This edition of AS 5100.5 has been prepared as an RMS interim concrete bridge design code to incorporate some of the necessary changes in AS 3600-2009 into AS 5100.5-2004, together with the contents of RMS Bridge Technical Directions BTD 2007/10 and BTD 2007/11 and the cement composition requirements of RMS B80.

The original Australian Standard® AS 5100.5-2004 was prepared by Standards Australia Committee BD-090 and was approved on behalf of the Council of Standards Australia on 9 December 2003. The original AS 5100.5 Australian Standard® was published on 23 April 2004.

Rev 2 – May 2015
**PREFACE**

This interim edition of AS 5100.5 has been prepared by RMS that incorporates some of the necessary changes in AS 3600-2009 into AS 5100.5-2004, the contents of RMS Bridge Technical Directions BTD 2007/10 and BTD 2007/11 and the cement composition requirements of RMS B80.

The changes are indicated in the text by a vertical bar in the margin, red font and the word “Interim” at the changed clause.

The original Standard was prepared by the Standards Australia Committee BD-090, Bridge Design, to supersede HB 77.5—1996, *Australian Bridge Design Code*, Section 5: Concrete.

This Standard also incorporates Amendment No. 1 (April 2010) and Amendment No. 2 (December 2010). The changes required by the Amendment are indicated in the text by a marginal bar and amendment number against the clause, note, table, figure or part thereof affected.

The words ‘shall’ and ‘may’ are used consistently throughout this Standard to indicate respectively, a mandatory provision and an acceptable or permissible alternative.

Statements expressed in mandatory terms in Notes to tables are deemed to be requirements of this Standard.

The terms ‘normative’ and ‘informative’ have been used in this Standard to define the application of the appendix to which they apply. A ‘normative’ appendix is an integral part of a Standard, whereas an ‘informative’ appendix is only for information and guidance.

The major differences between this interim edition and AS 5100.5-2004 are as follows:

<table>
<thead>
<tr>
<th>No</th>
<th>Major Changes Included in the Interim Edition</th>
<th>Clause/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Changes in design properties of materials</td>
<td>Section 6</td>
</tr>
<tr>
<td>2</td>
<td>Inclusion of a new clause regarding linear elastic stress (e.g., finite element) analysis and updates for strut-and-tie analysis.</td>
<td>Section 7, Clause 12.1 and Appendix I</td>
</tr>
<tr>
<td>3</td>
<td>Updates for shear and torsion in beams, including:</td>
<td>Clause 8.2.7.1, 8.2.8, 8.2.9, 8.2.12.4, 8.2.7.4, 8.2.2c, 8.3, 8.4</td>
</tr>
<tr>
<td></td>
<td>• shear strength excluding shear reinforcement</td>
<td>Clause 8.2.7.1</td>
</tr>
<tr>
<td></td>
<td>• minimum shear reinforcement</td>
<td>Clause 8.2.8</td>
</tr>
<tr>
<td></td>
<td>• shear strength with minimum shear reinforcement</td>
<td>Clause 8.2.9</td>
</tr>
<tr>
<td></td>
<td>• anchorage of shear reinforcement</td>
<td>Clause 8.2.12.4</td>
</tr>
<tr>
<td></td>
<td>• interaction between shear and torsion</td>
<td>Clauses 8.2.7.4 and 8.2.2c</td>
</tr>
<tr>
<td></td>
<td>• strength of beams in torsion</td>
<td>Clause 8.3</td>
</tr>
<tr>
<td></td>
<td>• longitudinal design in beams</td>
<td>Clause 8.4</td>
</tr>
<tr>
<td>4</td>
<td>Update of expression for short-term deflection of beams and slabs using simplified calculation</td>
<td>Clauses 8.5 and 9.3</td>
</tr>
<tr>
<td>5</td>
<td>Updates for crack control of beams, slabs and non-flexural members</td>
<td>Clauses 8.6.1, 9.4.1a and 12.4</td>
</tr>
<tr>
<td>6</td>
<td>Introduction of provisions for detailing of highly loaded columns with $f_c &gt; 50$ MPa</td>
<td>Clause 10.7.3.1</td>
</tr>
<tr>
<td>7</td>
<td>Updates for stress development and splicing of reinforcement and tendons</td>
<td>Section 13</td>
</tr>
<tr>
<td>8</td>
<td>Update for minimum reinforcement in slabs</td>
<td>Clause 9.1.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ID</strong> Figure 6.3.4 Revised</td>
<td><strong>ID</strong> Equation 8.5.3.1(1)</td>
</tr>
<tr>
<td>Equation 8.3.3(1) Symbol description amended</td>
<td>Definition of $I_{cr}$ and $I_{c,max}$ corrected</td>
</tr>
</tbody>
</table>
## CONTENTS

<table>
<thead>
<tr>
<th>SECTION/CLAUSE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>PREFACE</td>
<td>2</td>
</tr>
<tr>
<td>CONTENTS</td>
<td>3</td>
</tr>
<tr>
<td><strong>SECTION 1 SCOPE AND GENERAL</strong></td>
<td></td>
</tr>
<tr>
<td>1.1 SCOPE AND APPLICATION</td>
<td>7</td>
</tr>
<tr>
<td>1.2 REFERENCED DOCUMENTS</td>
<td>7</td>
</tr>
<tr>
<td>1.3 DEFINITIONS</td>
<td>7</td>
</tr>
<tr>
<td>1.4 NOTATION</td>
<td>12</td>
</tr>
<tr>
<td>1.5 USE OF ALTERNATIVE MATERIALS OR METHODS</td>
<td>22</td>
</tr>
<tr>
<td>1.6 DESIGN</td>
<td>22</td>
</tr>
<tr>
<td>1.7 MATERIALS AND CONSTRUCTION REQUIREMENT</td>
<td>23</td>
</tr>
<tr>
<td><strong>SECTION 2 DESIGN REQUIREMENTS AND PROCEDURES</strong></td>
<td></td>
</tr>
<tr>
<td>2.1 DESIGN REQUIREMENTS</td>
<td>24</td>
</tr>
<tr>
<td>2.2 STRENGTH</td>
<td>24</td>
</tr>
<tr>
<td>2.3 DURABILITY</td>
<td>24</td>
</tr>
<tr>
<td>2.4 FIRE RESISTANCE</td>
<td>27</td>
</tr>
<tr>
<td>2.5 FATIGUE</td>
<td>27</td>
</tr>
<tr>
<td>2.6 DESIGN FOR STABILITY</td>
<td>28</td>
</tr>
<tr>
<td>2.7 DEFLECTIONS OF BEAMS AND SLABS</td>
<td>28</td>
</tr>
<tr>
<td>2.8 CRACKING</td>
<td>28</td>
</tr>
<tr>
<td>2.9 VIBRATION</td>
<td>29</td>
</tr>
<tr>
<td>2.10 DESIGN FOR STRENGTH AND SERVICEABILITY BY PROTOTYPE TESTING</td>
<td>29</td>
</tr>
<tr>
<td>2.11 OTHER DESIGN REQUIREMENTS</td>
<td>29</td>
</tr>
<tr>
<td><strong>SECTION 3 LOADS AND LOAD COMBINATIONS FOR STABILITY, STRENGTH AND SERVICEABILITY</strong></td>
<td></td>
</tr>
<tr>
<td>3.1 LOADS AND OTHER ACTIONS</td>
<td>30</td>
</tr>
<tr>
<td>3.2 LOAD COMBINATIONS</td>
<td>30</td>
</tr>
<tr>
<td><strong>SECTION 4 DESIGN FOR DURABILITY</strong></td>
<td></td>
</tr>
<tr>
<td>4.1 APPLICATION</td>
<td>31</td>
</tr>
<tr>
<td>4.2 DESIGN FOR DURABILITY</td>
<td>31</td>
</tr>
<tr>
<td>4.3 EXPOSURE CLASSIFICATION</td>
<td>31</td>
</tr>
<tr>
<td>4.4 MEMBERS NOT CONTAINING MATERIAL REQUIRING PROTECTION</td>
<td>33</td>
</tr>
<tr>
<td>4.5 EXPOSURE CLASSIFICATIONS A, B1, B2 AND C</td>
<td>33</td>
</tr>
<tr>
<td>4.6 EXPOSURE CLASSIFICATION U</td>
<td>34</td>
</tr>
<tr>
<td>4.7 ABRASION</td>
<td>34</td>
</tr>
<tr>
<td>4.8 FREEZING AND THAWING</td>
<td>34</td>
</tr>
<tr>
<td>4.9 CHEMICAL CONTENT IN CONCRETE</td>
<td>35</td>
</tr>
<tr>
<td>4.10 COVER TO REINFORCING STEEL AND TENDONS</td>
<td>35</td>
</tr>
<tr>
<td>4.11 PROVISIONS FOR STRAY CURRENT CORROSION</td>
<td>38</td>
</tr>
<tr>
<td><strong>SECTION 5 DESIGN FOR FIRE RESISTANCE</strong></td>
<td>39</td>
</tr>
</tbody>
</table>
SECTION/CLAUSE | PAGE
---|---
SECTION 6 DESIGN PROPERTIES OF MATERIALS
6.1 PROPERTIES OF CONCRETE | 40
6.2 PROPERTIES OF REINFORCEMENT | 46
6.3 PROPERTIES OF TENDONS | 47
6.4 LOSS OF PRESTRESS IN TENDON | 49
SECTION 7 METHODS OF STRUCTURAL ANALYSIS
7.1 GENERAL | 53
7.2 LINEAR ELASTIC ANALYSIS | 54
7.3 ELASTIC ANALYSIS OF FRAMES INCORPORATING SECONDARY BENDING MOMENTS | 56
7.4 LINEAR ELASTIC STRESS (e.g., FINITE ELEMENT) ANALYSIS | 56
7.5 ANALYSIS USING STRUT-AND-TIE MODELS | 57
7.6 RIGOROUS STRUCTURAL ANALYSIS | 57
7.7 PLASTIC METHODS OF ANALYSIS OF SLABS | 57
7.8 PLASTIC METHODS OF ANALYSIS OF FRAMES | 58
7.9 SEISMIC ANALYSIS METHODS | 58
SECTION 8 DESIGN OF BEAMS FOR STRENGTH AND SERVICEABILITY
8.1 STRENGTH OF BEAMS IN BENDING | 59
8.2 STRENGTH OF BEAMS IN SHEAR | 66
8.3 STRENGTH OF BEAMS IN TORSION | 70
8.4 LONGITUDINAL SHEAR IN BEAMS | 73
8.5 DEFLECTION OF BEAMS | 75
8.6 CRACK CONTROL OF BEAMS | 77
8.7 VIBRATION OF BEAMS | 79
8.8 PROPERTIES OF BEAMS | 79
8.9 SLENDERNESS LIMITS FOR BEAMS | 79
SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY
9.1 STRENGTH OF SLABS IN BENDING | 81
9.2 STRENGTH OF SLABS IN SHEAR | 82
9.3 DEFLECTION OF SLABS | 83
9.4 CRACK CONTROL OF SLABS | 84
9.5 VIBRATION OF SLABS | 86
9.6 MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS | 86
9.7 LONGITUDINAL SHEAR IN SLABS | 87
9.8 FATIGUE OF SLABS | 87
SECTION 10 DESIGN OF COLUMNS AND TENSION MEMBERS FOR STRENGTH AND SERVICEABILITY
10.1 GENERAL | 88
10.2 DESIGN PROCEDURES | 88
10.3 DESIGN OF SHORT COLUMNS | 89
10.4 DESIGN OF SLENDER COLUMNS | 89
10.5 SLENDERNESS | 91
10.6 STRENGTH OF COLUMNS IN COMBINED BENDING AND COMPRESSION | 94
10.7 REINFORCEMENT FOR COLUMNS | 95
10.8 DESIGN OF TENSION MEMBERS | 100
<table>
<thead>
<tr>
<th>SECTION/CLAUSE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>SECTION 11 DESIGN OF WALLS</td>
<td></td>
</tr>
<tr>
<td>11.1 APPLICATION</td>
<td>101</td>
</tr>
<tr>
<td>11.2 DESIGN PROCEDURES</td>
<td>101</td>
</tr>
<tr>
<td>11.3 BRACING OF WALLS</td>
<td>101</td>
</tr>
<tr>
<td>11.4 SIMPLIFIED DESIGN METHOD FOR BRACED WALLS SUBJECT TO VERTICAL IN-PLANE LOADS ONLY</td>
<td>102</td>
</tr>
<tr>
<td>11.5 DESIGN OF WALLS FOR IN-PLANE HORIZONTAL FORCES</td>
<td>102</td>
</tr>
<tr>
<td>11.6 REINFORCEMENT FOR WALLS</td>
<td>102</td>
</tr>
<tr>
<td>SECTION 12 DESIGN OF NON-FLEXURAL MEMBERS, END ZONES AND BEARING SURFACES</td>
<td></td>
</tr>
<tr>
<td>12.1 DESIGN OF NON-FLEXURAL MEMBERS</td>
<td>103</td>
</tr>
<tr>
<td>12.2 PRESTRESSING ANCHORAGE ZONES</td>
<td>105</td>
</tr>
<tr>
<td>12.3 BEARING SURFACES</td>
<td>108</td>
</tr>
<tr>
<td>12.4 CRACK CONTROL</td>
<td>109</td>
</tr>
<tr>
<td>SECTION 13 STRESS DEVELOPMENT AND SPLICING OF REINFORCEMENT AND TENDONS</td>
<td></td>
</tr>
<tr>
<td>13.1 STRESS DEVELOPMENT IN REINFORCEMENT</td>
<td>110</td>
</tr>
<tr>
<td>13.2 SPLICING OF REINFORCEMENT</td>
<td>116</td>
</tr>
<tr>
<td>13.3 STRESS DEVELOPMENT IN TENDONS</td>
<td>118</td>
</tr>
<tr>
<td>13.4 COUPLING OF TENDONS</td>
<td>119</td>
</tr>
<tr>
<td>SECTION 14 JOINTS, EMBEDDED ITEMS, FIXING AND CONNECTIONS</td>
<td></td>
</tr>
<tr>
<td>14.1 DESIGN OF JOINTS</td>
<td>120</td>
</tr>
<tr>
<td>14.2 EMBEDDED ITEMS AND HOLES IN CONCRETE</td>
<td>120</td>
</tr>
<tr>
<td>14.3 REQUIREMENTS FOR FIXINGS</td>
<td>121</td>
</tr>
<tr>
<td>14.4 CONNECTIONS</td>
<td>121</td>
</tr>
<tr>
<td>SECTION 15 PLAIN CONCRETE MEMBERS</td>
<td></td>
</tr>
<tr>
<td>15.1 APPLICATION</td>
<td>122</td>
</tr>
<tr>
<td>15.2 DESIGN</td>
<td>122</td>
</tr>
<tr>
<td>15.3 STRENGTH IN BENDING</td>
<td>122</td>
</tr>
<tr>
<td>15.4 STRENGTH IN SHEAR</td>
<td>122</td>
</tr>
<tr>
<td>15.5 STRENGTH IN AXIAL COMPRESSION</td>
<td>123</td>
</tr>
<tr>
<td>15.6 STRENGTH IN COMBINED BENDING AND COMPRESSION</td>
<td>123</td>
</tr>
<tr>
<td>15.7 REINFORCEMENT AND EMBEDDED ITEMS</td>
<td>123</td>
</tr>
<tr>
<td>SECTION 16 MATERIAL AND CONSTRUCTION REQUIREMENTS</td>
<td></td>
</tr>
<tr>
<td>16.1 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR CONCRETE AND GROUT</td>
<td>124</td>
</tr>
<tr>
<td>16.2 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR REINFORCING STEEL</td>
<td>127</td>
</tr>
<tr>
<td>16.3 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR PRESTRESSING DUCTS, ANCHORAGES AND TENDONS</td>
<td>128</td>
</tr>
<tr>
<td>16.4 CONSTRUCTION REQUIREMENTS FOR JOINTS AND EMBEDDED ITEMS</td>
<td>131</td>
</tr>
<tr>
<td>16.5 TOLERANCES FOR STRUCTURES AND MEMBERS</td>
<td>131</td>
</tr>
<tr>
<td>16.6 FORMWORK</td>
<td>132</td>
</tr>
<tr>
<td>SECTION/CLAUSE</td>
<td>PAGE</td>
</tr>
<tr>
<td>----------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>SECTION 17 TESTING OF MEMBERS AND STRUCTURES</td>
<td></td>
</tr>
<tr>
<td>17.1 GENERAL</td>
<td>133</td>
</tr>
<tr>
<td>17.2 TESTING OF MEMBERS</td>
<td>133</td>
</tr>
<tr>
<td>17.3 PROOF TESTING</td>
<td>134</td>
</tr>
<tr>
<td>17.4 PROTOTYPE TESTING</td>
<td>134</td>
</tr>
<tr>
<td>17.5 QUALITY CONTROL</td>
<td>136</td>
</tr>
<tr>
<td>17.6 TESTING FOR STRENGTH OF HARDENED CONCRETE IN PLACE</td>
<td>136</td>
</tr>
<tr>
<td>APPENDICES</td>
<td></td>
</tr>
<tr>
<td>A REFERENCED DOCUMENTS</td>
<td>138</td>
</tr>
<tr>
<td>B DESIGN OF SEGMENTAL CONCRETE BRIDGES</td>
<td>140</td>
</tr>
<tr>
<td>C BEAM STABILITY DURING ERECTION</td>
<td>143</td>
</tr>
<tr>
<td>D SUSPENSION REINFORCEMENT DESIGN PROCEDURES</td>
<td>145</td>
</tr>
<tr>
<td>E COMPOSITE CONCRETE MEMBERS DESIGN PROCEDURES</td>
<td>151</td>
</tr>
<tr>
<td>F BOX GIRDERS</td>
<td>157</td>
</tr>
<tr>
<td>G END ZONES FOR PRESTRESSING ANCHORAGES</td>
<td>159</td>
</tr>
<tr>
<td>H STANDARD PRECAST PRESTRESSED CONCRETE GIRDER</td>
<td>163</td>
</tr>
<tr>
<td>I STRUT-AND-TIE MODELLING</td>
<td>173</td>
</tr>
<tr>
<td>J REFERENCES</td>
<td>180</td>
</tr>
</tbody>
</table>
1.1 SCOPE AND APPLICATION

1.1.1 Scope

This Standard sets out minimum requirements for the design and construction of concrete bridges and associated structures including members that contain reinforcing steel or tendons, or both. It also sets out minimum requirements for plain concrete members.

1.1.2 Application

This Standard applies to concrete structures made using the following materials:

(a) Concrete with a characteristic compressive strength at 28 days \( f'_c \) in the range of 25 MPa to 65 MPa and with a saturated surface-dry density in the range of 2100 kg/m\(^3\) to 2800 kg/m\(^3\).

(b) Reinforcing steels complying with AS/NZS 4671, and the following criteria:

\( f_{sy} \) of 500 MPa and Ductility Class N. These reinforcing materials may be used, without restriction, in all applications referred to in this Standard.

\( f_{sy} \) of 500 MPa and Ductility Class L. These reinforcing materials shall not be used in any situation where the reinforcement is expected to undergo large deformation under strength limit state conditions or any situation where the bar is likely to be bent or rebent on site (see Note 1).

Round bars of yield strength \( f_{sy} \) of 250 MPa and Ductility Class N. These reinforcing bars shall only be used for fitments.

(c) Prestressing tendons complying with AS/NZS 4672.

NOTES:
1 The use of Ductility Class L reinforcement is further limited by other clauses within this Standard.
2 The design of a structure or member to which this Standard applies should be the responsibility of an engineer as defined in AS 5100.1.

1.2 REFERENCED DOCUMENTS

Documents referred to in this Standard are listed in Appendix A.

1.3 DEFINITIONS

For the purpose of this Standard, the definitions below apply. Definitions peculiar to a particular Clause are also given in that Clause.

1.3.1 Action

Any agent, such as an imposed load, foundation movement or temperature gradient, that may act on a structure.

1.3.2 Action effects

The forces and moments, deformations, cracks and other effects that are produced in a structure or in its component members by an action.

1.3.3 Approved

Except as may be otherwise stated, approved by the authority.

1.3.4 Authority

A body with jurisdiction over the provision of bridges and/or responsible for the design, construction and maintenance of bridges within its jurisdiction.
1.3.5 **Average ambient temperature**
The mean value of the daily maximum and minimum ambient temperature at a site, averaged over the relevant period.

1.3.6 **Bottle-shaped compression field**
Compression field that is wider at mid-length than at its ends [see Figure I2.1(c)].

1.3.7 **Cement**
Portland or blended cement complying with AS 3972 or a mixture of these with one or more supplementary cementitious materials complying with AS 3582.

1.3.8 **Characteristic strength**
That value of the material strength, as assessed by standard test, that is exceeded by 95% of the material.

1.3.9 **Composite concrete flexural member**
A member consisting of concrete components constructed separately but structurally connected so that the member responds as a unit to applied actions.

1.3.10 **Concrete**
A mixture of cement, aggregates, and water, with or without additional chemical admixtures.

1.3.11 **Construction joint**
A joint, including a joint between precast segments, that is located in a part of a structure for convenience of construction and made so that the load-carrying capacity and serviceability of the structure will be unimpaired by the inclusion of the joint.

1.3.12 **Cover**
The distance between the outside of the reinforcing steel or tendons and the nearest permanent surface of the member excluding any surface finish. The tolerances on the position of reinforcement and tendons apply.

1.3.13 **Creep coefficient**
Mean value of the ratio of creep strain to elastic strain under conditions of constant stress.

1.3.14 **Critical tensile zone**
A region of a beam or slab where the design bending moment at the serviceability limit state \( M_{S1}^* \), calculated with short-term load factor \( \psi = 1.0 \), is greater than or equal to the critical moment for flexural cracking \( M_{cri} \), which is calculated assuming a flexural tensile strength of concrete equal to 3.0 MPa.

1.3.15 **Curing**
The protection of concrete surfaces after initial set from drying.

1.3.16 **D-region**
Portion of a member within a distance equal to the member depth \( D \), from a discontinuity or abrupt change in geometry or loading, including prestress.

1.3.17 **Design strength**
The ultimate strength modified by the strength reduction factor.

1.3.18 **Development length**
The length over which the full strength of reinforcement or tendons is developed.
1.3.19 Deviator
A device, e.g., concrete block, steel assembly or cross-beam, around which a tendon is bent and where the tendon exerts a radial force on the structure.

1.3.20 Drawings
The drawings forming part of the documents setting out the work to be executed.

1.3.21 Ductility class
A designation relating to the ductility of reinforcement, (‘L’ designates ‘low’, ‘N’ designates ‘normal’).

1.3.22 Effective depth
The distance from the extreme compressive fibre of the concrete to the resultant tensile force in the reinforcing steel and tendons in the zone that will be tensile at the ultimate strength condition in pure bending.

1.3.23 Effective span
The lesser of the \((L_n + D)\) and \(L\).

1.3.24 Engineer
See AS 5100.1.

1.3.25 Exposure classification
A classification representing the degree of severity of exposure to the environment.

1.3.26 External tendon
A post-tensioned tendon situated outside the concrete section but inside the envelope of the concrete structure, only connected to the structure by anchorages and deviators.

1.3.27 Fan-shaped compression field
Compression field that has non-parallel straight sides [see Figure I2.1(b)].

1.3.28 Fitment
Unit of reinforcement commonly used to restrain from buckling the longitudinal reinforcing bars in beams, columns and piles; carry shear, torsion and diagonal tension; act as hangers for longitudinal reinforcement; or provide confinement to the core concrete.
Also referred to commonly as a stirrup, ligature or helical reinforcement.

1.3.29 Flat slab
A continuous two-way solid or ribbed slab, with or without drop-panels, having at least two spans in each direction supported internally by columns without beams and supported externally by walls or columns with or without spandrel beams, or both.

1.3.30 Fly ash
The solid material extracted from the flue gases of a boiler fired with pulverized coal.

1.3.31 Footing
A part of a structure in direct contact with and transmitting load to the supporting foundation.

1.3.32 Foundation
The soil, subsoil or rock, whether built-up or natural, upon which a structure is supported.

1.3.33 Granulated iron blast-furnace slag
The glassy granular material resulting from the rapid chilling of molten iron blast-furnace slag.
1.3.34 **Grout**
A mixture of cement and water, with or without the addition of sand or chemical admixtures, proportioned to produce a pourable liquid without segregation of the constituents.

1.3.35 **Headed reinforcement**
A steel bar that achieves anchorage by means of a suitably sized head or end plate (see Clause 13.1.4).

1.3.36 **Initial force**
The force immediately after transfer, at a stated position in a tendon.

1.3.37 **Internal unbonded tendon**
A cast-in, post-tensioned tendon fabricated from sheathed prestressing strands or steels, connected to the structure by anchorages only.

1.3.38 **Iron blast-furnace slag**
A material consisting essentially of silicates and aluminosilicates of calcium produced simultaneously with iron in a blast furnace.

1.3.39 **Jacking force**
The force in a tendon measured at the jack.

1.3.40 **Lightweight concrete**
Concrete made with lightweight coarse and normal-weight fine aggregate and having a saturated surface-dry density in the range of 1800 kg/m$^3$ to 2100 kg/m$^3$.

1.3.41 **Limit state**
See AS 5100.1.

1.3.42 **Mean strength**
Statistical average of a number of test results representative of the strength of a member, prototype or material.

1.3.43 **Movement joint**
A joint that is made in or between portions of a structure for the specific purpose of permitting relative movement between the parts of the structure on either side of the joint.

1.3.44 **Nodal zone**
Volume of concrete around a node, which is assumed to transfer strut-and-tie forces through the node.

1.3.45 **Node**
Point in a joint in a strut-and-tie model where the axes of the struts, ties and concentrated forces acting on the joint intersect.

1.3.46 **Normal-class concrete**
Concrete that is specified primarily by a standard strength grade and which complies with Clause 16.1.2.

1.3.47 **One-way slab**
A slab characterized by flexural action mainly in one direction.

1.3.48 **Plain concrete member**
A member either unreinforced or containing reinforcement but assumed to be unreinforced.
1.3.49  **Post-tensioning**
The tensioning of tendons after the concrete has hardened.

1.3.50  **Prestressed concrete**
Concrete into which internal stresses are induced deliberately by tendons and includes concrete commonly referred to as partially prestressed.

1.3.51  **Prestressing steel**
*See* tendon.

1.3.52  **Pretensioning**
The tensioning of tendons before the concrete is placed.

1.3.53  **Prismatic compression field**
Compression field that is parallel sided [see Figure I2.1(a)].

1.3.54  **Reinforcement, reinforcing steel**
Steel bar, wire, or fabric but not tendons.

1.3.55  **Shear wall**
A wall that is intended to resist lateral forces acting in or parallel to the plane of the wall.

1.3.56  **Silica fume**
A very fine pozzolanic material composed mostly of amorphous silica produced by electric arc furnaces as a by-product of the production of elemental silicon or ferro-silicon alloys.

1.3.57  **Slag**
Ground granulated blast furnace slag complying with AS 3582.2.

1.3.58  **Special class concrete**
Concrete that is specified to have certain properties or characteristics different from or additional to those of normal class concrete and which complies with Clause 16.1.2.

