) The effect of increasing the number of propped levels is to allow a slab more time before it carries its maximum construction load, but not to modify that load significantly.

Table M.1 — Two Lifts of popping with repropping



Stage	Operation	Level	Structure (See Key)	Total load in props	Load in floor slab: Existing	Added	Total
I	Pour level 1	1		W			0
		G			0	W	W
II	Strike level G-1 and reprop	1		0	0	W	W
		G			W	$-W$	0
III	Pour level 2	2		W			0
		1		W	W	0	W
		G			0	W	W
IV	Strike level 1-2 and reprop: strike level G-1	2		0	0	W	W
		1			W	0	W
		G			W	$-W$	
V	Pour level 3	3		W			0
		2		0.5 W	W	0.5 W	1.5 W
		1			W	0.5 W	1.5 W
		G					
VI	Strike level 2-3 and reprop: Strike level 1-2	3		0	0	W	W
		2			1.5 W	-0.5 W	W
		1			1.5 W	-0.5 W	W
		G					

Cycle repeats (stages V–VI)

Key

Foundation level

Freshly cast slab

Completed slab

Free of construction loads

Propped level

Repropped level

Table M.2 — Two Lifts of popping with repropping (*continued*)

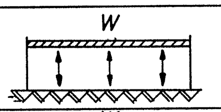
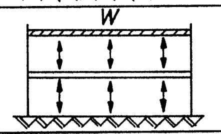
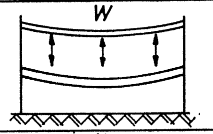
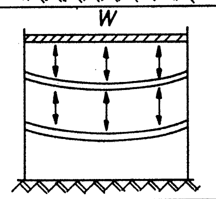
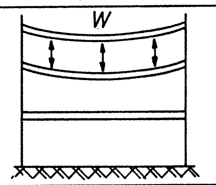
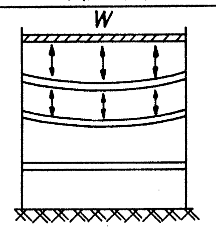
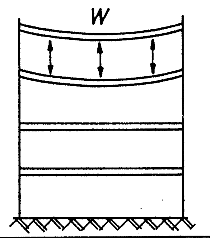
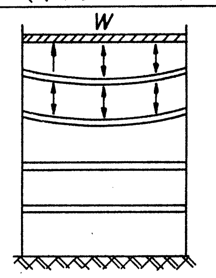
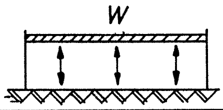
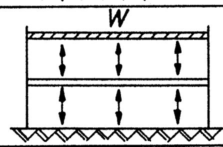
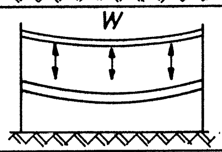
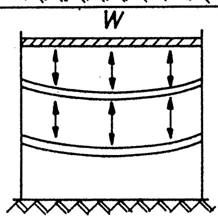
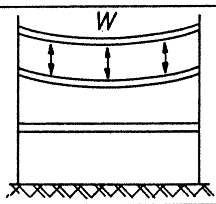
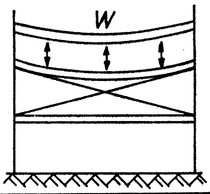
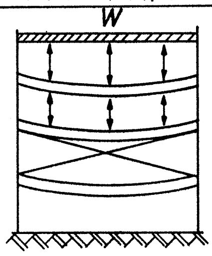
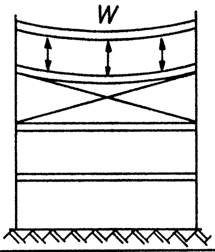
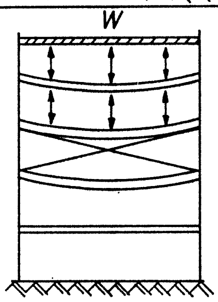
Stage	Operation	Level	Structure (see key)	Total load in props	Load in floor slab Existing	Load in floor slab Added	Load in floor slab Total
I	Pour level 1	1 G		W	0	W	W
II	Pour level 2	2 1 G		W $2W$	0 W	0 W	0 W
III	Props G-1 removed	2 1 G		0	0 0	W W	W W
IV	Pour level 3	3 2 1 G		W $0.5W$	W W	$0.5W$ $0.5W$	$1.5W$ $1.5W$
V	Props 1-2 removed	3 2 1 G		$0.75W$	0 $1.5W$	$0.25W$ $0.25W$	$0.25W$ $1.75W$ W
VI	Pour level 4	4 3 2 1 G		W $1.25W$	$0.25W$ $1.75W$	$0.5W$ $0.5W$	$0.75W$ $2.25W$ W
VII	Props 2-3 removed	4 3 2 1 G		$0.38W$	0 $0.75W$	$0.62W$ $0.63W$	$0.62W$ $1.38W$ W W
VIII	Pour level 5	5 4 3 2 1 G		$0.88W$	$0.62W$ $1.38W$	$0.5W$ $0.5W$	$1.12W$ $1.88W$ W W

Table M.3 — Two Lifts of popping with repropping

Stage	Operation	Level	Structure (see key)	Total load in props	Load in floor slab Existing	Load in floor slab Added	Load in floor slab Total
I	Pour level 1	1		W			
		G			0	W	W
II	Pour level 2	2					
		1		W	0	0	0
		G		2W	W	W	W
III		2		0	0	W	W
		1			0	W	W
	Props G-1 removed	G					
IV	Pour level 3	3		W			
		2			W	0.5W	1.5W
		1		0.5W	W	0.5W	1.5W
		G					
V		3		0.75W	0	0.25W	0.25W
		2			1.5W	0.25W	1.75W
	Props 1-2 removed	1					W
		G					

NOTE To this stage, propping procedure repeats stages I to V Table M.2.

Table M.3 — Two Lifts of popping with repropping (*concluded*)

Stage	Operation	Level	Structure (see key)	Total load in props	Load in floor slab Existing	Added	Total
VI	Repropped 1-2	3 2 1 G		0.75W 0	0.25W 1.75W W	0 0 0	0.25W 1.75W W
VII	Pour level 4	4 3 2 1		W 1.42W 0.33W	0.25W 1.75W W	0.33W 0.34W 0.33W	0.58W 2.09W 1.33W
VIII	Props 2-3 removed and repropped. Props 1-2 removed	4 3 2 1 G		0.29 W 0	0 0.58W W	0.71W 0.71W 0	0.71W 1.29W W W
IX	Pour level 5	5 4 3 2 1 G		0.96 W 0.33	0.71W 1.29W W	0.33 W 0.34W 0.33W	1.04W 1.63W 1.33W W

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[9)] This is one of seven standards which have replaced BS 4360:1990, the successor standard to BS 968.

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[10)] This is one of seven standards which have replaced BS 4360:1990, the successor standard to BS 968.

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