1.3.59  **Specification**
The specification forming part of the documents setting out the work to be executed.

1.3.60  **Strength grade**
The numerical value of the characteristic compressive strength of concrete at 28 days ($f'_c$) (see Clause 6.1.1).

1.3.61  **Strength ultimate limit state**
*See* AS 5100.1.

1.3.62  **Strut-and-tie model**
Truss model made up of struts and ties connected at nodes.

1.3.63  **Tendon**
A wire, strand or bar or any discrete group of such wires, strands or bars, which may be pretensioned or post-tensioned.

1.3.64  **Tensile reinforcement**
Reinforcement including tendons.

1.3.65  **Tie**
Tension member in a strut-and-tie model.
1.3.66 Transfer
The time of initial transfer of prestressing forces from the tendon to the concrete.

1.3.67 Transmission length
The length, at transfer, over which the stress in a pretensioned tendon builds up from zero at one end to its full value.

1.3.68 Two-way slab
A slab characterized by significant flexural action in two directions, usually at right angles to one another.

1.3.69 Ultimate strength
The capacity of a member to resist load, as determined from its dimensions and characteristic strengths.

1.3.70 Uniform elongation
Uniform elongation of reinforcement corresponding to the onset of necking, as specified in AS/NZS 4671.

1.3.71 Upper characteristic strength
Value of the material strength, as assessed by standard test, which is exceeded by 5% of the material.

1.4 NOTATION
The symbols used in this Standard are listed in Table 1.4. Symbols that occur in more than one clause are defined in the Table and used in the various clauses without further reference. Symbols that occur only in one clause are defined in that clause as well as being listed in the Table.

Unless a contrary intention appears, the following applies:
(a) The symbols used in this Standard shall have the following meanings ascribed to in Table 1.4, with respect to the structure, or member, or condition to which a clause is applied.
(b) Where non-dimensional ratios are involved, both the numerator and denominator are expressed in identical units.
(c) The dimensional units for length, force and stress in all expressions or equations are to be taken as millimetres (mm), newtons (N) and megapascals (MPa) respectively.
(d) An asterisk (*) placed after a symbol as a superscript denotes a design action effect due to the design load for the ultimate limit state specified in AS 5100.2.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Clause reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_b$</td>
<td>cross-sectional area of the reinforcing bar; or area of the bar being spliced</td>
<td>13.1.2.1, 13.2.5</td>
</tr>
<tr>
<td>$A_c$</td>
<td>area of the cross-section of the core measured over the outside of ties</td>
<td>10.7.3.5</td>
</tr>
<tr>
<td>$A_{cs}$</td>
<td>area of cast-in-place concrete</td>
<td>Paragraph E3.2.3, App. E</td>
</tr>
<tr>
<td>$A_{ct}$</td>
<td>area of concrete in the tensile zone, being that part of the section in tension, assuming the section is uncracked</td>
<td>8.6.1</td>
</tr>
<tr>
<td>$A_g$</td>
<td>gross cross-sectional area of the member</td>
<td>6.4.3.2</td>
</tr>
<tr>
<td>$A_k$</td>
<td>area of the base of all the keys in the failure plane</td>
<td>Paragraph B3, App. B</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Clause reference</td>
</tr>
<tr>
<td>--------</td>
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</tr>
<tr>
<td>$A_m$</td>
<td>area enclosed by the median lines of the walls of a single cell</td>
<td>8.3.3</td>
</tr>
<tr>
<td>$A_p$</td>
<td>cross-sectional area of prestressing steel</td>
<td>7.2.11.1</td>
</tr>
<tr>
<td>$A_{pt}$</td>
<td>cross-sectional area of the tendons in the zone that will be tensile under ultimate load conditions</td>
<td>8.1.5</td>
</tr>
<tr>
<td>$A_i$</td>
<td>cross-sectional area of the reinforcement; or area of the reinforcement forming the helix; or area of the horizontal reinforcement at the cantilever depth</td>
<td>6.4.3.2, 10.7.3.5 Paragraph D4, App. D</td>
</tr>
<tr>
<td>$A_{sc}$</td>
<td>cross-sectional area of compressive reinforcement; or area of the additional longitudinal reinforcement</td>
<td>8.1.5, 8.9.4</td>
</tr>
<tr>
<td>$A_{st}$</td>
<td>cross-sectional area of longitudinal reinforcement in the tensile zone</td>
<td>8.1.4.1</td>
</tr>
<tr>
<td>$A_{sv}$</td>
<td>area of the cross-section of the shear reinforcement</td>
<td>8.6.1</td>
</tr>
<tr>
<td>$A_{sv\text{.min}}$</td>
<td>minimum area of shear reinforcement</td>
<td>8.2.8</td>
</tr>
<tr>
<td>$A_{vw}$</td>
<td>cross-sectional area of the bar forming a closed tie; or area of reinforcement provided for suspension ties</td>
<td>8.3.5 Paragraph D2</td>
</tr>
<tr>
<td>$A_t$</td>
<td>area of a polygon with vertices at the centre of longitudinal bars at the corners of the cross-section</td>
<td>8.3.5</td>
</tr>
<tr>
<td>$A_{tr}$</td>
<td>area of the cross-section of the reinforcing bar positioned transversely to the main reinforcement</td>
<td>13.1.2.3</td>
</tr>
<tr>
<td>$A_{tr\text{.min}}$</td>
<td>cross-sectional area of the minimum transverse reinforcement along the development length</td>
<td>13.1.2.3</td>
</tr>
<tr>
<td>$A_1/A_2$</td>
<td>ratio of areas for flanges in shear</td>
<td>8.4.2</td>
</tr>
<tr>
<td>$A_3$</td>
<td>bearing area</td>
<td>12.3</td>
</tr>
<tr>
<td>$A_4$</td>
<td>largest area of the supporting surface that is geometrically similar to and concentric with $A_3$</td>
<td>12.3</td>
</tr>
<tr>
<td>$a$</td>
<td>distance from the stress in the tendon to the jacking end; or distance between points of zero bending moment; or perpendicular distance from the nearer support to the section under consideration; or minimum cover to the deformed bar or the clear distance between adjacent parallel bars developing stress, whichever is less; or dimension of the critical shear perimeter which is parallel to the direction of bending being considered; or geometrical dimension of fillet</td>
<td>6.4.2, 8.8.2, 9.6, 13.1.2.1, 15.4.2 (Figure 9.2.3) Figure F1</td>
</tr>
<tr>
<td>$a_v$</td>
<td>distance from the section at which shear is being considered to the face of the nearest support</td>
<td>8.2.7.1</td>
</tr>
<tr>
<td>$b$</td>
<td>width of the cross-section; or height of wall</td>
<td>2.5.4, 11.6.2</td>
</tr>
<tr>
<td>$b_{cs}$</td>
<td>width of the compression strut</td>
<td>12.1.2.2</td>
</tr>
<tr>
<td>$b_{ef}$</td>
<td>effective width of a compression face or flange of a member</td>
<td>8.1.5</td>
</tr>
<tr>
<td>$b_{tr}$</td>
<td>width of the shear interface</td>
<td>8.4.3</td>
</tr>
<tr>
<td>$b_{fc}$</td>
<td>width of the portion of the flange (in a box girder)</td>
<td>Paragraph F1</td>
</tr>
<tr>
<td>$b_{sc}$</td>
<td>width of the critical opening</td>
<td>9.2.3</td>
</tr>
<tr>
<td>$b_s$</td>
<td>width of the flange of a composite member</td>
<td>Paragraph H4</td>
</tr>
<tr>
<td>$b_c$</td>
<td>effective width of the web for shear</td>
<td>8.2.6</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Clause reference</td>
</tr>
<tr>
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</tr>
<tr>
<td>( b_w )</td>
<td>width of the web or the thickness of the wall being considered or minimum thickness of the wall of a hollow section</td>
<td>8.3.3</td>
</tr>
<tr>
<td>( c )</td>
<td>concrete cover to reinforcing steel or tendons</td>
<td>13.1.2.2</td>
</tr>
<tr>
<td>( c_d )</td>
<td>the smaller of the concrete cover to the deformed bar; and half the clear distance to the next parallel bar</td>
<td>13.1.2.3</td>
</tr>
<tr>
<td>( D )</td>
<td>overall depth of the cross-section in the plane of bending; or overall depth of the column in the plane of the bending moment; or thickness of wall</td>
<td>8.1.4.1, 10.1.2, 11.6.2</td>
</tr>
<tr>
<td>( D_c )</td>
<td>smaller column dimension if rectangular or the column diameter if circular; or diameter of the inside face of the helix</td>
<td>10.7.3.3, 10.7.3.5</td>
</tr>
<tr>
<td>( D_s )</td>
<td>overall depth of the slab</td>
<td>9.4.1</td>
</tr>
<tr>
<td>( d )</td>
<td>effective depth of the cross-section</td>
<td>8.1.3</td>
</tr>
<tr>
<td>( d_b )</td>
<td>nominal diameter of the bar, wire or tendon</td>
<td>8.1.8.3</td>
</tr>
<tr>
<td>( d_c )</td>
<td>width of the idealized strut</td>
<td>12.4</td>
</tr>
<tr>
<td>( d_d )</td>
<td>diameter of the grouted duct</td>
<td>8.2.6</td>
</tr>
<tr>
<td>( d_{ef} )</td>
<td>effective depth of a hypothetical symmetrically loaded prism of a post-tensioned end block</td>
<td>Paragraph G5.1</td>
</tr>
<tr>
<td>( d_{ef1}, d_{ef2} )</td>
<td>effective depths for various hypothetical prisms</td>
<td>Figure G1</td>
</tr>
<tr>
<td>( d_f )</td>
<td>thickness of the compression flange</td>
<td>8.1.6</td>
</tr>
<tr>
<td>( d_i )</td>
<td>minimum internal diameter of a bend in a reinforcing bar</td>
<td>Figure 13.1.2</td>
</tr>
<tr>
<td>( d_o )</td>
<td>distance from the extreme compressive fibre of the concrete to the centroid of the outermost layer of tensile reinforcement or tendons but not less than 0.8D</td>
<td>2.5.4</td>
</tr>
<tr>
<td>( d_{om} )</td>
<td>mean value of ( d_o ), averaged around the critical shear perimeter ((u))</td>
<td>9.2.3</td>
</tr>
<tr>
<td>( d_p )</td>
<td>distance from the extreme compressive fibre of the concrete to the centroid of the tendons in that zone, which will be tensile under ultimate limit state conditions</td>
<td>8.1.5</td>
</tr>
<tr>
<td>( d_s )</td>
<td>depth of slab in composite member</td>
<td>Figure H3, App. H</td>
</tr>
<tr>
<td>( d_{sc} )</td>
<td>distance from the extreme compressive fibre of the concrete to the centroid of compression reinforcement</td>
<td>8.1.5</td>
</tr>
<tr>
<td>( d_{sp} )</td>
<td>distance resisting splitting under a curved prestressing duct</td>
<td>8.1.7.5</td>
</tr>
<tr>
<td>( d_{x, sup} )</td>
<td>distance to extreme fibre from the major ( x-x ) axis</td>
<td>Figure C1</td>
</tr>
<tr>
<td>( E_c )</td>
<td>mean value of the modulus of elasticity of concrete at 28 days</td>
<td>6.1.2</td>
</tr>
<tr>
<td>( E_{c,i} )</td>
<td>mean value of modulus of elasticity of concrete at the appropriate age</td>
<td>6.1.2</td>
</tr>
<tr>
<td>( E_d )</td>
<td>design action effect</td>
<td>2.2</td>
</tr>
<tr>
<td>( E_p )</td>
<td>modulus of elasticity of tendons</td>
<td>6.3.2</td>
</tr>
<tr>
<td>( E_s )</td>
<td>modulus of elasticity of reinforcement</td>
<td>6.2.2</td>
</tr>
<tr>
<td>( e )</td>
<td>base of Napierian logarithms; or eccentricity of the prestressing force ((P)), measured from the centroidal axis of the uncracked section</td>
<td>6.4.2 (Paragraph E3.2) 8.1.4.1</td>
</tr>
<tr>
<td>( e_o )</td>
<td>vertical eccentricity between the centre of gravity of a beam and the longitudinal axis through the lifting points</td>
<td>Appendix C</td>
</tr>
<tr>
<td>( e_s )</td>
<td>distance between the location of the tendon centre of gravity and the centre-line of the duct</td>
<td>8.1.9</td>
</tr>
<tr>
<td>( e_s )</td>
<td>eccentricity of the lifting point to the minor centroidal axis of a beam</td>
<td>Appendix C</td>
</tr>
<tr>
<td>( F' )</td>
<td>design suspended load at the strength ultimate limit state (action effect)</td>
<td>Paragraph D2</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Clause reference</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
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</tr>
<tr>
<td>$f_c^*$</td>
<td>absolute value of the design force in the compression zone as a result of</td>
<td>8.3.6</td>
</tr>
<tr>
<td></td>
<td>flexure</td>
<td></td>
</tr>
<tr>
<td>$F^*_t$</td>
<td>design tensile force in a tensile tie of an analogous truss; or</td>
<td>Paragraph D3</td>
</tr>
<tr>
<td></td>
<td>the design vertical reaction per web of the main girder</td>
<td></td>
</tr>
<tr>
<td>$f'_c$</td>
<td>characteristic compressive cylinder strength of concrete at 28 days</td>
<td>1.1.2</td>
</tr>
<tr>
<td>$f_{ct}$</td>
<td>characteristic uniaxial tensile strength of concrete</td>
<td>6.1.1</td>
</tr>
<tr>
<td>$f_{ct}t$</td>
<td>characteristic flexural tensile strength of concrete</td>
<td>6.1.1</td>
</tr>
<tr>
<td>$f_t$</td>
<td>uniaxial tensile strength of concrete</td>
<td>6.1.1</td>
</tr>
<tr>
<td>$f_{ct}t$</td>
<td>measured flexural tensile strength of concrete</td>
<td>6.1.1</td>
</tr>
<tr>
<td>$f_{cs}$</td>
<td>measured splitting tensile strength of concrete</td>
<td>6.1.1</td>
</tr>
<tr>
<td>$f_{cal}$</td>
<td>calculated compressive strength of concrete in a compressive strut</td>
<td>12.1.2.2</td>
</tr>
<tr>
<td>$f_{cm}$</td>
<td>mean value of the compressive strength of concrete at the relevant age</td>
<td>6.1.2</td>
</tr>
<tr>
<td>$f_{cmi}$</td>
<td>mean value of the in situ compressive strength of concrete at the relevant</td>
<td>6.1.1 and 6.1.2</td>
</tr>
<tr>
<td></td>
<td>age</td>
<td></td>
</tr>
<tr>
<td>$f_{cp}$</td>
<td>compressive strength of concrete at transfer</td>
<td>8.1.4.2</td>
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<tr>
<td>$f_{cs}$</td>
<td>maximum shrinkage-induced tensile stress on the uncracked section at the</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td></td>
<td>extreme fibre at which cracking occurs</td>
<td></td>
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<tr>
<td>$f_{cv}$</td>
<td>concrete shear strength</td>
<td>9.2.3</td>
</tr>
<tr>
<td>$f_{ph}$</td>
<td>characteristic minimum breaking strength</td>
<td>6.3.1, 6.3.4 and 13.3</td>
</tr>
<tr>
<td>$f_{ps}$</td>
<td>yield strength of tendons</td>
<td>6.3.1</td>
</tr>
<tr>
<td>$f_s$</td>
<td>maximum tensile stress permitted in the reinforcement immediately after</td>
<td>8.6.1</td>
</tr>
<tr>
<td></td>
<td>formation of crack</td>
<td></td>
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<tr>
<td>$f_{si}$</td>
<td>stress in reinforcement in the $i$th direction crossing a strut</td>
<td>12.4</td>
</tr>
<tr>
<td>$f_{sy}$</td>
<td>yield strength of reinforcing steel</td>
<td>1.1.2</td>
</tr>
<tr>
<td>$f_{sy}t$</td>
<td>yield strength of the reinforcement used as fitments</td>
<td>8.2.8</td>
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<tr>
<td>$g_p$</td>
<td>permanent distributed load normal to the shear interface per unit length,</td>
<td>8.4.3</td>
</tr>
<tr>
<td></td>
<td>newtons per millimetre (N/mm)</td>
<td></td>
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<tr>
<td>$I$</td>
<td>second moment of area of the gross concrete cross-section</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td>$I_c$</td>
<td>second moment of area of a column</td>
<td>7.3.1</td>
</tr>
<tr>
<td>$I_{ct}$</td>
<td>second moment of area of a cracked section with the reinforcement</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td></td>
<td>transformed to an equivalent area of concrete</td>
<td></td>
</tr>
<tr>
<td>$I_{ct}$</td>
<td>effective second moment of area of the member</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td>$I_{ct, max}$</td>
<td>maximum of the effective second moment of area of a member</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td>$I_t$</td>
<td>second moment of area of a flexural member</td>
<td>7.3.1</td>
</tr>
<tr>
<td>$(I/L)_c$</td>
<td>stiffness in the plane of bending of only the column under consideration</td>
<td>10.5.4</td>
</tr>
<tr>
<td>$(I/L)_n$</td>
<td>bending stiffness in the plane of bending of a flexural member</td>
<td>10.5.4</td>
</tr>
<tr>
<td>$J_e$</td>
<td>torsional modulus</td>
<td>8.3.3</td>
</tr>
<tr>
<td>$J_{th}$</td>
<td>torsional modulus of a standard precast prestressed concrete member</td>
<td>Paragraph H4</td>
</tr>
<tr>
<td>$J_{tn}$</td>
<td>torsional modulus of a composite section</td>
<td>Paragraph H4</td>
</tr>
<tr>
<td>$j$</td>
<td>time after prestressing, in days</td>
<td>6.3.4.3</td>
</tr>
<tr>
<td>$K$</td>
<td>a factor that accounts for the position of the bars being anchored with</td>
<td>13.1.2.3</td>
</tr>
<tr>
<td></td>
<td>respect to the transverse reinforcement</td>
<td></td>
</tr>
<tr>
<td>$k$</td>
<td>effective length factor</td>
<td>10.5.3</td>
</tr>
<tr>
<td>$K_{so}$</td>
<td>cohesion coefficient</td>
<td>8.4.3</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Clause reference</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td>------------------</td>
</tr>
<tr>
<td>$k_{cs}$</td>
<td>a factor used in serviceability design to take account of the long-term effects of creep and shrinkage</td>
<td>8.5.3.2</td>
</tr>
<tr>
<td>$k_{m}$</td>
<td>a coefficient</td>
<td>10.4.2</td>
</tr>
<tr>
<td>$k_{r}$</td>
<td>ratio of the depth or breadth of an anchorage bearing plate to the corresponding depth or breadth of the symmetrical prism</td>
<td>12.2.4</td>
</tr>
<tr>
<td>$k_{s}$</td>
<td>a coefficient that takes into account the shape of the stress distribution within the section immediately prior to cracking, as well as the effect of non-uniform self-equilibrating stresses</td>
<td>8.6.1</td>
</tr>
<tr>
<td>$k_{u}$</td>
<td>neutral axis parameter, being the ratio, at ultimate strength and under any combination of bending and compression, of the depth of the neutral axis from the extreme compressive fibre, to $d$</td>
<td>7.2.8.2</td>
</tr>
<tr>
<td>$k_{ud}$</td>
<td>neutral axis depth</td>
<td>8.1.6</td>
</tr>
<tr>
<td>$k_{uo}$</td>
<td>ratio, at ultimate strength, of the depth to the neutral axis from the extreme compressive fibre to $d_{o}$</td>
<td>Table 2.2</td>
</tr>
<tr>
<td>$k_{1}$</td>
<td>shrinkage strain coefficient</td>
<td>6.1.7</td>
</tr>
<tr>
<td>$k_{2}$</td>
<td>a coefficient for determining creep</td>
<td>6.1.8.3</td>
</tr>
<tr>
<td>$k_{3}$</td>
<td>maturity coefficient</td>
<td>6.1.8.3</td>
</tr>
<tr>
<td>$k_{4}$</td>
<td>a coefficient for determining creep; or a coefficient, dependent on the duration of the prestressing force</td>
<td>6.1.8.3</td>
</tr>
<tr>
<td>$k_{5}$</td>
<td>a coefficient for determining creep; or a coefficient, dependent on the stress in the tendon as a proportion of $f_{pb}$</td>
<td>6.1.8.3</td>
</tr>
<tr>
<td>$k_{6}$</td>
<td>a function, dependent on the average annual temperature ($T$)°C</td>
<td>6.3.4.3</td>
</tr>
<tr>
<td>$k_{7}$, $k_{8}$</td>
<td>factors used in calculating development length</td>
<td>13.1.2.1</td>
</tr>
<tr>
<td>$L$</td>
<td>centre-to-centre distance between the supports of a flexural member</td>
<td>8.8.2</td>
</tr>
<tr>
<td>$L_{a}$</td>
<td>minimum length of bar to form standard hook</td>
<td>Figure 13.1.2</td>
</tr>
<tr>
<td>$L_{c}$</td>
<td>effective length of the column</td>
<td>10.3.1</td>
</tr>
<tr>
<td>$L_{L}$</td>
<td>distance between centres of lateral restraints</td>
<td>8.9.2</td>
</tr>
<tr>
<td>$L_{n}$</td>
<td>length of a clear span in the direction in which moments are being determined, measured face-to-face of supporting beams, columns or walls; or for a cantilever, the clear projection</td>
<td>8.9.3</td>
</tr>
<tr>
<td>$L_{p}$</td>
<td>development length of pretensioned tendons for gradual release</td>
<td>13.3.2</td>
</tr>
<tr>
<td>$L_{pa}$</td>
<td>length of tendons</td>
<td>8.1.6</td>
</tr>
<tr>
<td>$L_{pa}$</td>
<td>length of the tendon from the jacking end to the point at a distance ($a$) from that end</td>
<td>6.4.2</td>
</tr>
<tr>
<td>$L_{pe}$</td>
<td>effective length of the tendons</td>
<td>8.1.6</td>
</tr>
<tr>
<td>$L_{pt}$</td>
<td>transmission length of pretensioned tendons for gradual release</td>
<td>13.3.2</td>
</tr>
<tr>
<td>$L_{sc}$</td>
<td>development length of a bar for a compressive stress less than the yield stress</td>
<td>13.1.5.4</td>
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<tr>
<td>$L_{sl}$</td>
<td>development length (of a bar) to develop a tensile stress less than the yield strength</td>
<td>13.1.2.4</td>
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<tr>
<td>$L_{sy,c}$</td>
<td>development length in compression, being the length of embedment required to develop the yield strength of a deformed bar in compression</td>
<td>13.1.5.1</td>
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<tr>
<td>$L_{sy,cb}$</td>
<td>basic development length of a deformed bar in compression</td>
<td>13.1.5.2</td>
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<tr>
<td>$L_{sy,t}$</td>
<td>development length in tension, to develop the characteristic yield strength of a deformed bar in tension</td>
<td>13.1.2</td>
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<tr>
<td>$L_{sy,t,lap}$</td>
<td>the tensile lap length for either contact or non-contact splices</td>
<td>13.2.2</td>
</tr>
<tr>
<td>$L_{sy,th}$</td>
<td>basic development length of a deformed bar in tension</td>
<td>13.1.2.2</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Clause reference</td>
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<tr>
<td>---------</td>
<td>-----------------------------------------------------------------------------</td>
<td>--------------------------------</td>
</tr>
<tr>
<td>$L_u$</td>
<td>unsupported length of the column, taken as the clear distance between the</td>
<td>7.3.2, 10.2.2, 10.5.3</td>
</tr>
<tr>
<td></td>
<td>faces of members capable of providing lateral support to the column or</td>
<td></td>
</tr>
<tr>
<td></td>
<td>between the lowest extremity of the capital or haunch where column capitals</td>
<td></td>
</tr>
<tr>
<td></td>
<td>or haunches are present</td>
<td></td>
</tr>
<tr>
<td>$L_1$</td>
<td>half the maximum length of positive design moment on the span of a box</td>
<td>Paragraph F1</td>
</tr>
<tr>
<td></td>
<td>girder</td>
<td></td>
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<tr>
<td>$L_2$</td>
<td>maximum length of negative design moment on the span of a box girder</td>
<td>Paragraph F1</td>
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<tr>
<td>$I_b$</td>
<td>length of the bursting zone</td>
<td>12.4</td>
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<tr>
<td>$M'$</td>
<td>design bending moment (action effect) at a cross-section calculated using</td>
<td>8.2.7.2</td>
</tr>
<tr>
<td></td>
<td>the design loads for strength specified in AS 5100.2, i.e., the design</td>
<td></td>
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<tr>
<td></td>
<td>bending moment</td>
<td></td>
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<tr>
<td>$M_{h1}$</td>
<td>design lateral bending moment in a beam during erection</td>
<td>Appendix C</td>
</tr>
<tr>
<td>$M_{s1}$</td>
<td>design bending moment at the serviceability limit state</td>
<td>9.4.1</td>
</tr>
<tr>
<td>$M_x^<em>$, $M_y^</em>$</td>
<td>design bending moment about the major $x$-axis and minor $y$-axis, respectively</td>
<td>10.6.5</td>
</tr>
<tr>
<td>$M_1^<em>$, $M_2^</em>$</td>
<td>design bending moments at the ends of the column</td>
<td>10.3.1</td>
</tr>
<tr>
<td>$M_{ci}$</td>
<td>bending moment causing cracking of a section with due consideration to</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td></td>
<td>prestress, restrained shrinkage and temperature stresses</td>
<td></td>
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<tr>
<td>$M_{cr}$</td>
<td>critical moment for flexural cracking</td>
<td>9.4.1</td>
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<tr>
<td>$M_{pl}$</td>
<td>particular ultimate strength in bending of the cross-section assuming $k_u$</td>
<td>10.4.4</td>
</tr>
<tr>
<td></td>
<td>equals 0.545</td>
<td></td>
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<tr>
<td>$M_{ud}$</td>
<td>reduced ultimate strength of the cross-section in bending without axial force</td>
<td>8.1.3 (Table 2.2)</td>
</tr>
<tr>
<td>$M_{uo}$</td>
<td>ultimate strength in bending without axial force at a cross-section</td>
<td>8.1.4.1 (Table 2.2)</td>
</tr>
<tr>
<td>$M_{ux}$</td>
<td>ultimate strength in bending about the major $x$-axis of a column under the</td>
<td>10.6.5</td>
</tr>
<tr>
<td></td>
<td>design axial force ($N'$)</td>
<td></td>
</tr>
<tr>
<td>$M_{uy}$</td>
<td>ultimate strength in bending about the minor $y$-axis of a column under the</td>
<td>10.6.5</td>
</tr>
<tr>
<td></td>
<td>design axial force ($N'$)</td>
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</tr>
<tr>
<td>$N_c$</td>
<td>buckling load used in column design</td>
<td>10.4.4</td>
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<tr>
<td>$N_u$</td>
<td>ultimate strength in compression or tension at a cross-section of an</td>
<td>Table 2.2</td>
</tr>
<tr>
<td></td>
<td>eccentrically loaded member</td>
<td></td>
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<tr>
<td>$N_{ub}$</td>
<td>particular ultimate strength in compression of a cross-section when $k_{uu}$</td>
<td>Table 2.2</td>
</tr>
<tr>
<td></td>
<td>is equal to 0.6</td>
<td></td>
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<tr>
<td>$N_{uo}$</td>
<td>ultimate strength in compression of an axially loaded cross-section, without</td>
<td>10.6.3</td>
</tr>
<tr>
<td></td>
<td>bending forces</td>
<td></td>
</tr>
<tr>
<td>$N_{uot}$</td>
<td>ultimate strength in tension of an axially loaded cross-section, without</td>
<td>Table 2.2</td>
</tr>
<tr>
<td></td>
<td>bending forces</td>
<td></td>
</tr>
<tr>
<td>$n$</td>
<td>number of stress cycles; or</td>
<td>2.5.5</td>
</tr>
<tr>
<td></td>
<td>number of bars uniformly spaced around a helix</td>
<td>13.2.4</td>
</tr>
<tr>
<td>$n_s$</td>
<td>number of support hinges crossed by the tendon (draped tendons only)</td>
<td>8.1.6</td>
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</table>
### TABLE 1.4 (continued)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Clause reference</th>
</tr>
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<tbody>
<tr>
<td>$P$</td>
<td>prestressing force; or maximum force occurring at the anchorage during jacking</td>
<td>8.1.4.1 12.2.4</td>
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<tr>
<td>$P_v$</td>
<td>vertical component of the prestressing force at the section under consideration</td>
<td>8.2.5</td>
</tr>
<tr>
<td>$p$</td>
<td>steel ratio</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td>$R'$</td>
<td>design vertical reaction at the strength ultimate limit state</td>
<td>Paragraph D3, App. D</td>
</tr>
<tr>
<td>$R_h'$</td>
<td>design horizontal reaction at the strength ultimate limit state</td>
<td>Paragraph D4, App. D</td>
</tr>
<tr>
<td>$R_v'$</td>
<td>design vertical reaction at the strength ultimate limit state</td>
<td>Paragraph D3, App. D</td>
</tr>
<tr>
<td>$R$</td>
<td>design relaxation of a tendon</td>
<td>6.3.4.3</td>
</tr>
<tr>
<td>$R_b$</td>
<td>basic relaxation of a tendon after 1000 h</td>
<td>6.3.4.2</td>
</tr>
<tr>
<td>$R_d$</td>
<td>design capacity</td>
<td>2.2.2</td>
</tr>
<tr>
<td>$R_u$</td>
<td>ultimate strength</td>
<td>2.2.2</td>
</tr>
<tr>
<td>$r$</td>
<td>radius of curvature of the prestressing tendon; or radius of curvature of the duct; or radius of gyration of the cross-section</td>
<td>7.2.11.1 8.1.7.4 10.5.2</td>
</tr>
<tr>
<td>$S'$</td>
<td>design action effect</td>
<td>2.2</td>
</tr>
<tr>
<td>$s$</td>
<td>centre-to-centre spacing of shear or torsional reinforcement, measured parallel to the longitudinal axis of a member; or standard deviation; or maximum spacing of transverse reinforcement within $L_{yc}^c$, or spacing of fitments, or spacing of successive turns of helical reinforcement, all measured centre-to-centre, in millimetres; or spacing of anchored shear reinforcement crossing interface</td>
<td>8.2.10 13.2.4 8.4.3</td>
</tr>
<tr>
<td>$s_b$</td>
<td>clear distance between bars of the non-contact lapped splice</td>
<td>13.2.2</td>
</tr>
<tr>
<td>$s_d$</td>
<td>centre-to-centre distance between lines of ducts in the plane of the curvature</td>
<td>8.1.7.4</td>
</tr>
<tr>
<td>$s_L$</td>
<td>clear distance between bars of the non-contact lapped splice</td>
<td>13.2.2</td>
</tr>
<tr>
<td>$s_1$, $s_2$</td>
<td>spacing of transverse wires in welded wire fabric</td>
<td>Figure 13.2.4</td>
</tr>
<tr>
<td>$T'$</td>
<td>design torsional moment at a cross-section (action effect) calculated using the design loads for strength specified in AS 5100.2, i.e., the design torsional moment</td>
<td>8.3.3</td>
</tr>
<tr>
<td>$T$</td>
<td>temperature; or bursting force resultant of tensile stresses</td>
<td>6.3.4.3 8.3.3</td>
</tr>
<tr>
<td>$T_{uc}$</td>
<td>ultimate strength in pure torsion of a beam without torsional reinforcement</td>
<td>8.3.5</td>
</tr>
<tr>
<td>$T_{b}$</td>
<td>bursting force calculated at the ultimate limit state</td>
<td>12.4</td>
</tr>
<tr>
<td>$T_{b,s}$</td>
<td>bursting force calculated at the serviceability state</td>
<td>12.4</td>
</tr>
<tr>
<td>$T_{b,cr}$</td>
<td>bursting (or splitting) force across a strut caused at the time of cracking of the strut</td>
<td>12.4</td>
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<tr>
<td>$T_{us}$</td>
<td>ultimate strength in pure torsion of a beam with torsional reinforcement (closed ties)</td>
<td>8.3.5</td>
</tr>
<tr>
<td>$T_{u,\text{max.}}$</td>
<td>ultimate torsional strength of a beam limited by web crushing failure</td>
<td>8.3.3</td>
</tr>
<tr>
<td>$T_v$</td>
<td>vertical component of the force carried by the secondary struts</td>
<td>12.1.2.1</td>
</tr>
<tr>
<td>$t_b$</td>
<td>thickness of the bottom slab of standard Super T-girder</td>
<td>Figure H1</td>
</tr>
<tr>
<td>$t_f$</td>
<td>thickness of topping or flange anchored by shear reinforcement</td>
<td>8.4.4</td>
</tr>
<tr>
<td>$t_h$</td>
<td>hypothetical thickness of the member (used in determining creep and shrinkage)</td>
<td>6.1.7</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Clause reference</td>
</tr>
<tr>
<td>--------</td>
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<tr>
<td>$t_j$</td>
<td>time when continuity and composite action are established in all spans simultaneously (used to calculate the restraint moment)</td>
<td>Paragraph E3.2.2, App. E</td>
</tr>
<tr>
<td>$t_w$</td>
<td>thickness of the wall</td>
<td>11.6.3</td>
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<tr>
<td>$u$</td>
<td>length of the critical shear perimeter</td>
<td>9.2.3</td>
</tr>
<tr>
<td>$u_e$</td>
<td>exposed perimeter of a member cross-section plus half the perimeter of any closed voids contained therein</td>
<td>6.1.7</td>
</tr>
<tr>
<td>$u_s$</td>
<td>perimeter of the polygon defined for $A_1$</td>
<td>8.3.6</td>
</tr>
<tr>
<td>$p'$</td>
<td>design shear force (action effect) calculated using the design loads for strength specified in AS 5100.2, i.e., the design shear force</td>
<td>8.1.8.3</td>
</tr>
<tr>
<td>$V_o$</td>
<td>shear force which would occur at the section under consideration when the bending moment at that section was equal to the decompression moment ($M_o$)</td>
<td>8.2.7.2</td>
</tr>
<tr>
<td>$V_i$</td>
<td>shear force which, in combination with the prestressing force and other action effects at the section, would produce a principal tensile stress of 0.33 $f'_c$ at either the centroidal axis or the intersection of flange and web, whichever is the more critical</td>
<td>8.2.7.2</td>
</tr>
<tr>
<td>$V_a$</td>
<td>ultimate shear strength</td>
<td>8.2.6</td>
</tr>
<tr>
<td>$V_{a,\text{max}}$</td>
<td>ultimate shear strength limited by web crushing failure</td>
<td>8.2.6</td>
</tr>
<tr>
<td>$V_{a,\text{min}}$</td>
<td>ultimate shear strength of a beam provided with minimum shear reinforcement</td>
<td>8.2.9</td>
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<tr>
<td>$V_{uc}$</td>
<td>ultimate shear strength excluding the contribution of the shear reinforcement</td>
<td>8.2.7.1</td>
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<td>$V_{uf}$</td>
<td>ultimate longitudinal shear strength at an interface</td>
<td>8.4.3</td>
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<tr>
<td>$V_{uj}$</td>
<td>nominal shear resistance at the joint for structures utilizing dry joints</td>
<td>Paragraph B3, App. B</td>
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<tr>
<td>$V_{uo}$</td>
<td>ultimate shear strength of a slab with no moment transfer</td>
<td>9.2.3</td>
</tr>
<tr>
<td>$V_{us}$</td>
<td>contribution to the ultimate shear strength by perpendicular shear reinforcement in a beam or wall</td>
<td>8.2.10</td>
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<tr>
<td>$w$</td>
<td>width of loaded area or node</td>
<td>Figure 12.1.2.1 and Figure 12.4 (A)</td>
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<tr>
<td>$X$</td>
<td>shortest overall dimension of the effective loaded area measured perpendicular to $Y$</td>
<td>9.2.3</td>
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<tr>
<td>$x$</td>
<td>shorter overall dimension of a rectangular part of a cross-section</td>
<td>8.3.3</td>
</tr>
<tr>
<td>$Y$</td>
<td>longest overall dimension of the effective loaded area</td>
<td>9.2.3</td>
</tr>
<tr>
<td>$y$</td>
<td>longer overall dimension of a rectangular part of a cross-section</td>
<td>8.3.3</td>
</tr>
<tr>
<td>$y_b$</td>
<td>depth from the centroidal axis to the extreme fibre at the bottom of the section</td>
<td>Figure H1</td>
</tr>
<tr>
<td>$y_t$</td>
<td>depth from the centroidal axis to the extreme fibre at the top of the section</td>
<td>Appendix C</td>
</tr>
<tr>
<td>$y_c$</td>
<td>larger dimension of the closed tie; or larger core dimension of the closed rectangular tie</td>
<td>8.3.7, 10.7.3.5</td>
</tr>
<tr>
<td>$Z$</td>
<td>section modulus of the uncracked cross-section</td>
<td>8.1.4.1</td>
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<tr>
<td>$Z_b$</td>
<td>section modulus of area about the centroidal axis of the bottom of an uncracked cross-section</td>
<td>Paragraph H3, App. H</td>
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<tr>
<td>$Z_t$</td>
<td>section modulus of area about the centroidal axis of the top of an uncracked cross-section</td>
<td>Paragraph H3, App. H</td>
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<tr>
<td>$z$</td>
<td>projection of the inclined compressive strut normal to the shear span</td>
<td>12.4</td>
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<tr>
<td>$\alpha$</td>
<td>angle of divergence between bottle shape compression fields and idealized parallel sided strut</td>
<td>12.4</td>
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<tr>
<td>$\alpha_\ell$</td>
<td>stress range factor</td>
<td>2.5.5</td>
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TABLE 1.4 (continued)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Clause reference</th>
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<tbody>
<tr>
<td>(\alpha_c)</td>
<td>modular ratio of the cast-in-place concrete and the precast beam concrete in a composite member</td>
<td>Paragraph E2.2, App. E</td>
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<tr>
<td>(\alpha_s)</td>
<td>angle of the inclination of inclined bars in a stepped joint</td>
<td>Paragraph D4, App. D</td>
</tr>
<tr>
<td>(\alpha_n)</td>
<td>a coefficient</td>
<td>10.6.5</td>
</tr>
<tr>
<td>(\alpha_{\text{tot}})</td>
<td>sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length ((L_{pa}))</td>
<td>6.4.2</td>
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<tr>
<td>(\alpha_e)</td>
<td>angle between the inclined shear reinforcement and the longitudinal tensile reinforcement</td>
<td>8.2.10</td>
</tr>
<tr>
<td>(\beta)</td>
<td>an effective compression strength factor; or a design longitudinal shear stress factor; or a fixity factor</td>
<td>2.2.3, 8.4.2, 10.5.4</td>
</tr>
<tr>
<td>(\beta_1, \beta_2, \beta_3)</td>
<td>coefficients for calculating shear strength</td>
<td>8.2.7</td>
</tr>
<tr>
<td>(\beta_d)</td>
<td>ratio of dead load to total load at the ultimate limit state</td>
<td>10.4.3</td>
</tr>
<tr>
<td>(\beta_h)</td>
<td>ratio of the longest overall dimension of the effective loaded area ((Y)) to the shortest overall dimension ((X)) measured perpendicular to (Y)</td>
<td>9.2.3</td>
</tr>
<tr>
<td>(\beta_n)</td>
<td>factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone</td>
<td>14.2</td>
</tr>
<tr>
<td>(\beta_p)</td>
<td>estimate in radians per metre of the angular deviation due to wobble effects</td>
<td>6.4.2</td>
</tr>
<tr>
<td>(\beta_s)</td>
<td>strut efficiency factor</td>
<td>12.2</td>
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<tr>
<td>(\beta_v)</td>
<td>angle of tilt assumed for the calculation of stability of a slender beam during erection</td>
<td>Appendix C</td>
</tr>
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<td>(\gamma)</td>
<td>ratio, under design bending or combined bending and compression, of the depth of the assumed rectangular compressive stress block to (k_u d); or end restraint coefficients of a column</td>
<td>8.1.2.2, 10.5.4</td>
</tr>
<tr>
<td>(\gamma_1, \gamma_2)</td>
<td>end restraint coefficients of a column</td>
<td>10.5.3</td>
</tr>
<tr>
<td>(\gamma_i)</td>
<td>angle between the axis of a strut and the bars in the (i)th direction of reinforcement crossing that strut</td>
<td>12.4</td>
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<td>(\Delta_h)</td>
<td>lateral deviation of a slender beam at mid-span from the specified datum line immediately after transfer</td>
<td>Appendix C</td>
</tr>
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<td>(\Delta \sigma_p)</td>
<td>change in the stress due to the change in length of the prestressed tie</td>
<td>13.2</td>
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<tr>
<td>(\Delta_y)</td>
<td>lateral deflection caused by the self weight of a slender beam about the (y-y) axis during erection</td>
<td>Appendix C</td>
</tr>
<tr>
<td>(\delta, \delta_0, \delta_1)</td>
<td>moment magnifiers for braced columns</td>
<td>10.4.2, 10.2.2, 10.4.3</td>
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<tr>
<td>(\varepsilon_{cc})</td>
<td>strain due to concrete creep</td>
<td>6.4.3.3</td>
</tr>
<tr>
<td>(\varepsilon_{cs})</td>
<td>design shrinkage strain of concrete</td>
<td>6.1.7</td>
</tr>
<tr>
<td>(\varepsilon_{cs.b})</td>
<td>basic shrinkage strain of concrete</td>
<td>6.1.7</td>
</tr>
<tr>
<td>(\varepsilon_{cs}^*)</td>
<td>final design shrinkage strain</td>
<td>6.1.7.2</td>
</tr>
<tr>
<td>(\varepsilon_{cs.d})</td>
<td>drying shrinkage strain</td>
<td>6.1.7.2</td>
</tr>
<tr>
<td>(\varepsilon_{cs.e})</td>
<td>autogenous shrinkage strain</td>
<td>6.1.7.2</td>
</tr>
<tr>
<td>(\varepsilon_{cs.e}^*)</td>
<td>final autogenous shrinkage strain</td>
<td>6.1.7.2</td>
</tr>
<tr>
<td>(\varepsilon_{cs.d.b})</td>
<td>basic drying shrinkage strain</td>
<td>6.1.7.2</td>
</tr>
<tr>
<td>(\varepsilon_{cs.d.b}^*)</td>
<td>final drying basic shrinkage strain</td>
<td>6.1.7.2</td>
</tr>
<tr>
<td>(\varepsilon_{pu})</td>
<td>strain at maximum stress of a prestressing tendon</td>
<td>6.2.1</td>
</tr>
<tr>
<td>(\varepsilon_{su})</td>
<td>uniform elongation (taken as the strain in the steel reinforcement or prestressing steel at ultimate strength)</td>
<td>6.2.1</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Clause reference</td>
</tr>
<tr>
<td>-------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------------------</td>
</tr>
<tr>
<td>θ</td>
<td>angle measured between the axis of the strut and the axis of a tie passing</td>
<td>12.2</td>
</tr>
<tr>
<td></td>
<td>through a common node</td>
<td></td>
</tr>
<tr>
<td>θ_c</td>
<td>angle between the axis of the concrete compression strut and the longitudinal</td>
<td>8.2.10 and 8.3.5</td>
</tr>
<tr>
<td></td>
<td>axis of the member</td>
<td></td>
</tr>
<tr>
<td>λ</td>
<td>a factor</td>
<td>13.1, 2.3</td>
</tr>
<tr>
<td>λ_{uc}</td>
<td>ratio of the elastic critical buckling load of the entire frame to the design</td>
<td>10.4.3</td>
</tr>
<tr>
<td></td>
<td>load for strength</td>
<td></td>
</tr>
<tr>
<td>μ</td>
<td>friction curvature coefficient; or coefficient of friction</td>
<td>6.4.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8.4.3</td>
</tr>
<tr>
<td>ν</td>
<td>Poisson’s ratio of concrete in kilograms per metre cubed</td>
<td>6.1.5</td>
</tr>
<tr>
<td>ρ</td>
<td>density of concrete</td>
<td>6.1.3</td>
</tr>
<tr>
<td>ρ_p</td>
<td>transverse compressive pressure, in megapascals, at the ultimate limit state</td>
<td>13.1.2.3</td>
</tr>
<tr>
<td></td>
<td>along the development length perpendicular to the plane of splitting</td>
<td></td>
</tr>
<tr>
<td>σ_{ea}</td>
<td>stress on a cross-section at the inner end of an anchorage zone caused by</td>
<td>Figure G1</td>
</tr>
<tr>
<td></td>
<td>prestress</td>
<td></td>
</tr>
<tr>
<td>σ_{ci}</td>
<td>sustained stress in the concrete at the level of the centroid of the</td>
<td>6.4.3.3</td>
</tr>
<tr>
<td></td>
<td>tendons, calculated using the initial prestressing force prior to any time-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>dependent losses and the sustained portions of all the service loads</td>
<td></td>
</tr>
<tr>
<td>σ_{cp}</td>
<td>average intensity of effective prestress in concrete</td>
<td>8.3.5</td>
</tr>
<tr>
<td>σ_{cp,t}</td>
<td>compressive stress as a result of prestress at the extreme fibre where</td>
<td>8.2.7.2</td>
</tr>
<tr>
<td></td>
<td>cracking occurs</td>
<td></td>
</tr>
<tr>
<td>σ_{cs}</td>
<td>maximum shrinkage-induced tensile stress on the uncracked section at the</td>
<td>8.5.3.1</td>
</tr>
<tr>
<td></td>
<td>extreme fibre at which cracking occurs</td>
<td></td>
</tr>
<tr>
<td>σ_o</td>
<td>a constant sustained stress</td>
<td>6.1.8.1</td>
</tr>
<tr>
<td>σ_pa</td>
<td>stress in the tendon at a distance (a) measured from the jacking end</td>
<td>6.4.2</td>
</tr>
<tr>
<td>σ_{pef}</td>
<td>effective stress in the tendon (after losses)</td>
<td>8.1.6</td>
</tr>
<tr>
<td>σ_{pi}</td>
<td>stress in the tendon immediately after transfer</td>
<td>6.4.3.4</td>
</tr>
<tr>
<td>σ_{pj}</td>
<td>stress in the tendon at the jacking end</td>
<td>6.4.2</td>
</tr>
<tr>
<td>σ_{pu}</td>
<td>stress in the tendons at the ultimate limit state</td>
<td>8.1.5</td>
</tr>
<tr>
<td>σ_{sc}</td>
<td>a compressive stress being developed in a bar in compression</td>
<td>13.1.5.4</td>
</tr>
<tr>
<td>σ_{scf}</td>
<td>tensile steel stress at the serviceability limit state for a beam in flexure</td>
<td>8.6.1 and 9.4.1</td>
</tr>
<tr>
<td></td>
<td>or in tension or for a slab in flexure</td>
<td></td>
</tr>
<tr>
<td>σ_{scf,t}</td>
<td>tensile stress in reinforcement at a cracked section, due to the short-term</td>
<td>8.6.1</td>
</tr>
<tr>
<td></td>
<td>load combination for the serviceability limit states, calculated with ( \psi_s = 1.0 ), when</td>
<td></td>
</tr>
<tr>
<td></td>
<td>direct loads are applied</td>
<td></td>
</tr>
<tr>
<td>σ_{si}</td>
<td>tensile stress in reinforcement used to calculate the development length</td>
<td>13.1.2.4</td>
</tr>
<tr>
<td>r^*</td>
<td>design shear stress acting on the interface</td>
<td>8.4.2</td>
</tr>
<tr>
<td>r_a</td>
<td>unit shear strength</td>
<td>8.4.3</td>
</tr>
<tr>
<td>φ</td>
<td>capacity reduction factor</td>
<td>2.2</td>
</tr>
<tr>
<td>ϕ_{cc,j}</td>
<td>residual creep coefficient of concrete</td>
<td>Paragraph E3.2.2,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>App. E</td>
</tr>
<tr>
<td>ϕ_{cs,j}</td>
<td>differential shrinkage</td>
<td>Paragraph E3.2.3,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>App. E</td>
</tr>
<tr>
<td>ϕ_s</td>
<td>correlation factor; or stress reduction factor for design using linear stress</td>
<td>10.4.3</td>
</tr>
<tr>
<td></td>
<td>analysis</td>
<td>2.2.3</td>
</tr>
<tr>
<td>ϕ_m</td>
<td>strength reduction factor for design using strut-and-tie analysis</td>
<td>Appendix I</td>
</tr>
</tbody>
</table>
### TABLE 1.4 (continued)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Clause reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_{cc}$</td>
<td>design creep coefficient at any time $t$</td>
<td>6.1.8</td>
</tr>
<tr>
<td>$\phi^*_{cc}$</td>
<td>final design creep coefficient</td>
<td>6.1.8</td>
</tr>
<tr>
<td>$\phi_{cc,b}$</td>
<td>basic creep coefficient of concrete</td>
<td>6.1.8</td>
</tr>
<tr>
<td>$\psi$</td>
<td>short-term factor</td>
<td>1.3.14</td>
</tr>
<tr>
<td>$\psi_{f}$</td>
<td>factor of safety against lateral buckling of a beam</td>
<td>Appendix C</td>
</tr>
<tr>
<td>$\Omega$</td>
<td>dimension of node</td>
<td>12.4(A)</td>
</tr>
</tbody>
</table>

### 1.5 USE OF ALTERNATIVE MATERIALS OR METHODS

#### 1.5.1 General

Provided that the requirements of Section 2 are met, this Standard shall not be interpreted so as to prevent the use of materials or methods of design, or construction not specifically referred to herein.

**NOTE:** Where the intended use is subject to the control of an authority, approval for the use of alternative materials or methods will need to be obtained from the authority.

#### 1.5.2 Use of other materials or methods

Where new materials, methods of design or construction are used, the designer shall satisfy the authority as to the relevance of the provisions of this Clause to those materials or methods.

#### 1.5.3 Existing structures

Where the strength or serviceability of a new member or prototype is to be tested, the actual geometry, section sizes and condition of the member, as well as the material properties of the member shall be used (see also Section 17).

Where the strength or serviceability of an existing structure is to be evaluated, the provisions of AS 5100.7 shall be used.

#### 1.5.4 Lightweight structural concrete

The design requirements for lightweight structural concrete are not fully covered by this Standard and if lightweight concrete is used, the design requirements for the material shall be specified by the authority.

### 1.6 DESIGN

#### 1.6.1 Design data

The following design data shall be shown in the drawings, in addition to the data specified in AS 5100.2:

(a) Exposure classification for durability.

(b) Class and, where appropriate, grade designation of concrete.

(c) Grade and type of reinforcement and tendons.

#### 1.6.2 Design details

(a) The drawings or specification for concrete members and structures shall include, as appropriate, the following:

(b) The shape and size of each member.

(c) The finish and method of control for unformed surfaces.

(d) Class of formwork for the surface finish specified.
(e) The size, quantity and location of all reinforcement, tendons and structural fixings and the cover to each.

(f) Any required properties of the concrete.

(g) The curing procedure.

(h) The force required in each tendon, the maximum jacking force to be applied and the order in which tendons are to be stressed.

(i) The minimum strength the concrete has to obtain before the application of prestressing forces.

(j) The location and details of planned construction or movement joints, connections and splices, and the method to be used for their protection.

(k) The minimum period before stripping of forms and removal of shores.

(l) Any constraint on construction assumed in the design including, where relevant, the casting procedure.

(m) Any other requirements.

### 1.7 MATERIALS AND CONSTRUCTION REQUIREMENT

The requirements of this Standard are such that the standards of materials and construction achieved for workmanship shall be not less than and the tolerances not greater than those set out in Section 16.
SECTION 2 DESIGN REQUIREMENTS AND PROCEDURES

2.1 DESIGN REQUIREMENTS

2.1.1 Aim

The aim of structural design shall be to provide a structure that does not reach any of the limit states defined in AS 5100.1, which requires that the structure be durable, serviceable and be adequately strong while serving its intended function. The structure shall also satisfy other relevant requirements, such as robustness, ease of construction and economy.

2.1.2 Design for ultimate limit states

The structure, as a whole, and its components shall be designed for the requirements of all the ultimate limit states specified in AS 5100.1.

2.1.3 Design for serviceability limit states

The structure and its components shall be designed for serviceability by controlling deflection, cracking and vibration, as appropriate, in accordance with the relevant requirements of Clauses 2.7 to 2.9.

2.2 STRENGTH

2.2.1 General

Strength checks for concrete structures and their component members shall be carried out using the procedures specified in Clauses 2.2.2 to 2.2.4, and methods of structural analysis specified in Section 7, as appropriate to the strength check procedures being used.

It shall be permissible to use different strength check procedures for different members in a structure, and for the structure as a whole, provided it can be shown that all external actions and forces and calculated internal stress resultants are consistent with the requirements of equilibrium and compatibility for the entire structure.

The design of segmental bridges shall be in accordance with Appendix B.

2.2.2 Strength check procedure for use with methods of analysis other than linear elastic stress analysis and strut-and-tie analysis

The strength check procedure for use in conjunction with—

(a) linear elastic methods of analysis of indeterminate structures and members; or

(b) static analysis of determinate structures,

shall be carried out as follows:

(i) It shall be confirmed that the design capacity is equal to or greater than the design action effect, for all critical cross-sections and regions—

\[ R_d \geq E_d \] . . . 2.2.2

where

\[ R_d = \text{design capacity (equal to } \phi R_u \) \]

\[ E_d = \text{design action effect} \]

(ii) The design capacity, \( R_d = \phi R_u \), shall be obtained using the appropriate capacity reduction factor (\( \phi \)), given in Table 2.2.2, and the ultimate strength (\( R_u \)), determined in accordance with the relevant sections of this Standard using characteristic values for the material strengths.
(iii) The design action effect \( (E_d) \), shall be determined for the critical combination of factored actions specified in AS 5100.1 by one of the following methods of analysis:

(A) Linear elastic analysis in accordance with Clause 7.2.

(B) Linear elastic analysis incorporating secondary bending moments due to lateral joint displacement in accordance with Clause 7.3.

(D) Equilibrium analysis of a statically determinate structure.

### TABLE 2.2.2

**CAPACITY REDUCTION FACTORS \( (\phi) \)**

<table>
<thead>
<tr>
<th>Type of action effect</th>
<th>Capacity reduction factor ( (\phi) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Axial force without bending:</td>
<td></td>
</tr>
<tr>
<td>(i) Tension</td>
<td>0.8</td>
</tr>
<tr>
<td>(ii) Compression</td>
<td>0.6</td>
</tr>
<tr>
<td>(b) Bending without axial tension or compression—</td>
<td></td>
</tr>
<tr>
<td>(i) for members with Class N reinforcement only</td>
<td>( 0.6 \leq (1.19 - 13kuo/12) \leq 0.8 )</td>
</tr>
<tr>
<td>(ii) for members with Class L reinforcement</td>
<td>( 0.6 \leq (1.19 - 13kuo/12) \leq 0.64 )</td>
</tr>
<tr>
<td>(c) Bending with axial tension</td>
<td>( \phi + [(0.8-\phi)(Nu/Nuot)] ) and ( \phi ) is obtained from Item (b)</td>
</tr>
<tr>
<td>(d) Bending with axial compression, where—</td>
<td></td>
</tr>
<tr>
<td>(i) ( Nu \geq Nu_{ab} )</td>
<td>0.6</td>
</tr>
<tr>
<td>(ii) ( Nu &lt; Nu_{ab} )</td>
<td>( 0.6 + [(\phi-0.6)(1-Nu/Nu_{ab})] ) ( \phi ) is obtained from Item (b)</td>
</tr>
<tr>
<td>(e) Shear</td>
<td>0.7</td>
</tr>
<tr>
<td>(f) Torsion</td>
<td>0.7</td>
</tr>
<tr>
<td>(g) Bearing</td>
<td>0.6</td>
</tr>
<tr>
<td>(h) Bending, shear and compression in plain concrete</td>
<td>0.6</td>
</tr>
<tr>
<td>(i) Bending, shear and tension in fixings</td>
<td>0.6</td>
</tr>
</tbody>
</table>

#### 2.2.3 Strength check procedure for use with linear elastic stress (e.g., finite element) analysis

The strength check procedure for use with a linear elastic stress (e.g., finite element) analysis of a structure or member shall be made as follows:

(a) The structure or member shall be analysed for the critical combination of factored actions, as specified in AS 5100.1, by linear stress analysis, in accordance with Clause 7.4, assuming the concrete to be uncracked, and using accepted principles of mechanics.

(b) The calculated principal compressive stresses shall not exceed the following value:

\[
\phi_s \beta 0.9 f'_c
\]

where

- \( \phi_s \) = stress reduction factor with values taken from Table 2.2.3
- \( \beta \) = an effective compressive strength factor, to be evaluated as follows:

(i) In regions not containing effective confining reinforcement—
\[ \beta = 1.0 \text{ when the principal tensile stress does not exceed } f_u', \text{ otherwise } \beta = 0.6 \]

(ii) In regions where effective confining reinforcement is provided, \( \beta \) shall be evaluated by rational calculation taking account of the amount of confining steel and the details used, but shall not exceed 2.

(c) Reinforcement and/or tendons shall be provided to carry all of the internal tensile forces, with stresses not exceeding \( \phi_s f_{sy} \) and \( \phi_s f_{py} \) respectively, where values for the stress reduction factor (\( \phi_s \)) are in accordance with Table 2.2.3.

(d) In determining the areas of steel reinforcement, it shall be permissible to reduce the peak stresses by averaging the stresses over an area appropriate to the size of the member.

(e) The stress development of the reinforcement and tendons shall be determined in accordance with Clauses 13.1 and 13.3 respectively.

\[ \begin{array}{|c|c|} 
\hline 
\text{Material} & \text{Stress reduction factor } (\phi_s) \\
\hline 
\text{Concrete in compression} & 0.6 \\
\text{Steel in tension} & \\
\quad \text{Class N} & 0.8 \\
\quad \text{Class L} & 0.64 \\
\quad \text{Tendons} & 0.8 \\
\hline 
\end{array} \]

2.2.4 Strength check procedure for use with strut-and-tie analysis

The strength check procedure for use with strut-and-tie analysis shall be carried out as follows:

(a) The strut-and-tie model shall satisfy the requirements of Appendix I.

(b) The forces acting on all struts and ties and nodes shall be determined for the critical combination of factored actions as specified in AS 5100.1 by an analysis of the strut-and-tie model in accordance with Appendix I.

(c) The compressive force in any concrete strut shall not exceed the design strength of that strut determined in accordance with Appendix I2.3. The strength reduction factor (\( \phi_{st} \)) to be used in determining the design strength shall be in accordance with Table 2.2.4.

(d) The tensile force in any tie shall not exceed the design strength of the tie determined in accordance with Appendix I3 where the strength reduction factor (\( \phi_{st} \)) is given in Table 2.2.4.

(e) The reinforcement and/or tendons in the ties shall be anchored in accordance with Appendix I3.

(f) The design strength of nodes shall be calculated in accordance with Appendix I4.2 and shall not be exceeded. The strength reduction factor (\( \phi_{st} \)) shall be in accordance with Table 2.2.4.

\[ \begin{array}{|c|c|} 
\hline 
\text{Material} & \text{Strength reduction factor } (\phi_{st}) \\
\hline 
\text{Concrete in compression} & 0.6 \\
\text{Steel in tension} & 0.8 \\
\hline 
\end{array} \]

2.3 DURABILITY

The structure and its components shall be designed for durability in accordance with Section 4.
2.4 FIRE RESISTANCE

Where it is considered necessary for a bridge or part thereof to be designed for fire resistance, the requirements of Section 5 shall apply.

2.5 FATIGUE

2.5.1 General

Fatigue shall be considered where relevant and, if significant, shall be taken into account in the design of the structure. Fatigue shall always be considered in the design of concrete railway bridges, but need not be considered in the design of concrete road bridges where the effective number of stress cycles is less than 500,000.

The fatigue loadings to be used and the number of stress cycles shall be determined in accordance with AS 5100.2.

Fatigue analysis shall be in accordance with the elastic methods specified in Clause 7.2, but moment redistribution shall not be used.

2.5.2 Maximum concrete compressive stresses

The maximum concrete compressive stress under the fatigue design loading specified in AS 5100.2 shall be limited to the smaller of $0.45f'_c$ and 18 MPa.

2.5.3 Shear limited by web compressive stresses

The maximum concrete compressive stresses in the webs of flexural members under the fatigue design loading shall be not greater than 0.60 times the value of $V_{u,\text{max}}$ specified in Clause 8.2.6.

2.5.4 Shear in slabs

The maximum shear force in concrete slabs, as determined in accordance with Clause 8.2.4 under the fatigue design loading, shall be limited to the values specified in this Clause.

Where the slab can act as a wide beam and a shear failure could occur across a substantial width, the maximum calculated shear shall be limited to 0.60 times the value of $V_{uc}$ specified in Clause 9.2.1(a) or Clause 8.2.7. For slabs where the effective number of stress cycles is greater than 2,000,000, the maximum calculated shear shall be limited to 0.54 times the value of $V_{uc}$ specified in Clause 9.2.1(a) or Clause 8.2.7. If the percentage of longitudinal tensile reinforcement $100A_{st}/bd_o$ is less than 1, the permissible shear shall be reduced by multiplying the permissible value of $V_{uc}$ by the factor $(100A_{st}/bd_o)^{1/3}$, where $b$ is the width of the cross-section and $d_o$ is the distance from the extreme compressive fibre of the concrete to the centroid of the outermost layer of tensile reinforcement or tendons but not less than 0.8$D$.

Where the potential failure surface could form a truncated cone or pyramid around a support or loaded area, the maximum calculated shear shall be limited to 0.50 times the value of $V_{uo}$ specified in Clause 9.2.3.

2.5.5 Tensile stress range in steel

The design stress ranges in the prestressing and reinforcing steel in the concrete, determined in accordance with AS 5100.2, shall be limited to the appropriate values given in Table 2.5.5. These stress ranges are applicable for 2,000,000 stress cycles.

To account for the design number of stress cycles ($n$), determined from AS 5100.2, the values given in Table 2.5.5 shall be multiplied by the stress range factor $\alpha_f$ —

where

$$\alpha_f = \left(\frac{2 \times 10^6}{n}\right)^{1/3} \geq 0.74$$

\ldots 2.5.5
The welding of reinforcement in areas of high fluctuating stresses, such as in deck slabs, shall be in accordance with the fatigue requirements of AS 5100.6, and welded lap splices shall not be used.

2.5.6 **Calculation of stresses in the reinforcement of flexural members**

When assessing steel stresses for fatigue in flexural members with shear reinforcement, the stress variations in both the longitudinal and shear reinforcement shall be calculated using the truss analogy method, assuming that all the shear is carried by the reinforcement. The angle between the compression struts and the axis of the member shall be chosen to be between 35° and 55°, except that for non-prestressed slabs and trough girders the angle shall be between 40° and 55°.

2.6 **DESIGN FOR STABILITY**

The structure as a whole and its parts shall be designed to maintain stability against sliding, overturning and uplift (see AS 5100.1).

### TABLE 2.5.5
**PERMISSIBLE STRESS RANGES IN STEEL**

<table>
<thead>
<tr>
<th>Steel type</th>
<th>Steel embedded in concrete (ordinary reinforcement or pretensioned wire or strand), or prestressing tendons in suitable grouted plastic ducts</th>
<th>Prestressing steel in grouted steel ducts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing</td>
<td>150</td>
<td>Not applicable</td>
</tr>
<tr>
<td>Steel wire</td>
<td>150</td>
<td>100</td>
</tr>
<tr>
<td>7 Wire strand</td>
<td>150</td>
<td>100</td>
</tr>
<tr>
<td>Stress bar</td>
<td>Not applicable</td>
<td>100</td>
</tr>
<tr>
<td>Deflected pretensioned strand</td>
<td>70</td>
<td>Not applicable</td>
</tr>
</tbody>
</table>

2.7 **DEFLECTIONS OF BEAMS AND SLABS**

The deflection of beams, box girders and slabs under service conditions shall comply with the deflections limits as specified in AS 5100.2. Deflections shall be calculated in accordance with Clause 8.5 or Clause 9.3, as appropriate.

2.8 **CRACKING**

The cracking of beams or slabs under service conditions shall be controlled in accordance with Clause 8.6 or Clause 9.4, as appropriate.

Where considered necessary for durability requirements, e.g., for exposure classifications B2 or more severe, or where crack width is considered detrimental to the appearance of the structure, consideration shall be given to limiting the steel stresses near the tension face to values less than those given in this Standard. In addition, in such conditions consideration shall be given to the detailing of the structure to minimize cracking due to restraint and shrinkage.

All concrete members shall be provided with a minimum of reinforcement as follows:

(a) For members with a thickness of 150 mm or less, a single layer of reinforcement of not less than 500 mm² per metre for each of two directions, at right angles to each other.

Reinforcement provided for structural reasons can be considered as contributing towards this requirement.
(b) For members with a thickness greater than 150 mm, each face of the member shall be reinforced with not less than 500 mm² per metre for each of two directions at right angles to each other. The layers shall be placed as close to each surface as cover and detailing permit.

Reinforcement provided for structural reasons and located within 80 mm of the face can be considered as contributing towards this requirement.

Reinforcement shall be provided in two directions at right angles to each other and with a spacing that is less than or equal to 300 mm.

The requirements of this Clause in relation to the quantity of reinforcement in a particular direction may be relaxed if the member is unrestrained against longitudinal movement in that direction and the effects of differential temperature and humidity are minimal.

2.9 VIBRATION

The vibration of beams or slabs under service conditions shall comply with Clause 8.7 or Clause 9.5, as appropriate.

2.10 DESIGN FOR STRENGTH AND SERVICEABILITY BY PROTOTYPE TESTING

Notwithstanding the requirements of Clause 2.2, a structure or a component may be designed for strength by testing a prototype in accordance with Clause 17.2.

If this alternative procedure is adopted, the requirements of Clauses 2.7 to 2.9 shall apply, as appropriate.

2.11 OTHER DESIGN REQUIREMENTS

Requirements, such as progressive collapse and any special performance requirements, shall be considered where relevant and, if significant, shall be taken into account in the design of the structure in accordance with the principles of this Standard and appropriate engineering principles.

The use of reinforcing steels complying with AS/NZS 4671 having a yield strength \( f_{y} \) of 500 MPa and Ductility Class E shall be considered for members and structures requiring increased ductility to satisfy seismic design requirements. Grade 500E reinforcement shall not be subjected to welding or heating.

Beam stability during lifting and erection shall be in accordance with Appendix C.
SECTION 3 LOADS AND LOAD COMBINATIONS FOR STABILITY, STRENGTH AND SERVICEABILITY

3.1 LOADS AND OTHER ACTIONS
The design of a structure for strength, stability and serviceability shall take account of the action effects directly arising from the loads, forces and effects set out in AS 5100.2.

3.2 LOAD COMBINATIONS
The design loads for strength, stability and serviceability shall be the combination of factored loads set out in AS 5100.2.
SECTION 4 DESIGN FOR DURABILITY

4.1 APPLICATION
This Section sets out requirements for plain, reinforced and prestressed concrete structures and members with a design life of 100 years (see AS 5100.1).

This Section includes specific requirements for exposure classification, concrete quality and concrete cover to reinforcement and tendons.

For structures with design lives of 40 to 60 years, the durability requirements of AS 3600 may be adopted.

NOTES:
1 Some relaxation of the requirements may be acceptable for temporary structures.
2 Durability is a complex topic and compliance with these requirements may not be sufficient to ensure a durable structure. AS 5100.5 Supp 1 (Commentary to this Standard) contains background to and guidance on the provisions of this Clause.

4.2 DESIGN FOR DURABILITY

4.2.1 General
Durability shall be allowed for in design by determining the exposure classification in accordance with Clause 4.3 and, for that exposure classification, complying with the appropriate requirements for—
(a) concrete quality and curing, in accordance with Clauses 4.4 to 4.6;
(b) chemical content restrictions, in accordance with Clause 4.9; and
(c) cover, in accordance with Clause 4.10.

4.2.2 Additional requirements
In addition to the requirements specified in Clause 4.2.1, the following shall apply:
(a) Members subject to abrasion from traffic, e.g., pavements and bridge decks, shall satisfy Clause 4.7.
(b) Members subject to cycles of freezing and thawing shall satisfy Clause 4.8.

4.3 EXPOSURE CLASSIFICATION
The exposure classification for a surface of a member shall be as given in Table 4.3 and as shown in Figure 4.3.

For determining concrete quality in accordance with Clauses 4.4 to 4.6, and Clause 4.9 as appropriate, the exposure classification for the member shall be taken as the most severe exposure of any of its surfaces.

For determining cover requirements for corrosion protection in accordance with Clause 4.10.3, the exposure classification shall be taken as the classification for the surface from which the cover is measured.

NOTE: In Table 4.3, classifications A, B1, B2 and C represent increasing degrees of severity of exposure, while classification U represents an exposure environment not given in the Table but for which a degree of severity of exposure should be appropriately assessed. Protective surface coatings may be taken into account in the assessment of the exposure classification.
### TABLE 4.3

EXPOSURE CLASSIFICATIONS

<table>
<thead>
<tr>
<th>Surface and exposure environment</th>
<th>Exposure classification</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1</strong> Surface of members in contact with the ground (see Note 1):</td>
<td></td>
</tr>
<tr>
<td>(a) Members in non-aggressive soil (see Note 2)</td>
<td>B1</td>
</tr>
<tr>
<td>(b) Members protected by a damp-proof membrane</td>
<td>A</td>
</tr>
<tr>
<td>(c) Members in aggressive soils (see Note 3)</td>
<td>U</td>
</tr>
<tr>
<td>(d) Members in salt-rich arid areas</td>
<td>C</td>
</tr>
<tr>
<td><strong>2</strong> Surfaces of members in interior environments:</td>
<td></td>
</tr>
<tr>
<td>Fully enclosed within a structure except for a brief period of weather exposure during construction</td>
<td>A</td>
</tr>
<tr>
<td><strong>3</strong> Surfaces of members in above-ground exterior environments in the following areas:</td>
<td></td>
</tr>
<tr>
<td>(a) Inland (greater than 50 km from coastline) environment being—</td>
<td></td>
</tr>
<tr>
<td>(i) non-industrial and arid climatic zone (see Notes 4 and 5);</td>
<td>A</td>
</tr>
<tr>
<td>(ii) non-industrial and temperate climatic zone;</td>
<td>A</td>
</tr>
<tr>
<td>(iii) non-industrial and tropical climatic zone; or</td>
<td>B1</td>
</tr>
<tr>
<td>(iv) industrial and any climatic zone.</td>
<td>B1</td>
</tr>
<tr>
<td>(b) Near-coastal (1 km to 50 km from coastline), any climatic zone</td>
<td>B1</td>
</tr>
<tr>
<td>(c) Coastal (up to 1 km from coastline but excluding tidal and splash zones) (see Note 6), any climatic zone</td>
<td>B2</td>
</tr>
<tr>
<td><strong>4</strong> Surfaces of members in water (see Note 1):</td>
<td></td>
</tr>
<tr>
<td>(a) In fresh water</td>
<td>B1</td>
</tr>
<tr>
<td>(b) In sea water</td>
<td></td>
</tr>
<tr>
<td>(i) Permanently submerged</td>
<td>B2</td>
</tr>
<tr>
<td>(ii) In tidal or splash zones</td>
<td>C</td>
</tr>
<tr>
<td>(c) In soft or running water</td>
<td>U</td>
</tr>
<tr>
<td><strong>5</strong> Surfaces of members in other environments:</td>
<td></td>
</tr>
<tr>
<td>Any exposure environment not described in Items 1 to 4 above</td>
<td>U</td>
</tr>
</tbody>
</table>

**NOTES:**

1. Members, such as piles without permanent steel casing, shall be classified as members in water unless it is proved by geotechnical investigation that no part of the member is below the permanent water table level.

2. If testing has been undertaken to ascertain that the soil in contact with concrete is non-aggressive, then exposure classification A may be used, provided that the soil is not subject to wetting and drying. Typically, members in the top 500 mm of soil would not qualify for this reduction.

3. Permeable soils with a pH less than 4.0 or with ground water containing more than 1 g per litre of sulfate ions, would be considered aggressive.

4. The climatic zones referred to are those shown in Figure 4.3, which is a simplified version of Plate 8 of the Bureau of Meteorology publication *Climate in Australia*, 1982 Edition.

5. Industrial refers to areas that are within 3 km of industries that discharge atmospheric pollutants.

6. For the purpose of this Table, the coastal zone includes locations within 1 km of the shoreline of large expanses of salt water, e.g., Port Phillip, Sydney Harbour east of the Spit Bridge and Harbour Bridge, Swan River west of the Narrows Bridge. Where there are strong prevailing winds or vigorous surf, the distance should be increased beyond 1 km and higher levels of protection should be considered. Proximity to small saltwater bays, estuaries and rivers may be disregarded, except for structures immediately over or adjacent to such bodies of water.
4.4 MEMBERS NOT CONTAINING MATERIAL REQUIRING PROTECTION

Members not containing material requiring protection shall be initially cured continuously for at least three days under ambient conditions.

4.5 EXPOSURE CLASSIFICATIONS A, B1, B2 AND C

Members subject to exposure classification A, B1, B2 or C shall be initially cured continuously for at least 7 days under ambient conditions, or cured by accelerated methods so that the average compressive strength of the concrete at the completion of the accelerated curing is not less than the appropriate value given in Table 4.5.

Concrete in the member shall have a compressive strength \( f'_c \) not less than the appropriate value given in Table 4.5.

For exposure classification B1, B2 and C additional parameters shall be specified, namely, cement type, with slag blended cement for exposure classification C and fly ash blended cement for exposure classification B2 to be used, cement content, total water to cement ratio and type of cement, taking account of the nature and severity of the exposure. This requires that special class of concrete be specified.
If the concrete strength for classification C cannot be satisfied because of inadequate aggregate strength, concrete with $f'_c$ not less than 40 MPa may be used for exposure classification C, provided that the cement content of the mix shall be not less than 420 kg/m$^3$ and the covers as specified in Clause 4.10.3 shall be increased by 10 mm.

### TABLE 4.5

<table>
<thead>
<tr>
<th>Exposure classification</th>
<th>Compressive strength of concrete at the completion of accelerated curing MPa</th>
<th>Minimum characteristic strength, $f'_c$ MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>15</td>
<td>25</td>
</tr>
<tr>
<td>B1</td>
<td>20</td>
<td>32</td>
</tr>
<tr>
<td>B2</td>
<td>25</td>
<td>40</td>
</tr>
<tr>
<td>C</td>
<td>32</td>
<td>50</td>
</tr>
</tbody>
</table>

### 4.6 EXPOSURE CLASSIFICATION U

Concrete in members subject to exposure classification U shall be specified to ensure durability under the particular exposure environment. Consideration shall be given to the suitability of concrete materials, mix proportions, methods of placement, cover and curing. Consideration shall also be given to the possible use of protective surface coatings to the member.

### 4.7 ABRASION

In addition to the other durability requirements, concrete for members exposed to abrasion from traffic shall have a characteristic compressive strength not less than the applicable value given in Table 4.7.

### TABLE 4.7

<table>
<thead>
<tr>
<th>Member or traffic, or both</th>
<th>Minimum characteristic compressive strength ($f'_c$) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footpaths and cycleways</td>
<td>25</td>
</tr>
<tr>
<td>Pavement and bridge decks subject to the following:</td>
<td></td>
</tr>
<tr>
<td>(a) Light pneumatic-tyred traffic (vehicles up to 3 t gross mass)</td>
<td>25</td>
</tr>
<tr>
<td>(b) Medium or heavy pneumatic-tyred traffic (vehicles heavier than 3 t gross mass)</td>
<td>32</td>
</tr>
<tr>
<td>(c) Non pneumatic-tyred traffic</td>
<td>40</td>
</tr>
<tr>
<td>(d) Steel-wheeled traffic</td>
<td>To be assessed but not less than 40</td>
</tr>
</tbody>
</table>

NOTE: $f'_c$ refers to the strength of the wearing course.

### 4.8 FREEZING AND THAWING

In addition to the other durability requirements, where the surface exposure includes exposure to cycles of freezing and thawing, concrete in the member shall—

(a) have a characteristic compressive strength ($f'_c$) not less than—

(i) 32 MPa for occasional exposure (less than or equal to 25 cycles per annum);  
(ii) 40 MPa for frequent exposure (greater than 25 cycles per annum); and
(b) contain a percentage of entrained air not outside the following ranges:
   (i) For 10 mm to 20 mm nominal size aggregate 8% to 4%.
   (ii) For 40 mm nominal size aggregate 6% to 3%;
   where the percentage of entrained air is determined in accordance with AS 1012.4.

4.9 CHEMICAL CONTENT IN CONCRETE

4.9.1 Restriction on chloride-ion content for corrosion protection

When determined in accordance with AS 1012.20, the mass of acid-soluble chloride-ion per unit volume of concrete as placed shall be not greater than the values given in Table 4.9.1.

Chloride salts or chemical admixtures containing significant chlorides shall not be used.

<table>
<thead>
<tr>
<th>TABLE 4.9.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAXIMUM VALUES OF ACID-SOLUBLE CHLORIDE-ION CONTENT</td>
</tr>
<tr>
<td>IN CONCRETE AS PLACED</td>
</tr>
<tr>
<td>------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Form of construction</td>
</tr>
<tr>
<td>Concrete not containing material requiring protection</td>
</tr>
<tr>
<td>Reinforced or prestressed concrete, plain concrete containing material</td>
</tr>
<tr>
<td>requiring protection</td>
</tr>
<tr>
<td>0.8</td>
</tr>
</tbody>
</table>

4.9.2 Restriction on sulfate content

When determined in accordance with AS 1012.20, the sulfate content of concrete as placed, expressed as the percentage by mass of acid-soluble SO₃ to cement, shall not be greater than 5.0%.

4.9.3 Restriction on other salts

Other strongly ionized salts, such as nitrates, shall not be added to concrete unless it can be shown that they do not adversely affect durability.

4.10 COVER TO REINFORCING STEEL AND TENDONS

4.10.1 General

The cover to reinforcing steel and tendons shall be the greatest of the values determined in accordance with Clauses 4.10.2 and 4.10.3, as appropriate, unless exceeded by the cover required for fire resistance.

For pre-tensioned systems, the ends of individual pre-tensioned tendons do not normally require concrete cover and shall be cut off flush with the end face of the member. The exposed ends of tendons shall be sealed against corrosion.

4.10.2 Cover for concrete placement

The cover for concrete placement shall be sufficient to ensure that the concrete can be placed and compacted. The cover shall be not less than the greater of the following:

(a) **Cover** 1.5 times the maximum nominal size of the aggregate.

(b) **Diameter** The diameter of the reinforcing bar being protected. For bundled bars, an equivalent diameter shall be taken as twice the diameter of the largest bar in the bundle.

(c) **Pre-tensioned tendons** The clear cover shall be twice the diameter of the tendon but shall be not less than 40 mm.
Where tendons are grouped together, especially in a horizontal plane, the cover shall be increased beyond the above minimum values to facilitate placing and compaction of the concrete.

(d) Post-tensioning ducts The minimum cover shall be the maximum of—

Text deleted

(i) 50 mm from the surface for any duct in the soffit of a member; and
(ii) 40 mm elsewhere.
(iii) Where external tendons are initially located outside the structural concrete and are to be subsequently protected by additional concrete, the minimum cover shall be the same as for tendons embedded in structural concrete.

Where the tendons are grouped together in a horizontal plane or where the ducts are used in thin members, special consideration shall be given to increasing the cover to facilitate the placing and compaction of the concrete.

(e) Post-tensioning anchorages The minimum cover at the ends of post-tensioned tendons or anchorage devices shall be 50 mm.

NOTE: This provision applies only when anchorages are protected by concrete. Where anchorage recesses are filled with other materials, e.g., epoxy resin or epoxy mortar, the minimum cover specified may be reduced at the discretion of the authority. When anchorages are permanently exposed in the finished structure, an appropriate corrosion protection should be specified.

4.10.3 Cover for corrosion protection

4.10.3.1 General

For corrosion protection, the nominal cover shall be not less than the appropriate value specified in Clauses 4.10.3.2 to 4.10.3.5. The nominal covers assume that the fixing tolerance on the specified position of reinforcement and tendons to the nearest surface shall be as follows:

(a) For formed slabs, beams, walls and columns ..................−5, +10 mm.
(b) For slabs on ground ..............................................................−10, +20 mm.
(c) For footings cast in ground and cast-in-place piles without permanent steel casing ..............................................................−20, +40 mm.

A positive value indicates the amount of cover increases and a negative value indicates the amount of cover decreases. Where those tolerances are likely to be exceeded during the construction, the appropriate cover shall be increased appropriately.

4.10.3.2 Standard formwork and compaction

Where concrete is cast in formwork complying with AS 3610 and transported, placed and compacted so as to—

(a) limit segregation or loss of materials;
(b) limit premature stiffening;
(c) produce a monolithic mass between planned joints or the extremities of members, or both;
(d) completely fill the formwork to the intended level, expel entrapped air and closely surround all reinforcement, tendons, ducts, anchorages and embedments; and
(e) provide the specified finish to the formed areas of the concrete; then

the nominal cover shall be not less than the value given in Table 4.10.3(A) as appropriate to the exposure classification and \( f'_c \).
4.10.3.3 Cast against ground

Where concrete is cast on or against excavated ground and compacted in accordance with Clause 4.10.3.2, the cover to a surface in contact with the ground shall be as given in Table 4.10.3(A) but increased by—

(a) 10 mm if the concrete surface is protected by a damp-proof membrane; or
(b) 30 mm otherwise.

4.10.3.4 Rigid formwork and intense compaction

Where concrete is precast in rigid steel forms and subjected to intense compaction, such as obtained with vibrating tables or form vibrators, the cover shall be not less than the values given in Table 4.10.3(B), as appropriate to the exposure classification and $f'_c$.

4.10.3.5 Structural members manufactured by spinning or rolling

Where structural members manufactured by spinning or rolling concrete having a total water to cement ratio less than 0.35, and provided that negative tolerance is not allowed on the fixing of reinforcement, the cover for corrosion protection shall be not less than the values given in Table 4.10.3(C) for the appropriate exposure classification.

**TABLE 4.10.3(A)**

NOMINAL COVER WHERE STANDARD FORMWORK AND COMPACTION ARE USED

<table>
<thead>
<tr>
<th>Exposure classification</th>
<th>25 MPa</th>
<th>32 MPa</th>
<th>40 MPa</th>
<th>≥50 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>35</td>
<td>30</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>B1</td>
<td>—</td>
<td>45</td>
<td>40</td>
<td>35</td>
</tr>
<tr>
<td>B2</td>
<td>—</td>
<td>—</td>
<td>55</td>
<td>45</td>
</tr>
<tr>
<td>C</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>70</td>
</tr>
</tbody>
</table>

NOTE: Increased values are required if Clause 4.10.3(c) applies.

**TABLE 4.10.3(B)**

NOMINAL COVER WHERE RIGID FORMWORK AND INTENSE COMPACTION ARE USED

<table>
<thead>
<tr>
<th>Exposure classification</th>
<th>25 MPa</th>
<th>32 MPa</th>
<th>40 MPa</th>
<th>≥50 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>B1</td>
<td>—</td>
<td>35</td>
<td>30</td>
<td>25</td>
</tr>
<tr>
<td>B2</td>
<td>—</td>
<td>—</td>
<td>45</td>
<td>35</td>
</tr>
<tr>
<td>C</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>50</td>
</tr>
</tbody>
</table>
TABLE 4.10.3(C)  
REQUIRED COVER FOR SPUN OR ROLLED MEMBERS

<table>
<thead>
<tr>
<th>Exposure classification</th>
<th>Concrete characteristic compressive strength ($f'_c$) MPa</th>
<th>Cover mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>A, B1</td>
<td>32</td>
<td>25</td>
</tr>
<tr>
<td>B2</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>≥50</td>
<td>25</td>
</tr>
<tr>
<td>C</td>
<td>≥50</td>
<td>35</td>
</tr>
</tbody>
</table>

4.11 PROVISIONS FOR STRAY CURRENT CORROSION

The effects of possible stray current corrosion shall be considered where necessary. Stray current corrosion is of particular concern for railway bridges carrying electrified railways and tramways, especially where they are powered by direct current.
SECTION 5 DESIGN FOR FIRE RESISTANCE

In instances where it is considered necessary for a bridge, or part thereof, to be designed for fire resistance, the relevant provisions of AS 3600 shall apply.

In instances where it is considered necessary for a bridge, or part thereof, to be designed for resistance to hydrocarbon fires, refer to specialist literature or seek specialist advice from a fire engineer.

NOTES

AS 3600 applied to buildings where the typical design fire is from a non-hydrocarbon fire and hence it is not applicable to many fires that may occur on the road and rail network.

Hydrocarbon fires may reach temperatures in excess of 1000°C almost instantaneously and may damage the surfaces of any concrete members in contact with the fire, with the depth of damage being dependant on the duration of exposure.
SECTION 6 DESIGN PROPERTIES OF MATERIALS

6.1 PROPERTIES OF CONCRETE

6.1.1 Strength

The characteristic strength of concrete shall be determined as follows:

(a) **Characteristic compressive strength** The characteristic compressive strength of concrete at 28 days \( f'_c \) may be either—

(i) taken as equal to the specified strength grade, provided that the appropriate curing is ensured, and that the concrete complies with Section 16; or

(ii) determined statistically from compressive strength tests carried out in accordance with AS 1012.9.

The characteristic compressive strengths of the standard strength grades of concrete are 25 MPa, 32 MPa, 40 MPa, 50 MPa and 65 MPa.

In the absence of more accurate data, the mean value of the in-situ compressive strength \( f_{cm} \) shall be taken as 90% of the mean value of the cylinder strength \( f_{cm} \) or shall be taken as those given in Table 6.1.2.

(b) **Tensile strength** The uniaxial tensile strength \( f_{ct} \) is the maximum stress that concrete can withstand when subjected to uniaxial tension.

The uniaxial tensile strength shall be determined from either the measured flexural strength \( f_{ct,f} \) or from the measured splitting tensile strength \( f_{ct,sp} \) using—

\[
f_{ct} = 0.6 f_{ct,f} \quad \text{or} \quad f_{ct} = 0.9 f_{ct,sp}
\]

where \( f_{ct,f} \) and \( f_{ct,sp} \) are determined statistically from—

(i) flexural strength tests carried out in accordance with AS 1012.11; or

(ii) indirect tensile strength tests carried out in accordance with AS 1012.10, respectively.

In the absence of more accurate data, the characteristic flexural tensile strength of concrete \( f'_{ct,f} \) and the characteristic uniaxial tensile strength of concrete \( f'_{ct} \) shall be taken as—

\[
f'_{ct,f} = 0.6 f'_c \quad \text{and} \quad f'_{ct} = 0.36 f'_c \quad \text{at 28 days and standard curing,}
\]

and where the mean and upper characteristic values are obtained by multiplying these values by 1.4 and 1.8, respectively.

6.1.2 Modulus of elasticity

The mean modulus of elasticity of concrete at the appropriate age \( E_{cj} \) may be either—

(a) taken as equal to—

(i) \( \left( \rho^{1.5} \right) \times \left( 0.043 f_{cm} \right) \) (in megapascals) when \( f_{cm} \leq 40 \text{ MPa} \); or

(ii) \( \left( \rho^{1.5} \right) \times \left( 0.024 f_{cm} + 0.12 \right) \) (in megapascals) when \( f_{cm} > 40 \text{ MPa} \),

with consideration being given to the fact that this value has a range of \( \pm 20\% \); or

(b) determined by test in accordance with AS 1012.17; or
(c) for standard strength grades at 28 days, equal to $E_c$ as determined from Table 6.1.2.

### Table 6.1.2

<table>
<thead>
<tr>
<th>$f'_c$ (MPa)</th>
<th>20</th>
<th>25</th>
<th>32</th>
<th>40</th>
<th>50</th>
<th>65</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{cmi}$ (MPa)</td>
<td>22</td>
<td>28</td>
<td>35</td>
<td>43</td>
<td>53</td>
<td>68</td>
</tr>
<tr>
<td>$E_c$ (MPa)</td>
<td>24 000</td>
<td>26 700</td>
<td>30 100</td>
<td>32 800</td>
<td>34 800</td>
<td>37 400</td>
</tr>
</tbody>
</table>

NOTE: Where a load is sustained and where the concrete deformation is unrestrained such that time-dependent deflections or strains are produced, or where a deformation is sustained so that time-dependent stress reductions occur, the final value of time-dependent effects may be calculated by using an effective modulus of elasticity equal to $E_c$ divided by $(1 + \varphi_{cc})$, where $\varphi_{cc}$ is taken at its final value.

#### 6.1.3 Density

The density of concrete ($\rho$) may be either—
- taken as not less than 2400 kg/m$^3$ for normal weight concrete; or
- determined by test in accordance with AS 1012.12.

#### 6.1.4 Stress-strain curves

A stress-strain curve for concrete may be either—
- assumed to be of curvilinear form defined by recognized simplified equations; or
- determined from suitable test data.

For design purposes, the shape of the curve shall be modified so that the maximum stress equals $0.85f'_c$.

#### 6.1.5 Poisson’s ratio

Poisson’s ratio of concrete ($\nu$) may be either—
- taken as equal to 0.2; or
- determined by test in accordance with AS 1012.17.

#### 6.1.6 Coefficient of thermal expansion

The coefficient of thermal expansion of concrete may be either—
- taken as equal to $11 \times 10^{-6}$ per degree Celsius. Consideration shall be given to the fact that this value has a range of ±20%; or
- determined from suitable test data.

#### 6.1.7 Shrinkage

##### 6.1.7.1 Calculation of design shrinkage strain

The design shrinkage strain of concrete ($\varepsilon_{cs}$) shall be determined—
- (a) from measurements on similar local concrete; or
- (b) by tests after eight weeks of drying modified for long-term value, in accordance with AS 1012.13; or
- (c) by calculation in accordance with Clause 6.1.7.2.

##### 6.1.7.2 Design shrinkage strain

When the design shrinkage strain of concrete ($\varepsilon_{cs}$) is to be calculated, it shall be determined as the sum of the chemical (autogenous) shrinkage strain ($\varepsilon_{cse}$) and the drying shrinkage strain ($\varepsilon_{csd}$)
The autogenous shrinkage strain shall be taken as—

\[ \varepsilon_{cse} = \varepsilon_{cse}^* \times \left(1.0 - e^{-0.1t}\right) \]

... 6.1.7.2(2)

where \( t \) is the time (in days) after setting and \( \varepsilon_{cse}^* \) is the final autogenous shrinkage strain given by—

\[ \varepsilon_{cse}^* = (0.06 f'_c - 1.0) \times 50 \times 10^{-6} \]

... 6.1.7.2(3)

At any time \( t \) (in days) after the commencement of drying, the drying shrinkage strain shall be taken as—

\[ \varepsilon_{csd} = k_1 k_4 \varepsilon_{csd.b} \]

... 6.1.7.2(4)

and \( k_1 \) is obtained from Figure 6.1.7.2 and \( k_4 \) is equal to 0.7 for an arid environment, 0.65 for an interior environment, 0.6 for a temperate inland environment and 0.5 for a tropical or near-coastal environment.

The basic drying shrinkage strain (\( \varepsilon_{csd.b} \)) is given by—

\[ \varepsilon_{csd.b} = (1.0 - 0.008 f'_c) \times \varepsilon_{csd.b}^* \]

... 6.1.7.2(5)

where the final drying basic shrinkage strain (\( \varepsilon_{csd.b}^* \)) depends on the quality of the local aggregates and shall be taken as \( 800 \times 10^{-6} \) for Sydney and Brisbane, \( 900 \times 10^{-6} \) for Melbourne and \( 1000 \times 10^{-6} \) elsewhere.

Based on a value of \( \varepsilon_{csd.b}^* = 1000 \times 10^{-6} \), this method gives the typical design shrinkage strains shown in Table 6.1.7.2.

NOTE: Concrete exposed to early drying undergoes shrinkage due to capillary suction. This can result in cracking and poor service performance, particularly of exposed slabs. The amount of shrinkage from suction depends on the ambient conditions and the concrete mix, and can exceed the combined shrinkage from other causes. Therefore, it is important to prevent excessive drying of concrete between the commencement of casting and the application of curing at the completion of finishing.

Consideration shall be given to the fact that \( \varepsilon_{cs} \) has a range of \( \pm 30\% \).
FIGURE 6.1.7.2 SHRINKAGE STRAIN COEFFICIENT ($k_1$) FOR VARIOUS VALUES OF $t_h$

TABLE 6.1.7.2
TYPICAL FINAL DESIGN SHRINKAGE STRAINS AFTER 30 YEARS

<table>
<thead>
<tr>
<th>$f'_c$ (MPa)</th>
<th>Arid environment</th>
<th>Interior environment</th>
<th>Temperate inland environment</th>
<th>Tropical, near-coastal and coastal environment</th>
</tr>
</thead>
<tbody>
<tr>
<td>t_h (mm)</td>
<td>$t_h$ (mm)</td>
<td>$t_h$ (mm)</td>
<td>$t_h$ (mm)</td>
<td>$t_h$ (mm)</td>
</tr>
<tr>
<td>50</td>
<td>990 870 710 550</td>
<td>920 810 660 510</td>
<td>850 750 610 470</td>
<td>720 630 510 400</td>
</tr>
<tr>
<td>32</td>
<td>950 840 680 530</td>
<td>880 780 640 500</td>
<td>820 720 590 460</td>
<td>690 610 500 390</td>
</tr>
<tr>
<td>40</td>
<td>890 790 650 510</td>
<td>830 740 610 480</td>
<td>780 690 570 450</td>
<td>660 590 490 390</td>
</tr>
<tr>
<td>50</td>
<td>830 740 610 490</td>
<td>770 690 580 460</td>
<td>720 650 540 440</td>
<td>620 550 470 380</td>
</tr>
<tr>
<td>65</td>
<td>730 650 560 460</td>
<td>680 620 530 440</td>
<td>640 580 500 410</td>
<td>560 510 440 370</td>
</tr>
<tr>
<td>80</td>
<td>630 570 500 420</td>
<td>590 540 480 410</td>
<td>560 520 450 390</td>
<td>500 460 410 360</td>
</tr>
<tr>
<td>100</td>
<td>490 460 420 380</td>
<td>480 450 410 370</td>
<td>460 430 400 360</td>
<td>420 400 370 340</td>
</tr>
</tbody>
</table>
6.1.8 Creep

6.1.8.1 General

The creep strain at any time \( t \) caused by a constant sustained stress \( \sigma_o \) shall be calculated from—

\[
\varepsilon_{cc} = \phi_{cc} \sigma / E_c
\]

where

\( E_c \) = mean modulus of elasticity of the concrete at 28 days

\( \phi_{cc} \) = design creep coefficient at time \( t \) determined in accordance with

Clause 6.1.8.3

6.1.8.2 Basic creep coefficient

The basic creep coefficient of concrete \( (\phi_{cc.b}) \) is the mean value of the ratio of final creep strain to elastic strain for a specimen loaded at 28 days under a constant stress of \( f'_c \) and shall be—

(a) determined from measurements on similar local concrete; or

(b) determined by tests in accordance with AS 1012.16; or

(c) taken as the value given in Table 6.1.8.2.

<table>
<thead>
<tr>
<th>Characteristic strength ( (f'_c) ), MPa</th>
<th>20</th>
<th>25</th>
<th>32</th>
<th>40</th>
<th>50</th>
<th>65</th>
<th>80</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic creep coefficient ( (\phi_{cc.b}) )</td>
<td>5.2</td>
<td>4.2</td>
<td>3.4</td>
<td>2.8</td>
<td>2.4</td>
<td>2.0</td>
<td>1.7</td>
<td>1.5</td>
</tr>
</tbody>
</table>

6.1.8.3 Design creep coefficient

The design creep coefficient for concrete at any time, \( t \), \( (\phi_{cc}) \) shall be determined from the basic creep coefficient \( (\phi_{cc.b}) \) by any accepted mathematical model for creep behaviour, calibrated such that \( \phi_{cc.b} \) is also predicted by the chosen model.

In the absence of more accurate methods, \( \phi_{cc} \) at any time shall be taken as—

\[
\frac{\phi_{cc}}{\phi_{cc.b}} = k_2 k_3 k_4 k_5
\]

where \( k_2 \) and \( k_3 \) are obtained from Figure 6.1.8.3(A) and Figure 3.1.8.3(B) respectively; \( k_4 = 0.70 \) for an arid environment, 0.65 for an interior environment, 0.60 for a temperate inland environment and 0.50 for a tropical or near-coastal environment; \( k_5 \) is a modification factor for high strength concrete and shall be taken as—

\[
k_5 = 1.0 \quad \text{when } f'_c \leq 50 \text{ MPa}; \text{ or}
\]

\[
k_5 = (2.0 - \alpha_3) - 0.02(1.0 - \alpha_3) f'_c \quad \text{when } 50 \text{ MPa} < f'_c \leq 100 \text{ MPa};
\]

the factor \( \alpha_3 = 0.7(k_4 \alpha_2); \text{ and } \alpha_2 \text{ is defined in Figure 6.1.8.3(A).}

Consideration shall be given to the fact that \( \phi_{cc} \) has a range of approximately \( \pm 30\% \). This range is likely to be exceeded if—

(a) the concrete member is subjected to prolonged periods of temperature in excess of \( 25^\circ\text{C} \); or

(b) the member is subject to sustained stress levels in excess of \( 0.5 f'_c \).

The final design creep coefficients \( (\phi'_{cc}) \) (after 30 years) predicted by this method for concrete first loaded at 28 days are given in Table 6.1.8.3.
**FIGURE 6.1.8.3(A) COEFFICIENT \( k_2 \)**

\[
k_2 = \frac{\alpha_2 t^{0.8}}{t^{0.8+0.15 t_h}}
\]

\[
\alpha_2 = 1.0 + 1.12 e^{-0.008 t_h}
\]

Where \( t \) is in days

---

**FIGURE 6.1.8.3(B) MATURITY COEFFICIENT \( k_3 \)**

---
TABLE 6.1.8.3

FINAL CREEP COEFFICIENTS (AFTER 30 YEARS) FOR CONCRETE FIRST LOADED AT 28 DAYS

<table>
<thead>
<tr>
<th>$f'_{c}$ (MPa)</th>
<th>Arid environment</th>
<th>Interior environment</th>
<th>Temperate inland environment</th>
<th>Tropical, near-coastal and coastal environment</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_{h}$ (mm)</td>
<td>$t_{h}$ (mm)</td>
<td>$t_{h}$ (mm)</td>
<td>$t_{h}$ (mm)</td>
<td>$t_{h}$ (mm)</td>
</tr>
<tr>
<td>25</td>
<td>4.82</td>
<td>3.90</td>
<td>3.27</td>
<td>4.48</td>
</tr>
<tr>
<td>32</td>
<td>3.90</td>
<td>3.15</td>
<td>2.64</td>
<td>3.62</td>
</tr>
<tr>
<td>40</td>
<td>3.21</td>
<td>2.60</td>
<td>2.18</td>
<td>2.98</td>
</tr>
<tr>
<td>50</td>
<td>2.75</td>
<td>2.23</td>
<td>1.89</td>
<td>2.56</td>
</tr>
<tr>
<td>65</td>
<td>2.07</td>
<td>1.75</td>
<td>1.53</td>
<td>1.95</td>
</tr>
<tr>
<td>80</td>
<td>1.56</td>
<td>1.40</td>
<td>1.29</td>
<td>1.50</td>
</tr>
<tr>
<td>100</td>
<td>1.15</td>
<td>1.14</td>
<td>1.11</td>
<td>1.15</td>
</tr>
</tbody>
</table>

6.2 PROPERTIES OF REINFORCEMENT

6.2.1 Strength and ductility

The yield strength of reinforcement ($f_{sy}$) shall be taken as not greater than the value given in Table 6.2.1 for the appropriate type of reinforcement (see Clause 1.1.2(b)).

The ductility of the reinforcement shall be characterized by its uniform elongation ($\varepsilon_{u}$) and tensile-to-yield stress ratio, and designated as low (L) or normal (N) as given in Table 6.2.1. Values of these parameters for each ductility class shall comply with AS/NZS 4671.

TABLE 6.2.1

STRENGTH AND DUCTILITY OF REINFORCEMENT

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Type</th>
<th>Designation grade</th>
<th>Yield strength ($f_{y}$) MPa</th>
<th>Ductility class (see Clause 6.2.1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bar: Plain (fitments only)</td>
<td>R250N</td>
<td>250</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>Bar: Deformed</td>
<td>D500N</td>
<td>500</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>Bar: Plain, deformed and indented (fitments only)</td>
<td>500L</td>
<td>500</td>
<td>L</td>
<td></td>
</tr>
<tr>
<td>Welded mesh: Plain, deformed and indented</td>
<td>500L</td>
<td>500</td>
<td>L</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: Reference shall be made to AS/NZS 4671 for explanation of designations.

6.2.2 Modulus of elasticity

The modulus of elasticity of reinforcement ($E_{s}$) for all stress values not greater than the yield strength ($f_{sy}$) shall be either—

taken as equal to $200 \times 10^3$ MPa; or
determined by test.

6.2.3 Stress-strain curves

A stress-strain curve for reinforcement shall be either—
assumed to be of a form defined by recognized simplified equations; or determined from suitable test data.

### 6.2.4 Coefficient of thermal expansion

The coefficient of thermal expansion of reinforcement shall be either—
taken as equal to $12 \times 10^{-6}$ per degree Celsius; or determined from suitable test data.

### 6.3 PROPERTIES OF TENDONS

#### 6.3.1 Strength

The following applies:

(a) The characteristic minimum breaking strength ($f_{pb}$) for commonly used tendons shall be as specified in Table 3.3.1. For tendons of dimensions not covered in Clause 3.3, refer to AS/NZS 4672.1.

(b) The yield strength of tendons ($f_{py}$) shall be taken either as the 0.1% proof stress as specified in AS/NZS 4672.1, or determined by test data. In the absence of test data it shall be taken as follows:

(i) For wire used in the as-drawn condition.................................................. $0.80f_{pb}$.  
(ii) For stress-relieved wire........................................................................... $0.83f_{pb}$.  
(iii) For all grades of strand ........................................................................... $0.82f_{pb}$.  
(iv) For hot-rolled bars (super grade)............................................................. $0.81f_{pb}$.  
(v) For hot-rolled ribbed bars ....................................................................... $0.89f_{pb}$.  

#### TABLE 6.3.1

<table>
<thead>
<tr>
<th>Material type and Standard</th>
<th>Nominal diameter</th>
<th>Area</th>
<th>Characteristic minimum breaking load</th>
<th>Characteristic minimum breaking strength ($f_{pb}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>As-drawn wire, AS/NZS 4672.1</td>
<td>5.0</td>
<td>19.6</td>
<td>34.7</td>
<td>1700</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>38.5</td>
<td>64.3</td>
<td>1670</td>
</tr>
<tr>
<td>Stress-relieved wire, AS/NZS 4672.1</td>
<td>5.0</td>
<td>19.9</td>
<td>33.8</td>
<td>1700</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>38.5</td>
<td>64.3</td>
<td>1670</td>
</tr>
<tr>
<td>7 wire ordinary strand, AS/NZS 4672.1</td>
<td>9.5</td>
<td>55.0</td>
<td>102</td>
<td>1850</td>
</tr>
<tr>
<td></td>
<td>12.7</td>
<td>98.6</td>
<td>184</td>
<td>1870</td>
</tr>
<tr>
<td></td>
<td>15.2</td>
<td>140</td>
<td>250</td>
<td>1790</td>
</tr>
<tr>
<td></td>
<td>15.2</td>
<td>143</td>
<td>261</td>
<td>1830</td>
</tr>
<tr>
<td>7 wire compacted strand, AS/NZS 4672.1</td>
<td>15.2</td>
<td>165</td>
<td>300</td>
<td>1820</td>
</tr>
<tr>
<td></td>
<td>18.0</td>
<td>223</td>
<td>380</td>
<td>1700</td>
</tr>
<tr>
<td>Hot-rolled bars, AS/NZS 4672.1 (Super grade only)</td>
<td>26</td>
<td>562</td>
<td>579</td>
<td>1030</td>
</tr>
<tr>
<td></td>
<td>29</td>
<td>693</td>
<td>714</td>
<td>1030</td>
</tr>
<tr>
<td></td>
<td>32</td>
<td>840</td>
<td>865</td>
<td>1030</td>
</tr>
<tr>
<td></td>
<td>36</td>
<td>995</td>
<td>1025</td>
<td>1030</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>1232</td>
<td>1269</td>
<td>1030</td>
</tr>
<tr>
<td></td>
<td>56</td>
<td>2428</td>
<td>2501</td>
<td>1030</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>4371</td>
<td>4502</td>
<td>1030</td>
</tr>
</tbody>
</table>

#### 6.3.2 Modulus of elasticity

The modulus of elasticity of commonly used tendons ($E_p$) shall be either—
(a) taken as equal to—
   (i) for as-drawn wire, to AS/NZS 4672.1 ........................................... 205 ±10 GPa;
   (ii) for stress-relieved steel wire, to AS/NZS 4672.1........................... 205 ±10 GPa;
   (iii) for stress-relieved steel strand, to AS/NZS 4672.1 .......................... 200 ±5 GPa;
   (iv) for hot-rolled high tensile alloy steel bars, to AS/NZS 4672.1 ... 205 ±10 GPa; or
(b) determined by test.

NOTE: Consideration should be given to the fact that the modulus of elasticity of tendons may vary by ±10% and will vary more when a multi-strand or multi-wire tendon is stressed as a single cable. This will influence the calculated extension.

6.3.3 Stress-strain curves
A stress-strain curve for tendons shall be determined from appropriate test data.

6.3.4 Relaxation of tendons

6.3.4.1 General
This Clause applies to the relaxation, at any age and stress level, of low relaxation wire, low relaxation strand, and alloy steel bars. The relaxation of a tendon \( R \) shall be determined in accordance with Clauses 6.3.4.2, 6.3.4.3, and 6.3.4.4.

6.3.4.2 Basic relaxation
The basic relaxation of a tendon \( R_b \) after one thousand hours at 20°C and \( 0.8f_{pb} \) shall be determined in accordance with AS/NZS 4672.1.

6.3.4.3 Design relaxation
Subject to Clause 6.4.3.4, the design relaxation of a tendon \( R \) shall be determined as follows:

\[
R = k_4 k_5 k_6 R_b
\]  

. . . 6.3.4.3(1)

Where

\[ k_4 = \text{a coefficient, dependent on the duration of the prestressing force} \]

\[ = \log[5.4 j^{1/6}] \]  

. . . 6.3.4.3(2)

\[ j = \text{time after prestressing, in days} \]

\[ k_5 = \text{a coefficient, dependent on the stress in the tendon as a proportion of } f_{pb}, \text{ as shown in Figure 6.3.4} \]

\[ k_6 = \text{a function, dependent on the average annual temperature } (T)°C, \text{ taken as } T/20, \text{ but not less than } 1.0 \]

6.3.4.4 Design relaxation for elevated temperature curing
Where curing of a prestressed member is carried out at elevated temperatures, ultimate relaxation shall be deemed to have occurred by the end of the curing cycle. In such cases, the design relaxation shall be taken as either—

the value determined from suitable test data; or
7% to 10% for low relaxation strand stressed to \( 0.8f_{pb} \).

NOTES:
1. For calculation of hogs and transfer stresses without test results, the relaxation should be in the range of 7%, and for calculation of final stress 10%.
2. Relevant information for normal relaxation wire and strand is given in AS 5100.5 Supp 1.
6.4 LOSS OF PRESTRESS IN TENDON

6.4.1 General

The loss of prestress at any given time shall be taken to be the sum of the immediate loss of prestress and the time-dependent loss of prestress, calculated in accordance with Clauses 6.4.2 and 6.4.3 respectively. Values given for estimating the losses apply only to design under normal conditions.

NOTE: Revision of these values will be necessary in the estimation of total loss of prestress for unusual conditions of exposure or where new processes or materials are introduced.

6.4.2 Immediate loss of prestress

The immediate loss of prestress shall be estimated by adding the calculated losses of prestress due to elastic deformation of the concrete, friction, anchoring and other immediate losses as may be applicable. The following loss of prestress shall be determined:

(a) Loss of prestress due to elastic deformation of concrete Calculation of the immediate loss of prestress due to elastic deformation of the concrete at transfer shall be based on the value of modulus of elasticity of the concrete at that age.

(b) Loss of prestress due to friction The stress variation along the design profile of a tendon due to friction in the jack, the anchorage and the duct shall be assessed as follows in order to obtain an estimate of the prestressing forces at the critical sections considered in the design:

(i) The extension of the tendon shall be calculated allowing for the variation in tension along its length.

(ii) The data on friction and slip shall be appropriate to the prestressing system used or based on data derived from tests on that system. In the absence of such data, the following may be used:

(A) Friction in the jack and anchorage The loss of prestress due to friction in the jack and anchorage shall be determined for the type of jack and anchorage system to be used.

(B) Friction along the tendon Friction loss shall be calculated from an analysis of the forces exerted by the tendon on the duct. In the absence of more detailed calculations, the stress in the tendon \( \sigma_{pa} \) at a distance \( a \) measured from the jacking end, may be taken as—

\[
\sigma_{pa} = \sigma_{pj} e^{-\rho(\sigma_{us} + \rho_{j}L_{ys})}
\]

where
\(\sigma_{pj}\) = stress in the tendon at the jacking end
\(e\) = base of Napierian logarithms
\(\mu\) = friction curvature coefficient for different conditions which, in the absence of specific data and when all tendons in contact in one duct are stressed simultaneously, may be taken as—
- 0.14 for polyethylene ducts
- 0.15 for greased and wrapped coating
- 0.15 to 0.20 for bright and zinc coated metal sheathing
- 0.20 for bright and zinc coated flat metal ducts

\(\alpha_{tot}\) = sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length of the tendon from the jacking end to a point at distance \((a)\) from that end \((L_{paa})\)

\(\beta_p\) = estimate in radians per metre of the angular deviation due to wobble effects, which as a first approximation may be taken as follows:
- 0.024 to 0.016 rad/m for sheathing containing tendons other than bars and with an internal diameter of less than 50 mm
- 0.016 to 0.012 rad/m for sheathing containing tendons other than bars and with an internal diameter of 50 to 90 mm
- 0.012 to 0.008 rad/m for sheathing containing tendons other than bars and with an internal diameter of 90 to 140 mm
- 0.024 to 0.016 rad/m for flat metal ducts containing tendons other than bars
- 0.016 to 0.008 rad/m for sheathing containing bars with an internal diameter of 50 mm or less
- 0.008 rad/m for bars of any diameter in a greased and wrapped coating

\(L_{paa}\) = length of the tendon from the jacking end to the point at a distance \((a)\) from that end

The magnitude of the friction due to duct curvature and wobble used in the design shall be verified during the stressing operation.

(c) \textit{Loss of prestress during anchoring} In a post-tensioned member, allowance shall be made for loss of prestress when the prestressing force is transferred from the tensioning equipment to the anchorage. The value of anchorage seating used in the design shall be stated on the drawings.

NOTES:
1. For strand tendons anchored with two- or three-piece wedges, anchorage seating may be approximated as 6 mm.
2. Anchor seating for bar tendons may be approximated as 1.5 mm.
(d) **Loss of prestress due to other considerations**  Loss of prestress due to the following shall be taken into account in the design, where applicable:

(i) Deformation of the forms for precast members.

(ii) Difference in temperature between stressed tendons and the actual stressed structures during heat treatment of the concrete.

(iii) Changes in temperature between the time of stressing the tendons and the time of casting the concrete.

(iv) Deformations in the construction joints of precast structures assembled in sections.

(v) Relaxation of the tendon before transfer.

### 6.4.3 Time-dependent losses of prestress

#### 6.4.3.1 General

The total time-dependent loss of prestress shall be estimated by adding the calculated losses of prestress due to shrinkage of the concrete, creep of the concrete, tendon relaxation, and other considerations as may be applicable.

#### 6.4.3.2 Loss of prestress due to shrinkage of the concrete

The loss of stress in the tendon due to shrinkage of the concrete shall be taken as \( E_p \varepsilon_{cs} \), modified to allow for the effects of reinforcement, provided the shrinkage restraint effects of the reinforcement are included in the serviceability design of the member, where \( \varepsilon_{cs} \) is determined in accordance with Clause 6.1.7.2.

Where reinforcement is distributed throughout the member so that its effect on shrinkage is mainly axial, the loss of prestress in the tendons may be taken as \( E_p \varepsilon_{cs} \) divided by—

\[
1 + 15 \frac{A_s}{A_g}
\]

where

\( A_s = \) cross-sectional area of the reinforcement

\( A_g = \) gross cross-sectional area of the member

#### 6.4.3.3 Loss of prestress due to creep of the concrete

The loss of prestress due to creep of the concrete shall be calculated from an analysis of the creep strains in the concrete. In the absence of more detailed calculations and provided that the sustained stress in the concrete at the level of the tendons is less than \( 0.5 f'_c \), the loss of stress in the tendon due to creep of the concrete may be taken as \( E_p \varepsilon_{cc} \), where \( \varepsilon_{cc} \) shall be calculated as follows:

\[
\varepsilon_{cc} = 0.8 \phi_{cc} \left( \frac{\sigma_{ci}}{E_c} \right)
\]

where

\( \varepsilon_{cc} = \) strain due to concrete creep

\( \phi_{cc} = \) design creep factor of concrete calculated in accordance with Clause 6.1.8.3

\( \sigma_{ci} = \) sustained stress in the concrete at the level of the centroid of the tendons, calculated using the initial prestressing force before any time-dependent losses and the sustained portions of all the service loads

#### 6.4.3.4 Loss of prestress due to tendon relaxation

The loss of stress in a tendon due to relaxation of the tendon in the member shall be determined by modifying the percentage loss of stress due to the calculated relaxation of the tendon \((R)\) taking into account the effects of shrinkage and creep.
In the absence of more detailed calculations, the percentage loss of stress in the tendon in the member may be taken as—

\[
A_1 = \frac{1 - \frac{\text{the loss of stress due to creep and shrinkage}}{\sigma_{pi}}}{R} \quad \ldots 6.4.3.4
\]

where

\[\sigma_{pi} = \text{stress in the tendon immediately after transfer}\]

6.4.3.5 Loss of prestress due to other considerations

Account shall be taken, if applicable, of the following:

(a) Losses due to deformations in the joints of precast structures assembled in sections.

(b) Losses due to the effects of any increase in creep caused by frequently repeated loads.
SECTION 7 METHODS OF STRUCTURAL ANALYSIS

7.1 GENERAL

7.1.1 Basis for structural analysis

Methods of analysis for concrete structures shall take into account the following:

(a) The strength and deformational properties of the member materials.
(b) The equilibrium requirements for all forces acting on and within the structure.
(c) The requirements of compatibility of deformations within the structure.
(d) The support conditions and, where appropriate, interaction of the structure with the foundation and other connecting or adjacent structures.

7.1.2 Interpretation of the results of analysis

Irrespective of the method chosen for the structural analysis, the simplifications, idealizations and assumptions implied in the analysis shall be considered in relation to the real, three-dimensional nature of the structure when the results of the analysis are interpreted.

NOTE: Users of software packages for analysis should ensure the package is appropriate for the analysis being undertaken.

7.1.3 Methods of analysis

For the purposes of complying with the requirements for strength, stability and serviceability specified in Section 2, the action effects in a structure and its components shall be determined by one of the following methods:

(a) For reinforced or prestressed structures, including frames and slabs—
   (i) static analysis for determinate structures;
   (ii) linear elastic analysis in accordance with Clause 7.2;
   (iii) elastic frame analysis incorporating secondary bending moments due to lateral joint displacement in accordance with Clause 7.3;
   (iv) linear elastic stress (e.g.; finite element) analysis, in accordance with Clause 7.4;
   (v) strut-and-tie method of analysis, in accordance with Clause 7.5.
   (vi) rigorous structural analysis in accordance with Clause 7.6.
(b) For slabs and frames, plastic methods of analysis in accordance with Clause 7.7 and Clause 7.8.
(c) For isolated footings and pile caps and, where applicable, for combined footings, mats and pile caps—
   (i) where flexural action may be assumed, the methods specified in this Clause; or
   (ii) where flexural action cannot be assumed, the methods specified in Section 12.
(d) For non-flexural members, the methods specified in Section 12.
(e) For any structure, member, or assembly of members, structural model tests designed and evaluated in accordance with the principles of structural mechanics.
(f) For seismic analysis, methods shall be in accordance with Clause 7.9.
7.2 LINEAR ELASTIC ANALYSIS

7.2.1 Application
Linear elastic analysis shall be used for the purpose of determining the action effects in a structure for strength and serviceability design.

For a structure that can be represented as a framework of line members, the analysis shall comply with Clauses 7.2.2 to 7.2.11. For other structures, the analysis shall comply with the general principles of Clauses 7.2.2 to 7.2.11, as appropriate.

7.2.2 General
The framework shall be analysed in its entirety, making due allowance for the effects of shear lag. In the absence of more accurate methods, allowance for shear lag shall be made in determining the properties of the beams for the analysis in accordance with Clause 8.8.

7.2.3 Span lengths
In any framed structure, the span lengths shall be determined from the intersections of the centroidal axes of the various members comprising the frames, and maximum negative moments shall be calculated at sections in accordance with Clause 7.2.10.

7.2.4 Arrangement of loads for bridges
For bridges, the arrangement of loads considered in the analysis shall be as set out in AS 5100.2.

7.2.5 Stiffness
The stiffness of members shall be determined as follows:

(a) Relative stiffness In the calculation of the relative stiffness of members for analysis, any reasonable assumption may be made.

All such assumptions shall be applied consistently throughout the analysis.

The effect of haunching and variation in the cross-section along the axis of a member shall be considered and, where significant, shall be taken into account in the determination of relative stiffness.

(b) Member stiffness The assumed stiffness of members shall be chosen to represent conditions at the limit state being analysed.

(c) Torsional stiffness Particular account shall be taken in accordance with Item (b) of the effect of cracking on torsional stiffness, which, in the absence of additional information, shall be not greater than 20% of the uncracked torsional stiffness at the strength ultimate limit state when checking for flexure.

7.2.6 Deflections
Calculated deflections shall be modified to allow for cracking, tension stiffening and creep and shrinkage, unless these effects have already been taken into account in the analysis.

7.2.7 Secondary bending moments and shears resulting from prestress
The secondary bending moments and shears, and associated deformations that are produced in an indeterminate structure by prestressing shall be taken into account in the design calculations for serviceability.

The secondary bending moments and shears shall be determined by elastic analysis of the unloaded, uncracked structure for the effects of prestress.

In design calculations for the strength ultimate limit state, the secondary bending moments and shears shall be included with a load factor of 1.0 when the design moments and shears for the load combinations specified in AS 5100.2 are calculated.
7.2.8 Moment redistribution in reinforced concrete for strength design

7.2.8.1 General requirements

In design calculations for strength of statically indeterminate reinforced concrete members, the elastically determined bending moments at any support may be reduced or increased by redistribution, provided an analysis is undertaken to show that there is adequate rotation capacity in critical moment regions to allow the assumed distribution of bending moments to be achieved.

The analysis shall take into account—

(a) the stress-strain curve of the steel reinforcement as specified in Clause 6.2.3 assuming for analysis purposes that fracture of the reinforcement occurs at $\varepsilon_{fu}$;
(b) static equilibrium of the structure after redistribution of the moments; and
(c) the properties of the concrete specified in Clause 6.1.

Special consideration shall be given to the detrimental effects that significant relative foundation movements can have on the strength of continuous beams and slabs incorporating Ductility Class L reinforcing steel (low ductility) as the main reinforcement.

7.2.8.2 Simplified approach for Class N reinforcement

The requirement of Clause 7.2.8.1 shall be deemed to be met provided the following are satisfied:

(a) All of the main reinforcement in the member is Ductility Class N.
(b) The elastic bending moment distribution before redistribution is determined in accordance with Clause 7.2.5 assuming uncracked cross-sections.
(c) The positive bending moment is adjusted to maintain equilibrium.
(d) Where the neutral axis parameter ($k_u$) is less than or equal to 0.2 in all peak moment regions, the redistribution is not greater than 30%.
(e) Where $k_u$ is greater than 0.2 in one or more peak moment regions, but not greater than 0.4, the redistribution is not greater than $75(0.4 - k_u)$%.
(f) Where $k_u$ is greater than 0.4 in any peak region, redistribution shall not be made.

NOTES:
1 The values of $k_u$ are calculated for cross-sections that have been designed on the basis of the redistributed moment diagram.
2 The amount of redistribution is measured by the percentage of the bending moment before redistribution.

7.2.8.3 Class L reinforcement

Where Ductility Class L reinforcement is used, moment redistribution shall not be permitted unless an analysis as specified in Clause 7.2.8.1 is undertaken.

7.2.9 Moment redistribution in prestressed concrete for strength design

In the design of continuous prestressed concrete members at the strength ultimate limit state, the resultant elastically determined negative moment at any intermediate support, including the secondary moment, may be reduced or increased in accordance with Clause 7.2.8.2.

7.2.10 Critical section for negative moments

The critical section for maximum negative bending moment shall be taken as follows:

(a) For flexural members, a distance from the face toward the centre-line of the support of 0.15 times the dimension of the support in the direction of bending, or half the overall depth of the member, whichever is less.

NOTE: Circular or regular polygon-shaped supports may be treated as square supports having the same cross-sectional area.
(b) For footings supporting either—
   (i) a masonry wall, at halfway between the middle and the edge of the wall; or
   (ii) a column having a steel base, at the face of the column, unless analysis indicates that
        another section is more appropriate.

7.2.11 Unbonded prestress

7.2.11.1 Actions for unbonded prestress
Where prestressing tendons are not bonded (external or internal unbonded tendons), the effects of
prestressing shall be considered as a set of forces exerted on the concrete as follows:
(a) Concentrated forces at anchorages.
(b) Concentrated radial forces at deviators.
(c) Radial distributed forces of intensity \( \sigma_{p,ef} / (A_p r) \), \( r \) being the radius of curvature of the
    prestressing tendon.

NOTE: At each location, check should be undertaken considering the peak force in the tendon, which
may occur during jacking.

7.2.11.2 Strain compatibility
Strain compatibility analysis may be used for computation of the flexural resistance of bridges with
unbonded or partially bonded tendons. The analysis shall correctly recognize the differences in
strain between the tendons and the concrete section, and the effect of deflection geometry changes
on the effective stress in the tendons.

7.3 ELASTIC ANALYSIS OF FRAMES INCORPORATING SECONDARY BENDING
MOMENTS

7.3.1 General
An elastic analysis incorporating secondary bending moments shall comply with Clause 7.2 and the
following:
(a) The effect of lateral joint displacements shall be taken into account.
(b) For design at the strength ultimate limit state of a rectangular framed structure, the cross-
    sectional stiffness of the flexural members and columns shall be taken as \( 0.4E_c I_f \) and \( 0.8E_c I_c \)
    respectively, where \( I_f \) is the second moment of area of a flexural member and \( I_c \) is the second
    moment of area of a column.
(c) For very slender members, the change in bending stiffness due to axial compression shall be
    considered.

7.3.2 Application
This Clause applies to the analysis of frames not restrained by either bracing or shear walls, and
for which the relative displacement at the ends of compression members is less than \( L_u / 250 \) under
the design loads for strength, where \( L_u \) is the unsupported length of the column, taken as the clear
distance between the faces of members capable of providing lateral support to the column.

7.4 LINEAR ELASTIC STRESS (e.g., FINITE ELEMENT) ANALYSIS

7.4.1 General
This Clause applies to the linear elastic stress analysis of structures and parts of structures using
numerical methods, including finite element analysis.

7.4.2 Analysis
The analysis shall comply with the requirements of Clause 7.1.1. The results of the analysis shall
be interpreted in accordance with the requirements of Clause 7.1.2.
7.4.3  Sensitivity of analysis to input data and modelling parameters
Checks shall be made to investigate the sensitivity of the results of a linear elastic stress analysis to variations in input data and modelling parameters.

7.5  ANALYSIS USING STRUT-AND-TIE MODELS

7.5.1  General
When strut-and-tie modelling is used in the analysis of a concrete structure or local region, the relevant requirements of Appendix I shall be satisfied.

7.5.2  Sensitivity of analysis to input data and modelling parameters
Checks shall be made to investigate the sensitivity of the results of a strut-and-tie analysis to variations in geometry and modelling parameters.

7.6  RIGOROUS STRUCTURAL ANALYSIS

7.6.1  General
A rigorous structural analysis shall take into account the relevant material properties, geometric effects, three-dimensional effects and interaction with the foundations as specified in Clauses 7.6.2 to 7.6.6.

7.6.2  Material properties
The influence of the following material properties shall be taken into account:
(a)  Non-linear relation between stress and strain in the concrete.
(b)  Creep and shrinkage of the concrete.
(c)  Concrete cracking.
(d)  Tension stiffening.
(e)  Non-linear behaviour of steel.

7.6.3  Geometric effects
Equilibrium of the structure in the deformed condition shall be taken into account whenever the deflections within the length of an axially loaded member, or relative end displacements significantly influence the magnitude and distribution of action effects in the structure.

7.6.4  Three-dimensional effects
The three-dimensional nature of the structure shall be taken into account in the interpretation of the results of the analysis and, if relevant, in the analysis itself.

7.6.5  Interaction with the foundations
Interaction with the foundations shall be taken into account in the analysis.

7.6.6  Construction sequence and propping
The effects of construction sequences and techniques, including temporary propping, shall be considered in the analysis.

7.7  PLASTIC METHODS OF ANALYSIS FOR SLABS

7.7.1  General
Plastic methods of analysis based on lower bound or yield line theory may be used for the analysis for strength of one-way and two-way slabs, provided Ductility Class N reinforcement is used throughout.

The reinforcement shall be arranged with due regard to the serviceability requirements.
7.7.2 Lower bound method
The design bending moments obtained using lower bound theory shall satisfy the requirements of equilibrium and the boundary conditions applicable to the slab.

7.7.3 Yield line method
A yield line analysis for design at the strength ultimate limit state of a slab shall satisfy the following:
(a) The design bending moments shall be obtained from calculations based on the need for a mechanism to form over the whole or part of the slab at collapse.
(b) The mechanism that gives rise to the most severe design bending moments shall be used for the design of the slab.

7.8 PLASTIC METHODS OF ANALYSIS OF FRAMES
Plastic methods of analysis may be used for design at the strength ultimate limit state of frames and continuous beams provided that the members can be shown to possess the moment rotation capacities required to achieve the plastic redistribution of moments implied in the analysis.

7.9 SEISMIC ANALYSIS METHODS
Seismic analysis shall be in accordance with AS 5100.2.
For a bridge structure in earthquake design category BEDC-4, the collapse mechanism shall be defined using a post-elastic analysis and it shall be ensured that there is a unique and enforceable strength hierarchy within the structural system. Primary load-resisting members shall be chosen and suitably detailed for energy dissipation under severe inelastic deformations. All other structural members shall be provided with sufficient strength so that the chosen means of energy dissipation can be reliably maintained. Potential plastic hinges shall possess a substantial capacity to deform in a ductile manner.
SECTION 8  DESIGN OF BEAMS FOR STRENGTH AND SERVICEABILITY

8.1  STRENGTH OF BEAMS IN BENDING

8.1.1  General

The strength of a beam cross-section under bending shall be determined in accordance with Clauses 8.1.2 to 8.1.9, the material properties specified in Section 6, and the beam properties specified in Clause 8.8.

This Clause does not apply to members specified in Section 12.

8.1.2  Basic principles

8.1.2.1  Combined bending and axial force

Calculations for strength of cross-sections in bending, or in bending combined with axial force, shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following:

(a)  Plane sections normal to the axis remain plane after bending.
(b)  The concrete has no tensile strength.
(c)  The distribution of compressive stress in the concrete shall be determined by a recognized stress-strain relationship for the concrete in compression. A simplified stress-strain relationship is specified in Clause 8.1.2.2.
(d)  The strain in the compressive reinforcement is not greater than 0.003.

NOTE: If a curvilinear stress-strain relationship is used, Clause 6.1.4 places a limit on the value of the maximum concrete stress.

8.1.2.2  Rectangular stress block

Where the neutral axis lies within the cross-section and provided that the maximum strain in the extreme compression fibre of the concrete is taken as 0.003, Clause 8.1.2.1(c) shall be deemed to be satisfied by assuming that a uniform compressive stress of \( f'_c \) acts on an area bounded by—

(a)  the edges of the cross-section; and
(b)  a line parallel to the neutral axis at the strength limit state under the load concerned, and located at a distance \( \gamma k_u d \) from the extreme compressive fibre, where \( \gamma \) is the ratio, under design bending or combined bending and compression, of the depth of the assumed rectangular compressive stress block to \( k_u d \) and calculated as follows:

\[
\gamma = [0.85 - 0.007 (f'_c - 28)]
\]

within the limits 0.65 to 0.85 (see Figure 8.1.2).

NOTE: The modification specified in Clause 6.1.4 is included in the rectangular stress block assumptions.

8.1.2.3  Dispersion angle of prestress

In the absence of a more exact calculation, the dispersion angle of the prestressing force from the anchorage shall be assumed to be 60°, i.e., 30° either side of the centre-line.
8.1.2 Design strength in bending

The design strength in bending of a section with the neutral axis parameter \( k_u \) not greater than 0.4 shall be taken as \( \phi M_{uo} \), where \( M_{uo} \) is the ultimate strength in bending without axial force at a cross-section.

In peak moment regions, sections with \( k_u \) greater than 0.4 shall be avoided and shall not be used unless all of the following conditions are met:

(a) The structural analysis shall be carried out in accordance with Clauses 7.2, 7.3 and 7.6.
(b) Compression reinforcement of at least 0.01 times the area of concrete in compression shall be provided.
(c) The design strength in bending shall be taken as \( \phi M_{uo} \), where \( \phi \) shall be as given in Table 2.2.2 for bending without axial tension or compression, and \( k_u \) greater than 0.4.

In the determination of \( \phi \), \( M_{ud} \) shall be the reduced ultimate strength of the cross-section in bending, where \( k_u \) equals 0.4 and the tensile force has been reduced to balance the reduced compressive force.

\( M_{ud} \) may be calculated by assuming that—

(i) there are no axial forces acting on the cross-section;
(ii) the concrete strain at the extreme compressive fibre equals 0.003;
(iii) the effective depth \( d \) shall be as calculated for \( M_{uo} \);
(iv) \( k_u \) shall be reduced to 0.4; and
(v) the resultant of the tensile forces in the reinforcement and tendons shall be equal to the reduced compressive force calculated in accordance with Items (i) and (iv).

8.1.4 Minimum strength requirements

8.1.4.1 General

The ultimate strength in bending \( (M_{uo}) \) at critical sections shall not be less than \( (M_{uo})_{\text{min.}} \) calculated as follows:

\[
(M_{uo})_{\text{min.}} = 1.2 \left[ Z \left( f'_{ct} + \frac{P}{A_g} \right) + Pe \right]
\]

... 8.1.4.1(1)
where

\[ Z = \text{section modulus of the uncracked section, referred to the extreme fibre at which cracking occurs} \]

\[ f'_{cf} = \text{characteristic flexural tensile strength of the concrete} \]

\[ P = \text{prestressing force} \]

\[ A_g = \text{gross cross-sectional area of the member} \]

\[ e = \text{eccentricity of the prestressing force (P), measured from the centroidal axis of the uncracked section} \]

The requirement of this Clause may be waived at sections where it can be demonstrated that a sudden increase in deflection due to cracking will not lead to the sudden collapse of a span.

For rectangular reinforced concrete cross-sections, the requirement that \( M_{uo} \) shall not be less than \( (M_{uo})_{min} \) shall be deemed to be satisfied for the direction of bending being considered if minimum tensile reinforcement is provided such that—

\[
\frac{A_{st}}{b d} \geq 0.22 \left( \frac{D}{d} \right)^2 \frac{f'_{cf}}{f_{cy}}
\]

where

\[ A_{st} = \text{cross-sectional area of longitudinal reinforcement in the tensile zone} \]

\[ D = \text{overall depth of the cross-section in the plane of bending} \]

For earthquake design category BEDC-4, the ultimate strength in bending \( (M_{uo}) \) shall be not less than 1.1 \( (M_{uo})_{min} \) after allowing for the effects of axial loads.

**8.1.4.2 Prestressed beams at transfer**

The strength of a prestressed beam at transfer shall be determined using the load combinations specified in AS 5100.2 and the provisions of Clause 7.2.7, using a strength reduction factor (\( \phi \)) of 0.6. This requirement shall be deemed to be satisfied if the maximum compressive stress in the concrete, under the loads at transfer, is not greater than 0.5\( f_{cp} \) for a rectangular distribution of stress or 0.6\( f_{cp} \) for a triangular distribution of stress, and flexural cracking is controlled in accordance with Clause 8.6.2, where \( f_{cp} \) is the compressive strength of concrete at transfer.

**8.1.5 Stress in reinforcement and bonded tendons at ultimate strength**

The stress in the reinforcement at ultimate strength shall be not greater than \( f_{sy} \).

In the absence of a more accurate calculation, and provided that the minimum effective stress in the tendons is not less than 0.5\( f_{pb} \), the maximum stress in bonded tendons at the ultimate limit state (\( \sigma_{pu} \)) shall be calculated as follows:

\[
\sigma_{pu} = f_{pb} \left( 1 - \frac{k_1 k_2}{\gamma} \right)
\]

where

\[ k_1 = 0.4; \text{ or} \]

\[ = 0.28 \text{ if } f_{sy}/f_{pb} \geq 0.9 \]

\[ k_2 = \frac{1}{b_{ef} d_p f_{cy}} \left[ A_{pt} f_{pb} + (A_{st} - A_{sc}) f_{sy} \right]
\]

\[ b_{ef} = \text{effective width of a compression face or flange of a member} \]

\[ d_p = \text{distance from the extreme compressive fibre of the concrete to the} \]
centroid of the tendons in the zone that will be tensile under ultimate limit state conditions

\[ A_{pt} = \text{cross-sectional area of the tendons in the zone that will be tensile under} \]
\[ \text{ultimate load conditions} \]

\[ A_{sc} = \text{cross-sectional area of compressive reinforcement} \]

Compression reinforcement may only be taken into account if the distance \( d_{sc} \) from the extreme compressive fibre of the concrete to the centroid of compression reinforcement is not greater than \( 0.15d_p \), in which case \( k_2 \) shall be not less than 0.17.

8.1.6 Stress in tendons that are not bonded

Where the tendon is not bonded, the stress in the tendon at the strength ultimate limit state \( (\sigma_{pu}) \) shall be determined from the following equation, but in no case shall \( \sigma_{pu} \) be greater than \( f_{py} \):

\[ \sigma_{pu} = \sigma_{p,ef} + 6200 \left( \frac{d_p - k_u d}{L_{pe}} \right) \]

where

\[ \sigma_{p,ef} = \text{effective stress in the tendon (after losses)} \]

\[ k_u d = \text{neutral axis depth} \]

\[ = \left[ A_p f_{py} + (A_u - A_{sc}) f_{sy} - 0.85\gamma(b - b_u) d_f f'_c \right] \]

\[ = 0.85\gamma b_u f'_c \]

\[ = \frac{k_2 d_p}{0.85\gamma} \]

\( d_f = \text{thickness of the compression flange} \)

\( k_2 = \text{see Equation 8.1.5(2)} \)

\[ L_{pe} = \frac{L_{pa}}{1 + (n_s/2)} \]

\( L_{pa} = \text{length of the tendons} \)

\( n_s = \text{number of support hinges crossed by the tendon (draped tendons only)} \)

8.1.7 Spacing of reinforcement, tendons and ducts

8.1.7.1 General

In all members, where the concrete is to be compacted by internal vibration, the spacing of tendons, groups of tendons, reinforcement or ducts (especially in the horizontal plane), shall make provision for the insertion of vibrators at adequate intervals, and shall allow the concrete to completely fill the formwork to the intended level, expel entrapped air and closely surround all reinforcement, tendons, ducts, anchorages and embedments.

The clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8.1.7.2 Spacing

The spacing shall be determined as follows:

(a) The minimum clear horizontal distance between parallel bars, tendons, ducts, bundled bars and the like shall be not less than—
(i) 1.5 times the maximum nominal size of the aggregate;
(ii) 1.5 times the diameter of the bar or bundled bar;
(iii) 70 mm between prestressing ducts, except for grouped ducts, where the provisions of Clauses 8.1.7.3 to 8.1.7.5 are applicable;
(iv) 40 mm; or
(v) 30 mm for pretensioned strands in precast concrete, except for grouped tendons and ducts where the provisions of Clause 8.1.7.3 are applicable.

(b) Where positive or negative reinforcement is placed in two or more horizontal layers, the bars in the upper layers shall be placed directly above those in the bottom layers with a clear distance between layers not less than—

(i) 1.5 times the maximum nominal size of the aggregate;
(ii) 1.5 times the diameter of the bar or bundled bar;
(iii) 70 mm between prestressing ducts, except for grouped ducts, where the provisions of Clauses 8.1.7.3 to 8.1.7.5 are applicable;
(iv) 40 mm; or
(v) 30 mm for pretensioned strands in precast concrete, except for grouped tendons and ducts where the provisions of Clause 8.1.7.3 are applicable; or
(vi) 1.0 times the diameter of the bars or bundled bars in layers.

8.1.7.3 Grouping of tendons and ducts

Ducts shall not be placed in contact in a vertical plane.
The maximum number of ducts that may be placed in contact in a horizontal plane shall be two.

Where tendons or ducts are grouped together in contact, the spacing and concrete cover shall be detailed to facilitate the placing and compaction of the concrete.

Deflected pre-tensioned tendons may be grouped together provided that—

(a) they are grouped only in the middle third of the span, or in the case of a cantilever, they are grouped over the support; and
(b) where they diverge, they do so as rapidly as practicable.

8.1.7.4 Curvature and deviations of tendons and ducts

Where curved ducts are used, their position and sequence of tensioning and grouting shall be such that when a tendon is stressed, it cannot burst either into another duct or through the adjacent concrete.

To ensure that as a result of curvature a tendon cannot break through into an ungrouted duct, the spacing shall be such that—

\[ s_d \geq \frac{0.73P}{r} \]

where

- \( s_d \) = centre-to-centre distance between lines of ducts in the plane of the curvature
- \( P \) = prestressing force after initial losses
- \( r \) = radius of curvature of the duct

At all other locations where tendons curve or deviate, the adequacy of the concrete to carry the lateral force shall be assessed and, where necessary, the lateral load shall be carried by reinforcement designed in accordance with the principles set out in Section 12.
8.1.7.5 *Out-of-plane forces*

Curved tendons with multiple strands or wires also induce out-of-plane forces perpendicular to the plane of the tendon curvature. The distributed splitting force along the line of the tendon may be estimated as $0.16P/r$ in addition to any bursting forces calculated in accordance with Clause 12.2.4. The out-of-plane splitting force may be considered to be resisted over a distance $d_{sp}$ below the duct, that is, towards the centre of curvature, equal to the minimum of—

(a) twice the distance between the centre-line of the duct and the closest outer layer of non-stressed reinforcement parallel to the plane of curvature of the duct; and

(b) the clear distance between two ducts in the same or similar planes of curvature.

The splitting force may be resisted by the concrete in tension or by reinforcement designed in accordance with the principles set out in Section 12. The concrete tensile capacity may be taken as $f'_{ct}$, where $\phi$ is equal to 0.6. Transverse reinforcement, if provided, shall be spaced at less than or equal to 300 mm centres or $d_{sp}$, whichever is less.

**NOTE:** Further guidance is given in BS 5400.4.

8.1.8 *Detailing of flexural reinforcement and tendons*

8.1.8.1 *Distribution*

Tensile reinforcement shall be well distributed in zones of maximum concrete tension, including those portions of flanges of T-beams, L-beams and I-beams over a support.

8.1.8.2 *General procedure for arrangement*

The termination and anchorage of flexural reinforcement shall be based on a hypothetical bending-moment diagram formed by uniformly displacing the calculated positive and negative bending-moment envelopes a distance $D$ along the beam from each side of the relevant sections of maximum moment.

Not less than one-third of the total negative moment tensile reinforcement required at a support shall be extended a distance $D$ plus development length beyond the point of contraflexure.

8.1.8.3 *Anchorage of positive moment tensile reinforcement*

Anchorage of positive moment tensile reinforcement shall comply with the following:

(a) At a simple support, sufficient positive moment reinforcement shall be anchored for a length $(L_s)$ such that the anchored reinforcement shall develop a tensile force of—

(i) $V' \cot \theta_l/\phi$, where $\phi$ is as defined in Table 2.2.2 for shear; plus

(ii) the longitudinal torsion tensile force calculated in accordance with Clause 8.3.6, where $V'$ is the design shear force at a distance, $d \cot \theta_l$ from the anchor point; plus

(iii) any other longitudinal tensile forces in the reinforcement.

The anchor point shall be taken either halfway along the length of the bearing, or determined by calculating the width of the compressive strut in accordance with Clause I2 in Appendix I, taking account of both shear and torsion, and allowing for the truss angle being used. The truss angle ($\theta_l$) is as defined in Clause 8.2.10, and $L_{x,y}$ is determined from Clause 13.1.2.

(b) At a simple support, of the tensile reinforcement required at midspan, either—

(i) one half shall extend past the face of the support for a length of $12d_b$ or an equivalent anchorage, where $d_b$ is the nominal diameter of the bar, wire or tendon; or

(ii) one third shall extend past the face of the support for a length of $8d_b$ plus $d/2$.

(c) At a support where the beam is continuous or flexurally restrained, not less than one quarter of the total positive moment tensile reinforcement required at midspan shall continue past the near face of the support.
8.1.8.4 Shear strength requirements near terminated flexural reinforcement

If tensile reinforcement is terminated, the effect on the shear strength shall be assessed in accordance with the principles of the truss analogy.

This requirement shall be deemed to be satisfied if any one of the following conditions are met:

(a) Not more than a quarter of the maximum tensile reinforcement is terminated within any distance $2D$.

(b) At the cut-off point,

$$\phi V_u \geq 1.5V^+$$

where $V_u$ shall be determined in accordance with Clause 8.2.2.

(c) Stirrups are provided to give an area of shear reinforcement of $(A_{sv} + A_{sv,\text{min}})$ for a distance $D$ along the terminated bar from the cut-off point, where $A_{sv,\text{min}}$ and $A_{sv}$ shall be determined in accordance with Clauses 8.2.8 and 8.2.10 respectively.

8.1.8.5 Restraint of compression reinforcement

Compression reinforcement for strength in beams shall be adequately restrained by fitments specified in Clause 10.7.3.

8.1.8.6 Bundled bars

Groups of parallel longitudinal bars bundled to act as a unit shall be—

(a) limited to 4 in any one bundle with no more than 2 bars in one plane;

(b) tied together in contact; and

(c) enclosed within stirrups or ties.

Individual bars within a bundle, terminated within the span of flexural members, shall terminate at different points staggered by at least 40 times the diameter of the larger bar.

The unit of bundled bars shall be treated as an equivalent single bar of diameter derived from the total area of the bars in the bundle.

8.1.8.7 Special requirements for earthquake design category BEDC-4

For reinforced concrete members, the area of tensile and compression reinforcement shall be equal at sections where a plastic hinge is expected to develop. In addition, the member ultimate design axial compression force, under permanent loads and earthquake effects, at plastic hinge locations shall not be greater than 35% of the ultimate axial compression force capacity of the section.

For prestressed concrete members, in plastic hinge regions at least 40% of the total tensile steel shall be non-prestressed reinforcement.

The flexural strength shall be greater than 1.3 times the cracking moment at that section, after allowance for the effect of axial loads.

8.1.9 Displacement of tendons in ducts

In post-tensioned work, when detailing the profile of curved ducts, allowance shall be made for the relative displacement between the centroid of the tendon at high and low points of its profile and the centroid of the duct.

For both strength and serviceability stress calculations, the distance between the location of the tendon centre of gravity and the centre-line of the duct ($e_s$) may be assumed to be 0.2 times the duct diameter.
8.2 STRENGTH OF BEAMS IN SHEAR

8.2.1 Application

This Clause applies to reinforced and prestressed beams subjected to any combination of shear force, bending moment and axial force. When torsion acts in conjunction with shear force, the requirements given in Clause 8.3 also shall apply.

This Clause does not apply to members covered by Section 12.

8.2.2 Design shear strength of a beam

The design shear strength of a beam shall be taken as $\phi V_u$ where —

(a) $V_u$ equals $(V_{uc} + V_{us})$, taking account of Clauses 8.2.3 to 8.2.6, where $V_{uc}$ shall be determined in accordance with Clause 8.2.7 and $V_{us}$ determined in accordance with Clause 8.2.10, or where $V_{u,min}$ shall be determined in accordance with Clause 8.2.9; or

(b) $V_u$ is calculated by means of a method based on strut-and-tie modelling in accordance with Appendix 1; or

(c) where design for shear and torsion interaction is required in accordance with Clause 8.3, $V_{uc}$ shall be calculated in accordance with Clause 8.2.7.4.

8.2.3 Tapered members

In members that are tapered along their length, the components of inclined tension or compression forces shall be taken into account in the calculation of shear strength.

8.2.4 Maximum transverse shear near a support

The maximum transverse shear near a support shall be taken as the shear at—

(a) the face of the support; or

(b) a distance of $d_o$ from the face of the support, provided that—

(i) diagonal cracking can not take place at the support or extend into it;

(ii) there are no concentrated loads closer than $2d_o$ from the face of the support;

(iii) the value of $\beta_3$ specified in Clauses 8.2.7.1 and 8.2.7.2 is taken to be equal to one; and

(iv) the transverse shear reinforcement required at $d_o$ from the support is continued unchanged to the face of the support,

where

$$d_o = \text{distance from the extreme compressive fibre of the concrete to the centroid of the outermost layer of tensile reinforcement or tendons but not less than } 0.8D$$

$$\beta_3 = \text{coefficient}$$

In both Items (a) and (b), longitudinal tensile reinforcement required at $d_o$ from the face of the support shall be continued onto the support and shall be fully anchored past that face.

8.2.5 Requirements for shear reinforcement

For shear reinforcement, the following shall apply:

(a) Where $V^*$ is less than or equal to $\phi V_{u,min}$, minimum area of shear reinforcement ($A_{sv,min}$) shall be provided in accordance with Clause 8.2.8. This minimum shear reinforcement may be waived for beams where $V^*$ is less than or equal to $\phi V_{uc}$ and the total depth does not exceed the greater of 250 mm or half the width of the web.

(b) Where $V^*$ is greater than $\phi V_{u,min}$, shear reinforcement shall be provided in accordance with Clause 8.2.10.

(c) Where $P_v$ is greater than $V^*$, the original design shear force shall be modified as follows:
$$V^* = 1.2P_v - V^* \text{ (original)}$$  \ldots 8.2.5

where $P_v$ is the vertical component of the prestressing force at the section under consideration and shall be taken as zero for all subsequent shear calculations.

8.2.6 Shear strength limited by web crushing

In no case shall the ultimate shear strength ($V_u$) be greater than—

$$V_{\text{u, max.}} = 0.2f_y' b_v d_o + P_v; \text{ or}$$

$$V_{\text{u, max.}} = 0.2 \left(0.85 f_{cp} \right) b_v d_o + P_v, \text{ at transfer}$$  \ldots 8.2.6(1)

where

$V_{\text{u, max.}} = \text{ultimate shear strength limited by web crushing failure}$

$b_v = \text{effective width of the web for shear}$

$= (b_w - 0.5\Sigma d_d); \text{ or} \quad \ldots 8.2.6(2)$

$= (b_w - \Sigma d_d), \text{ at transfer} \quad \ldots 8.2.6(3)$

$\Sigma d_d = \text{sum of the diameters of the grouted ducts, if any, in a horizontal plane across the web}$

8.2.7 Shear strength of a beam excluding shear reinforcement

8.2.7.1 Reinforced beams

The ultimate shear strength ($V_{uc}$) of a reinforced beam, excluding the contribution of shear reinforcement, shall be calculated as follows:

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o \left( \frac{A_{st} f_z}{b_v d_o} \right)^{1/3}$$  \ldots 8.2.7.1(1)

where

for members where the cross-sectional area of shear reinforcement provided ($A_{sv}$) is equal to or greater than the minimum area specified in Clause 8.2.8—

$$\beta_1 = 1.1(1.6 - d_o/1000) \geq 1.1$$

otherwise—

$$\beta_1 = 1.1(1.6 - d_o/1000) \geq 0.8$$

$$\beta_2 = 1.0; \text{ or}$$

$$= 1 - \left( \frac{N^*}{3.5A_g} \right) \geq 0, \quad \text{for members subject to significant axial tension} \quad \ldots 8.2.7.1(3)$$

$$= 1 + \left( \frac{N^*}{14A_g} \right) \quad \text{for members subject to significant axial compression} \quad \ldots 8.2.7.1(4)$$

$$\beta_3 = 1; \text{ or may be taken as follows:}$$

$$= 2d_o/a_v \text{ but not greater than 2, provided that the applied loads and the support are oriented so as to create diagonal compression over the length } a_v$$

$A_{st} = \text{cross-sectional area of longitudinal reinforcement provided in the tension zone and fully anchored at the cross-section under consideration}$

$N^* = \text{design axial force}$
\[ a_v = \text{distance from the section at which shear is being considered to the face of the nearest support} \]

NOTE: Where significant reversal of loads may occur causing cracking in a zone usually in compression, the value of \( V_{uc} \) may not apply.

### 8.2.7.2 Prestressed beams

The ultimate shear strength \( (V_{uc}) \) of a prestressed beam, excluding the contribution of shear reinforcement, shall be taken as not greater than the lesser of the values obtained from Items (a) and (b) unless the cross-section under consideration is cracked in flexure, in which case only Item (b) applies.

\( V_o \) shall be determined as follows:

(a) **For flexural-shear cracking**

\[
V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o \left[ \frac{A_n + A_{pt}}{b_v d_o} \right]^{1/3} + V_o + P_v \quad \ldots \text{8.2.7.2(1)}
\]

where

\[ \beta_1, \beta_2, \beta_3, A_n \quad \text{as given in Clause 8.2.7.1 except that in determining } \beta_2, N^* \text{ shall be taken as the value of the axial force excluding prestress} \]

\[ V_o = \text{shear force, which would occur at the section under consideration when the bending moment at that section was equal to the decompression moment (} M_o \text{), calculated as follows:} \]

\[
M_o = Z \sigma_{cp.f} \quad \ldots \text{8.2.7.2(2)}
\]

\[ \sigma_{cp.f} = \text{compressive stress as a result of prestress at the extreme fibre where cracking occurs} \]

Where prestressing steel is not bonded, \( A_{pt} \) shall be taken as zero.

For simply supported conditions—

\[
V_o = \frac{M_o}{M^*/V^*} \quad \ldots \text{8.2.7.2(3)}
\]

where \( M^* \) and \( V^* \) are the design bending moment and design shear force respectively, at the section under consideration, due to the same design load.

\( M^* \) and \( V^* \) shall be calculated from—

(i) the design maximum moment and corresponding shear; and

(ii) the design maximum shear and corresponding moment.

For statically indeterminate structures, shear forces and bending moments due to the secondary effects of prestress shall be taken into account when determining \( M_o \) and \( V_o \). 

(b) **For web-shear cracking**

\[
V_{uc} = V_v + P_v \quad \ldots \text{8.2.7.2(4)}
\]
where

\[ V_t = \text{shear force which, in combination with the prestressing force and other action effects at the section, would produce a principal tensile stress of} \]

\[ 0.33 \sqrt{f_c^2} \] at either the centroidal axis or the intersection of flange and web, whichever is the more critical

### 8.2.7.3 Secondary effects on \( V_{uc} \)

Where stresses due to secondary effects such as creep, shrinkage and differential temperature are significant, they shall be taken into account in the calculation of \( V_{uc} \).

**NOTE:** Where significant reversal of loads may occur, causing cracking in a zone usually in compression, the value of \( V_{uc} \) obtained in accordance with Clause 8.2.7(a) and (b) may not apply.

### 8.2.7.4 Reversal of loads and members in torsion

Where loading cases occur that result in cracking in a zone usually in compression, the value of \( V_{uc} \) obtained from Clause 8.2.7.1 or 8.2.7.2 may not apply and \( V_{uc} \) shall be assessed or be taken as zero.

### 8.2.8 Minimum shear reinforcement

The minimum area of shear reinforcement \((A_{sv,\text{min.}})\) provided in a beam shall be calculated as follows:

\[
A_{sv,\text{min.}} = 0.06 \sqrt{f_c b_v s / f_{sy,t}} \geq 0.35 b_v f_{sy,t} \] \(
\ldots 8.2.8
\)

where

- \( s \) = centre-to-centre spacing of shear reinforcement, measured parallel to the longitudinal axis of the member
- \( f_{sy,t} \) = the yield strength of the reinforcement used in fitments

### 8.2.9 Shear strength of a beam with minimum reinforcement

The ultimate shear strength of a beam \( V_{u,\text{min.}} \) provided with minimum shear reinforcement \( A_{sv,\text{min.}}, \) shall be calculated as follows:

\[
V_{u,\text{min.}} = V_{uc} + 0.10 \sqrt{f_c b_v d_o} \geq V_{uc} + 0.6 b_v d_o \] \(
\ldots 8.2.9
\)

### 8.2.10 Contribution to shear strength by the shear reinforcement

The contribution to the ultimate shear strength by perpendicular shear reinforcement in a beam \( (V_{us}) \) shall be determined as follows:

\[
V_{us} = \frac{A_{sv} f_{sy,t} d_o}{s} \cot \theta_v \] \(
\ldots 8.2.10
\)

where

- \( A_{sv} \) = area of the cross-section of the shear reinforcement
- \( \theta_v \) = angle between the axis of the concrete compression strut and the longitudinal axis of the member, taken conservatively as 45°, or chosen in the range of 30° to 60° except that the minimum value of \( \theta_v \) shall be taken as varying linearly from 30°, when \( V' = \phi V_{u,\text{min}}, \) to 45°, when \( V' = \phi V_{u,\text{max}} \)
- \( s \) = centre-to-centre spacing of shear reinforcement, measured parallel to the longitudinal axis of the member

### 8.2.11 Suspension reinforcement

If forces are applied to a beam in such a way that hanging action is required, reinforcement or tendons shall be provided to carry all of the forces concerned.
Where, for some situations, suspension reinforcement is required, it shall be in accordance with the procedures given in Appendix D or by the provision of hanger reinforcement of area consistent with strut-and-tie modelling.

### 8.2.12 Detailing of shear reinforcement

#### 8.2.12.1 Types

Shear reinforcement shall comprise one or more of—

(a) stirrups or ties making an angle of 90° with the longitudinal bars; and

(b) helices.

#### 8.2.12.2 Spacing

Shear reinforcement shall be spaced longitudinally not further apart than 0.5D or 300 mm, whichever is the lesser.

The maximum transverse spacing across the width of the beam shall be not greater than the lesser of 600 mm and D.

#### 8.2.12.3 Extent

The shear reinforcement required at the critical cross-section shall be carried to the face of the support.

Shear reinforcement, of area not less than that calculated as being necessary at any cross-section, shall be provided for a distance D from that cross-section in the direction of decreasing shear. The first fitment at each end of a span shall be positioned not more than 50 mm from the face of the adjacent support.

Shear reinforcement shall extend as close to the compression face and the tension face of the member as cover requirements and the proximity of other reinforcement and tendons permit. Bends in bars used as fitments shall enclose a longitudinal bar with a diameter not less than the diameter of the fitment bar. The enclosed bar shall be in contact with the fitment bend.

#### 8.2.12.4 Anchorage

The anchorage of shear reinforcement transverse to the longitudinal flexural reinforcement may be achieved by a hook or cog complying with Clause 13.1.2.7 or by welding of the fitment to a longitudinal bar or by a welded splice.

**NOTE:** The type of anchorage used should not induce splitting or spalling of the concrete cover.

Notwithstanding the above, fitment cogs are not to be used when the fitment cog is located within 50 mm of any concrete surface.

### 8.3 STRENGTH OF BEAMS IN TORSION

#### 8.3.1 Application

This Clause applies to beams subjected to torsion and torsion in combination with shear. It does not apply to non-flexural members, which are covered by Section 12.

#### 8.3.2 Torsion redistribution

Where torsional strength is not required for the equilibrium of the structure and the torsion in a member is induced solely by the angular rotation of adjoining members, it shall be permissible to disregard the torsional stiffness in the analysis and torsion in the member, if the torsion reinforcement requirements of Clause 8.3.7 and the detailing requirements of Clause 8.3.8 are satisfied.

#### 8.3.3 Torsional strength limited by web crushing

To prevent web crushing under the combined action of torsion and flexural shear, beams shall be proportioned so that the following is satisfied:
\[
\frac{T^*}{\phi T_{u,\text{max}}} + \frac{V^*}{\phi V_{u,\text{max}}} \leq 1 \quad \ldots 8.3.3(1)
\]

where

- \( T^* \) = design torsional moment at a cross-section (action effect) calculated using the design loads for strength specified in AS 5100.2
- \( V_{u,\text{max}} \) = ultimate shear strength calculated in accordance with Clause 8.2.6
- \( T_{u,\text{max}} \) = ultimate torsional strength of a beam limited by web crushing failure
  \[ = 0.2 f'_c J_t \quad \ldots 8.3.3(2) \]
- \( J_t \) = torsional modulus
  \[
  = 0.33 x^2 y \quad \text{for solid rectangular sections} \quad \ldots 8.3.3(3)
  \]
  \[
  = 0.33 \Sigma x^2 y \quad \text{for solid T-, L- or I-shaped sections} \quad \ldots 8.3.3(4)
  \]
  \[
  = 2 A_m b_w \quad \text{for thin walled hollow sections} \quad \ldots 8.3.3(5)
  \]
- \( x \) = shorter overall dimension of a rectangular part of a cross-section
- \( y \) = longer overall dimension of a rectangular part of a cross-section
- \( A_m \) = area enclosed by the median lines of the walls of a single cell
- \( b_w \) = width of the web or the thickness of the wall of a hollow section

8.3.4 Requirements for torsional reinforcement

In the calculation of \( T^* \) and \( V^* \) in Items (a) and (b), the elastic uncracked stiffness shall be used.

Requirements for torsional reinforcement shall be determined as follows:

(a) Torsional reinforcement is not required if—

(i) \( T^* < 0.25 \phi T_{uc} \); or \ldots 8.3.4(1)

(ii) \[ \frac{T^*}{\phi T_{uc}} + \frac{V^*}{\phi V_{uc}} \leq 0.5; \] or \ldots 8.3.4(2)

(iii) \[ \frac{T^*}{\phi T_{uc}} + \frac{V^*}{\phi V_{uc}} \leq 1 \quad \text{and the overall depth of the beam is not greater than 250 mm or half the width of the web} \quad \ldots 8.3.4(3) \]

where \( T_{uc} \) and \( V_{uc} \) shall be calculated in accordance with Clauses 8.3.5 and 8.2.7, respectively and \( T^* \) shall be calculated taking into account the effect of cracking on the torsional stiffness.

(b) If Item (a) is not satisfied, torsional reinforcement, consisting of transverse closed ties and longitudinal reinforcement shall be provided so that the following is satisfied:

\[
\frac{T^*}{\phi T_{us}} \leq 1 \quad \ldots 8.3.4(4)
\]
where $T_{us}$ shall be calculated in accordance with Clause 8.3.5, considering all the closed ties provided, $V_{us}$ shall be calculated in accordance with Clause 8.2.10, considering all the closed and open ties provided and $T^*$ shall be calculated taking into account the effect of cracking on the torsional stiffness.

Longitudinal torsional reinforcement shall comply with Clause 8.3.6 and both transverse and longitudinal torsional reinforcement shall comply with Clause 8.3.7.

Shear reinforcement shall be provided with $V_{uc}$ assessed in accordance with Clause 8.2.7.4.

### 8.3.5 Torsional strength of a beam

For the purpose of Clause 8.3.4, the ultimate torsional strength of a beam in pure torsion ($T_{uc}$) or ($T_{us}$) shall be determined as follows:

(a) For a beam without closed ties, the ultimate strength in pure torsion ($T_{uc}$) shall be calculated as follows:

$$T_{uc} = J_t \left(0.3 \sqrt{f'_c}\right) \sqrt{1 + \frac{10 \sigma_{cp}}{f'_c}} \ldots \text{8.3.5(1)}$$

(b) For a beam with closed ties, the ultimate strength in pure torsion ($T_{us}$) shall be calculated as follows:

$$T_{us} = f_{sy,t} \left(A_{sw} / s\right) 2 A_t \cot \theta_v \ldots \text{8.3.5(2)}$$

where

- $\sigma_{cp}$ = average intensity of effective prestress in concrete
- $A_{sw}$ = cross-sectional area of the bar forming a closed tie
- $A_t$ = area of a polygon with vertices at the centre of longitudinal bars at the corners of the cross-section
- $\theta_v$ = angle between the axis of the concrete compression strut and the longitudinal axis of the member, taken conservatively as $45^\circ$, or chosen in the range of $30^\circ$ to $60^\circ$ except that the minimum value of $\theta_v$ shall be taken as varying linearly from $30^\circ$, when $V^* = \phi V_{u,\text{min}}$, to $45^\circ$, when $V^* = \phi V_{u,\text{max}}$

### 8.3.6 Longitudinal torsional reinforcement

Longitudinal torsional reinforcement shall be provided to resist the following design tensile forces, taken as additional to any design tensile forces due to flexure:

(a) In the flexural tensile zone, a force of—

$$0.5 f_{sy,t} \left(\frac{A_{sw}}{S}\right) u_t \cot^2 \theta_v ; \text{ and} \ldots \text{8.3.6(1)}$$

(b) In the flexural compressive zone, a force of—

$$0.5 f_{sy,t} \left(\frac{A_{sw}}{S}\right) u_t \cot^2 \theta_v - F^*_c ; \text{ but not less than zero,} \ldots \text{8.3.6(2)}$$

where

- $u_t$ = perimeter of the polygon defined for $A_t$
- $F^*_c$ = absolute value of the design force in the compression zone as a result of flexure

NOTE: Longitudinal torsional reinforcement including minimum reinforcement is additional to the longitudinal flexural reinforcement required for the load case of simultaneous flexure and torsion. The torsional reinforcement does not need to be added to the longitudinal flexural reinforcement provided for the load case, if any, of flexure without torsion.
**8.3.7 Minimum torsional reinforcement**

Where torsional reinforcement is required as specified in Clause 8.3.4—

(a) longitudinal torsional reinforcement shall be provided in accordance with Clause 8.3.6; and

(b) minimum transverse reinforcement shall be provided to satisfy the greater of—

(i) the minimum shear reinforcement required by Clause 8.2.8 in the form of closed ties or fitments; and

(ii) a torsional capacity equal to 0.25$T_{uc}$.

**8.3.8 Detailing of torsional reinforcement**

Torsional reinforcement shall be detailed as follows:

(a) Torsional reinforcement shall consist of both closed ties and longitudinal reinforcement.

(b) The closed ties shall be continuous around all sides of the cross-section and anchored so as to develop full strength at any point, unless a more refined analysis shows that full anchorage is not required over part of the tie. The spacing $(s)$ of the closed ties shall be not greater than the lesser of $0.12d_t$ and $300$ mm.

   In large members where a single closed loop of reinforcement is not possible, bars shall extend in one length over the full depth of the web, or width of the flange, with adequate anchorage by means of hooks or cogs at the intersection of webs and flanges.

(c) Additional longitudinal reinforcement shall be placed as close as practicable to the corners of the cross-section, and in all cases at least one longitudinal bar shall be provided at each corner of the closed ties.

**8.3.9 Concrete details**

Cross-sections of members subjected to torsion shall be designed to avoid sharp re-entrant corners by providing fillets.

**8.4 LONGITUDINAL SHEAR IN BEAMS**

**8.4.1 Application**

This Clause applies to the transfer of longitudinal shear forces across interface shear planes through webs and flanges for composite beams, and across shear planes through flanges cast monolithically.

This Clause also applies to the transfer of shear across any specific interface such as between precast and in situ concrete or across construction joints.

**8.4.2 Design shear stress**

The design shear stress $(\tau^*)$ acting on the interface shall be taken as follows:

$$\tau^* = \beta V/(z b_f)$$

where

$z = \text{internal moment lever arm of the section}$

For a shear plane that passes through a region in compression—

$$\beta = \text{ratio of the compressive force in the member (calculated between the extreme compressive fibre and the shear plane) and the total compression force in the section}$$

For a shear plane that passes through a region in tension—

$$\beta = \text{ratio of the tensile force in the longitudinal reinforcement (calculated between the extreme tensile fibre and the shear plane) and the total tension force in the section}$$
8.4.3 Shear stress capacity

The design shear stress at the shear interface shall not exceed \( \phi \tau_u \) where—

\[
\tau_u = \mu \left( \frac{A_{sf} f_{sy}}{s b_f} + \frac{g_p}{b_f} \right) + k_{co} f_{ct}' \leq \text{lesser of } 0.2 f'_{c} \text{ and } 10 \text{ MPa} \ldots 8.4.3
\]

where

- \( \tau_u \) = unit shear strength
- \( g_p \) = permanent distributed load normal to the shear interface per unit length, newtons per millimetre (N/mm)
- \( \mu \) = coefficient of friction given in Table 8.4.3
- \( k_{co} \) = cohesion coefficient given in Table 8.4.3
- \( b_f \) = width of the shear plane, in millimetres (mm)
- \( A_{sf} \) = area of fully anchored shear reinforcement crossing the interface (mm²)
- \( f_{sy} \) = yield strength of shear reinforcement not exceeding 500 MPa
- \( s \) = spacing of anchored shear reinforcement crossing interface

**TABLE 8.4.3**

<table>
<thead>
<tr>
<th>Surface condition of the shear plane</th>
<th>Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \mu )</td>
</tr>
<tr>
<td>A smooth surface, as obtained by casting against a form, or finished to a similar standard</td>
<td>0.6</td>
</tr>
<tr>
<td>A surface trowelled or tamped, so that the fines have been brought to the top, but where some small ridges, indentations or undulations have been left; slip-formed and vibro-beam screeded; or produced by some form of extrusion technique</td>
<td>0.6</td>
</tr>
<tr>
<td>A surface deliberately roughened—</td>
<td></td>
</tr>
<tr>
<td>(a) by texturing the concrete to give a pronounced profile;</td>
<td></td>
</tr>
<tr>
<td>(b) by compacting but leaving a rough surface with coarse aggregate protruding but firmly fixed in the matrix;</td>
<td>0.9</td>
</tr>
<tr>
<td>(c) by spraying when wet, to expose the coarse aggregate without disturbing it; or</td>
<td></td>
</tr>
<tr>
<td>(d) by providing mechanical shear keys.</td>
<td></td>
</tr>
<tr>
<td>Monolithic construction</td>
<td>0.9</td>
</tr>
</tbody>
</table>

**NOTE:** Where a beam is subjected to high levels of differential shrinkage, temperature effects, tensile stress or fatigue effects across the shear plane, the values of \( \mu \) and \( k_{co} \) in the above Table do not apply.

8.4.4 Shear plane reinforcement

Where reinforcement is required to increase the longitudinal shear strength, the reinforcement shall consist of shear reinforcement anchored to develop its full strength at the shear plane. Shear and torsional reinforcement already provided, and which crosses the shear plane, may be taken into account for this purpose.

The centre-to-centre spacing \( s \) of the shear reinforcement shall not exceed the maximum spacing—

\[
s_{\text{max}} = 3.5 t_f \ldots 8.4.4
\]

where
8.4.5 Minimum thickness of structural components

The minimum thickness of structural components subject to interface shear shall be not less than 70 mm.

8.5 DEFLECTION OF BEAMS

8.5.1 General

Where required, the instantaneous and final deflections of a beam shall be calculated in accordance with Clause 8.5.2 or Clause 8.5.3.

8.5.2 Beam deflection by refined calculation

The calculation of the deflection of a beam by refined calculation shall make allowance for the following:

(a) The expected shrinkage and creep properties of the concrete.

(b) The expected load history.

(c) The effect of cracking and tension stiffening.

8.5.3 Beam deflection by simplified calculation

8.5.3.1 Short-term deflection

The short-term deflections due to external loads and prestressing, which occur immediately on their application, shall be calculated using the value of $E_{cj}$ determined in accordance with Clause 6.1.2 and the value of the effective second moment area of the member ($I_{ef}$).

The value of $I_{ef}$ shall be determined from the values of $I_{ef}$ at nominated-cross-sections as follows:

(a) For simply supported span, the value at midspan.

(b) In a continuous beam—

(i) for an interior span, half the midspan value plus one quarter of each support value; or

(ii) for an end span, half the midspan value plus half the value at the continuous support.

(c) For a cantilever, the value at the support.

For the purpose of the determination of the $I_{ef}$ values specified in Items (a) to (c), the value of $I_{ef}$ at each of the cross-sections nominated in Items (a) to (c) is given by—

$$I_{ef} = I_{ct} + (I - I_{ct})(M_{ct} / M) \leq I_{e,max} \quad \ldots \text{8.5.3.1}(1)$$

where

$$I_{ct} = \text{second moment of area of a cracked section with the reinforcement transformed to an equivalent area of concrete}$$

$$I_{e,max} = \text{maximum effective second moment of area of the member}$$

$$= I, \text{ for prestressed sections}$$

$$= I, \text{ for reinforced sections} \quad \text{when } p = \frac{A_{st}}{bd} \geq 0.005 \quad \ldots \text{8.5.3.1}(2)$$

$$= 0.6I, \text{ for reinforced sections} \quad \text{when } p = \frac{A_{st}}{bd} < 0.005 \quad \ldots \text{8.5.3.1}(3)$$

$$I = \text{second moment of area of the gross concrete cross-section}$$

$$M_{ct} = \text{bending moment causing cracking of the section with due consideration to prestress, restrained shrinkage and temperature stresses}$$
76

\[ M_s = \text{maximum bending moment at the section, based on the short-term serviceability load or the construction load} \]

\[ Z = \text{section modulus of the uncracked section, referred to the extreme fibre at which cracking occurs} \]

\[ f'_{ct} = \text{characteristic flexural tensile strength of concrete} \]

\[ \sigma_{cs} = \text{maximum shrinkage-induced tensile stress on the uncracked section at the extreme fibre at which cracking occurs. In the absence of more refined calculation, } \sigma_{cs} \text{ may be taken as} \]

\[ \sigma_{cs} = \frac{2.5p_w - 0.8p_{cw}}{1 + 50p_w} E_s \varepsilon_{cs}^* \]

\[ p_w = \text{web reinforcement ratio for tensile reinforcement} = \frac{A_{st} + A_{pt}}{b_w d} \]

\[ p_{cw} = \text{web reinforcement ratio for compressive reinforcement} = \frac{A_{sc}}{b_w d} \]

\[ \varepsilon_{cs}^* = \text{final design shrinkage strain of concrete} \]

Where appropriate, \( \sigma_{cs} \) shall be increased to account for axial tension induced by restraint to shrinkage by the support to the beams.

Alternatively, as a further simplification but only for reinforced members, \( I_{ef} \) may be taken as—

\[ I_{ef} = [(5 - 0.04f'_{ct})p + 0.002] b_{ef} d^3 \leq [0.1 / \beta^{2/3}] b_{ef} d^3 \text{ when } p \geq 0.001 (f'_{ct})^{1/3} / \beta^{2/3} \]

\[ I_{ef} = [0.055 (f'_{ct})^{1/3} / \beta^{2/3} - 50 p] b_{ef} d^3 \leq [0.06 / \beta^{2/3}] b_{ef} d^3 \text{ when } p < 0.001 (f'_{ct})^{1/3} / \beta^{2/3} \]

where

\[ \beta = b_{ef}/b_w \geq 1 \]

\[ p = A_{st}/b_{ef} d \text{ at midspan} \]

8.5.3.2 Long-term deflection for beams cracked under permanent loads

In the absence of more accurate calculations, the additional long-term deflection due to creep and shrinkage of a beam cracked under permanent loads may be calculated by multiplying the short-term deflection caused by the sustained load considered, by a factor (\( k_{cs} \)) used in serviceability design to take account of the long-term effects of creep and shrinkage and calculated as follows:

\[ k_{cs} = \left[ 2 - 1.2 \left( \frac{A_{sc}}{A_{st}} \right) \right] \geq 0.8 \]

where \( A_{sc}/A_{st} \) is taken at—

(a) midspan, for a simply supported or continuous beam; or
(b) the support for a cantilever beam.

Where local conditions indicate that severe creep or shrinkage effects exist, a larger value of \( k_{cs} \) than that by given by Equation 8.5.3.2 shall be used, or the more general procedure specified in Clause 8.5.2 shall be used.
8.6 CRACK CONTROL OF BEAMS

8.6.1 Crack control for tension and flexure in reinforced beams

Cracking in reinforced beams subjected to tension or flexure shall be deemed to be controlled if the appropriate requirements in Items (a), (b) and (c), and either Item (d) for beams primarily in tension or Item (e) for beams primarily in flexure, are satisfied. In cases when the reinforcement has different yield strengths, the yield strength ($f_{y}$) shall be taken as the lowest yield strength of any of the tensile longitudinal reinforcement.

For the purpose of this Clause, the resultant action is considered to be primarily tension when the whole of the section is in tension, or primarily flexure when the tensile stress distribution within the section prior to cracking is triangular with some part of the section in compression. These resultant actions are referred to as tension and flexure, respectively.

(a) The minimum area of reinforcement in a tensile zone of a beam shall comply with Clause 8.1.4.1.

(b) The distance from the side or soffit of a beam to the centre of the nearest longitudinal bar shall be not greater than 100 mm. Bars with a diameter less than half the diameter of the largest bar in the cross-section shall be ignored. The centre-to-centre spacing of bars near a tension face of the beam shall be not greater than 300 mm. For T-beams and L-beams, the reinforcement required in the flange shall be distributed across the effective width.

(c) Load effects shall be considered for the following two cases:

(i) Serviceability limit state load combinations.

(ii) For bridges designed for exposure classifications B2, C and U only, permanent effects at the serviceability limit state.

(d) For beams subject to tension, the steel stress ($\sigma_{scr}$) calculated assuming the section is cracked shall not exceed the maximum steel stress given in Table 8.6.1(A) for the largest nominal diameter ($d_b$) of the bars in the section, and under direct loading the calculated tensile steel stress ($\sigma_{scr,1}$) shall not exceed $0.8f_{y}$.

<table>
<thead>
<tr>
<th>Nominal bar diameter ($d_b$) mm</th>
<th>Loading case specified in Item (c)(i)</th>
<th>Loading case specified in Item (c)(ii)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. steel stress ($\sigma_{scr}$) MPa</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>450</td>
<td>340</td>
</tr>
<tr>
<td>8</td>
<td>400</td>
<td>305</td>
</tr>
<tr>
<td>10</td>
<td>360</td>
<td>275</td>
</tr>
<tr>
<td>12</td>
<td>330</td>
<td>250</td>
</tr>
<tr>
<td>16</td>
<td>280</td>
<td>215</td>
</tr>
<tr>
<td>20</td>
<td>240</td>
<td>185</td>
</tr>
<tr>
<td>24</td>
<td>210</td>
<td>160</td>
</tr>
<tr>
<td>28</td>
<td>185</td>
<td>140</td>
</tr>
<tr>
<td>32</td>
<td>160</td>
<td>125</td>
</tr>
<tr>
<td>36</td>
<td>140</td>
<td>110</td>
</tr>
<tr>
<td>40</td>
<td>120</td>
<td>95</td>
</tr>
</tbody>
</table>

NOTE: Values for other bar diameters may be calculated using the appropriate equations, as follows:

Maximum steel stress equals:

(a) $[760 - 173 \log_{e}(d_b)]$ MPa for loading case specified in Item (c)(i).

(b) $[575 - 130 \log_{e}(d_b)]$ MPa for loading case specified in Item (c)(ii).
For beams subject to flexure, the steel stress ($\sigma_{scr}$) calculated assuming the section is cracked shall not exceed the maximum steel stress given in Table 8.6.1(A) for the largest nominal diameter ($d_b$) of bars in the tensile zone. Alternatively, the steel stress shall not exceed the maximum stress determined from Table 8.6.1(B) for the centre-to-centre spacing of adjacent parallel bars in the tensile zone. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored when determining spacing.

NOTE: In comparing the calculated steel stress ($\sigma_{scr}$) with the maximum steel stresses determined from Tables 8.6.1(A) and 8.6.1(B) depending on the bar diameter and bar spacing, respectively, the greater of these two maximum steel stresses may be used.

**TABLE 8.6.1(B)**

<table>
<thead>
<tr>
<th>Centre-to-centre spacing (mm)</th>
<th>Loading case specified in Item (c)(i)</th>
<th>Loading case specified in Item (c)(ii)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. steel stress ($\sigma_{scr}$)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MPa</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>360</td>
<td>280</td>
</tr>
<tr>
<td>100</td>
<td>320</td>
<td>240</td>
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<td>150</td>
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<td>250</td>
<td>200</td>
<td>120</td>
</tr>
<tr>
<td>300</td>
<td>160</td>
<td>80</td>
</tr>
</tbody>
</table>

NOTE: Values for other centre-to-centre spacings may be calculated using the appropriate equations, as follows:

1. Maximum steel stress equals:
   - (a) $[400 - (0.8 \times \text{centre-to-centre spacing})]$ MPa for loading case specified in Item (c)(i).
   - (b) $[320 - (0.8 \times \text{centre-to-centre spacing})]$ MPa for loading case specified in Item (c)(ii).

### 8.6.2 Crack control for flexure in prestressed beams

The crack control for flexure in prestressed beams shall be determined as follows:

(a) **Monolithic beams** Flexural cracking in a prestressed beam shall be deemed to be controlled if under the serviceability limit state load combinations, the resulting maximum tensile stress in the concrete is not greater than $0.25\sqrt{f'_{c}}$ or, if this stress is exceeded, reinforcement or bonded tendons, or both, near the tensile face shall be provided and either—

   - (i) the calculated maximum flexural tensile stress under the serviceability limit state load combination, including transfer shall be limited to—
     $$0.5\sqrt{f'_{c}} \text{ or } 0.5\sqrt{f'_{sp}}; \text{ or} \ldots 8.6.2$$

   - (ii) the increment in steel stress near the tension face shall be limited to 200 MPa as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the serviceability limit state load combination values. The centre-to-centre spacing of reinforcement, including bonded tendons shall be limited to 200 mm.

(b) **Segmental members at unreinforced joints** Under all serviceability limit state load combinations, tensile stress shall not be permitted.

(c) **Prestressed members in exposure classifications B2, C or U** The concrete at the level of each tendon shall be in compression under a serviceability limit state load combination that comprises permanent effects plus 50% of the serviceability live load.

(d) **Railway bridges** In addition to the above requirements, no tensile stresses shall occur under permanent load effects in prestressed beams for railway bridges, and a minimum compression of 1.0 MPa shall be maintained at unreinforced joints in segmental members.
8.6.3 Crack control in the side face of beams
Minimum reinforcement shall be provided for crack control in the face of beams in accordance with Clause 2.8. Reinforcement placed near the surface to carry other loads may be included in providing this area of steel.

8.6.4 Crack control at openings and discontinuities
Reinforcement shall be provided for crack control at openings and discontinuities in a beam.

8.7 VIBRATION OF BEAMS
Beams and box girders shall comply with the vibration requirements as specified in AS 5100.2 to ensure that vibrations induced by vehicular and pedestrian traffic shall not adversely affect the serviceability of the structure.

8.8 PROPERTIES OF BEAMS
8.8.1 General
Where a slab is assumed to provide the flange of a T-beam, L-beam or box section, the longitudinal shear shall be designed for in accordance with Clause 8.4.
Composite concrete members shall be designed in accordance with Appendix E.

8.8.2 Effective width of flange for analysis for serviceability
In the absence of a more accurate determination, allowance for the effect of shear lag shall be made by using an effective width of flange determined as follows:

(a) For T-beams and box-sections:
\[ b_{ef} = b_w + 0.2a \]  \[ . . . 8.8.2(1) \]

(b) For L-beams:
\[ b_{ef} = b_w + 0.1a \]  \[ . . . 8.8.2(2) \]

where \( a \) is the distance between points of zero bending moment, which, for continuous beams, may be taken as 0.7L.

In Items (a) and (b), the overhanging part of the flange considered effective shall be not greater than half the clear distance to the next member, or six times the thickness of the flange plus the smaller dimension of any fillet between flange and web, whichever is the lesser.

NOTE: For further information on box sections, see Appendix F.

8.8.3 Effective width of flange for analysis for strength
At the strength limit state, allowance need not be made for shear lag effects and the full section properties may be used, in which case a separate analysis for serviceability is required.
Alternatively, the properties specified in Clause 8.8.2 shall be used.

8.9 SLENDERNESS LIMITS FOR BEAMS
8.9.1 General
Unless a stability analysis is carried out, beams shall comply with the limits specified in Clauses 8.9.2 to 8.9.4, as appropriate.

8.9.2 Simply supported and continuous beams
For a simply supported or continuous beam, the distance (\( L_L \)) between points at which lateral restraint is provided shall be such that \( L_L/b_{ef} \) is not greater than the lesser of 240\( b_{ef}/D \) and 60.
8.9.3 Cantilever beams
For a cantilever beam having lateral restraint only at the support, the ratio of the clear projection \((L_n)\) to the width \((b_{ef})\) at the support shall be such that \(L_n/b_{ef}\) is not greater than the lesser of 100\(b_{ef}/d\) and 25.

8.9.4 Additional reinforcement for prestressed beams
For a prestressed beam in which \(L_n/b_{ef}\) is greater than 30 or for a prestressed cantilever beam in which \(L_n/b_{ef}\) is greater than 12, the following reinforcement shall be provided:

(a) Stirrups equal in area to at least—
\[
A_{sv,\text{min}} = \frac{0.35 b_w s}{f_{sy,t}}
\] . . . 8.9.4(1)

(b) Additional longitudinal reinforcement of area \((A_{sc})\) consisting of at least one bar in each corner of the compression face such that—
\[
A_{sc} \geq \frac{0.35 A_{pt} f_p}{f_{sy}}
\] . . . 8.9.4(2)
SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY

9.1 STRENGTH OF SLABS IN BENDING

9.1.1 General
Slabs in bridge structures shall in general be considered as one-way slabs and be designed for bending in accordance with Clauses 8.1.1 to 8.1.6 and 8.1.8, except that minimum tensile steel $A_{st}$ shall be provided such that—

$$A_{st} \geq 0.24 \left( \frac{D^2 b}{d} \right) \frac{f'_{ct}}{f_{sy}}$$

where the two-way design of flat slabs is considered necessary, the design shall comply with the relevant provisions of AS 3600.

9.1.2 Distribution reinforcement for slabs
Reinforcement shall be placed in the bottom of all slabs transverse to the main reinforcement.

For road bridges, unless a more accurate analysis is carried out, the amount of distribution reinforcement shall be a percentage of the main reinforcement required for positive moment as follows:

(a) Main reinforcement parallel to traffic—

Percentage $= \frac{1750}{\sqrt{L}}$  \hspace{1cm} \ldots 9.1.2(1)

(maximum 50%; minimum 30%)

(b) Main reinforcement perpendicular to traffic—

Percentage $= \frac{3500}{\sqrt{L}}$  \hspace{1cm} \ldots 9.1.2(2)

(maximum 67%; minimum 30%)

With main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement in the outer quarters of the span may be reduced by a maximum of 50%.

For railway bridges, the reinforcement for slabs shall be based on a rational analysis using the distribution of railway traffic loading specified in AS 5100.2.

9.1.3 Edge stiffening
The edge stiffening of slabs shall be considered as follows:

(a) *Longitudinal* Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. An edge beam may consist of a kerb section, a beam integral with the slab, or a slab edge additionally reinforced or extended.

(b) *Transverse* Transverse edges at the ends of the bridge and at intermediate points where the continuity of the slab is broken shall be additionally reinforced or supported by edge beams or diaphragms designed for the full effects of the wheel loads.

The need for longitudinal or transverse edge stiffening of slabs shall be based on a rational analysis of the slab using the specified loadings plus any other loading that may be applied to the edge of the slab during the life of the structure.

9.1.4 Minimum thickness of deck slabs
The thickness of the deck slab shall not be less than 150 mm.
9.2 STRENGTH OF SLABS IN SHEAR

9.2.1 Application

The strength of a slab in shear shall be calculated as follows:

(a) Where a slab with a depth of less than or equal to 300 mm may act essentially as a wide beam and a shear failure may occur across the entire width or over a substantial width, the strength shall be calculated in accordance with Clause 8.2, except that—

(i) for a reinforced concrete slab without shear reinforcement, the minimum value of $V_u$ may be taken as follows:

$$V_u = 0.17 \sqrt{f'_c} b d_o$$

. . . 9.2.1(1)

(ii) for a prestressed concrete slab without shear reinforcement, the minimum value of $V_u$ may be taken as follows:

$$V_u = 0.17 \sqrt{f'_c} b d_o + V_o + P_v$$

. . . 9.2.1(2)

(b) Where the potential failure surface may form a truncated cone or pyramid around the support or loaded area, the strength of the slab shall be determined in accordance with Clauses 9.2.3 and 9.2.4.

Where failure modes specified in Items (a) and (b) are possible, the shear strength shall be calculated in accordance with both Items (a) and (b), and the smaller value shall be taken as the critical strength.

9.2.2 Design shear strength of slabs

The design shear strength of a slab shall be taken as $\phi V_u$, where $V_u$ shall be determined in accordance with Clause 8.2, Clause 9.2.3 or Clause 9.2.4 as appropriate.

9.2.3 Shear strength of slabs without moment transfer

The ultimate shear strength of a slab with no moment transfer ($V_{uo}$), taken equal to $V_u$, shall be calculated as follows:

(a) Where no shear reinforcement or fabricated shear head is provided—

$$V_{uo} = u d_{om} (f_{cv} + 0.3 \sigma_{cp})$$

. . . 9.2.3(1)

(b) Where shear reinforcement or a fabricated shear head is provided—

$$V_{uo} = u d_{om} (0.5 \sqrt{f'_c} + 0.3 \sigma_{cp}) \leq 0.2 u d_{om} f'_c$$

. . . 9.2.3(2)

where

- $u =$ length of the critical shear perimeter as defined below
- $d_{om} =$ mean value of $d_o$, averaged around the critical shear perimeter ($u$)
- $f_{cv} =$ concrete shear strength

$$= 0.17 \left(1 + \frac{2}{\beta_h}\right) \sqrt{f'_c} \leq 0.34 \sqrt{f'_c}$$

. . . 9.2.3(3)

- $\sigma_{cp} =$ average intensity of effective prestress in the concrete
- $\beta_h =$ ratio of the longest overall dimension of the effective loaded area ($Y$) to the shortest overall dimension ($X$) measured perpendicular to $Y$ (see Figure 9.2.3)

For the purpose of this Clause, the critical shear perimeter ($u$) is defined by a line geometrically similar to the boundary of the effective area of a support or load and located at a distance of $d_{om}/2$.
from the boundary as shown in Figure 9.2.3. The effective area of a support or load shall be that area totally enclosing the actual support or load for which the perimeter is a minimum.

That part of the critical shear perimeter that is enclosed by radial projections from the centroid of the support or load to the extremities of any critical opening shall be regarded as ineffective.

An opening shall be regarded as critical if it is located at a clear distance of less than $2.5b_o$ from the critical shear perimeter, where $b_o$ is the width of the critical opening as shown in Figure 9.2.3(b).

9.2.4 Shear strength of slabs with moment transfer
If a bending moment is designated to be transferred from a slab to a support, it shall comply with the relevant provisions of AS 3600.

9.3 DEFLECTION OF SLABS
9.3.1 General
Where required, the deflection of a slab shall be determined in accordance with Clause 9.3.2 or Clause 9.3.3.

9.3.2 Slab deflection by refined calculation
The calculation of the deflection of a slab by refined calculation shall make allowance for the following:
(a) Two-way action.
(b) Shrinkage and creep properties of the concrete.
(c) Expected load history.
(d) Cracking and tension stiffening.

9.3.3 Slab deflection by simplified calculation
The deflection of slabs spanning one way shall be calculated in accordance with Clause 8.5.3 on the basis of an equivalent beam taken as a prismatic beam of unit width.
9.4 CRACK CONTROL OF SLABS

9.4.1 Crack control for flexure in reinforced slabs

For the purpose of this Clause, a critical tensile zone is defined as a region of a slab where the design bending moment at the serviceability limit state \( M_{s1} \) is greater than or equal to the critical moment for flexural cracking \( M_{crk} \), which is calculated assuming a flexural tensile strength of concrete equal to 3.0 MPa.

In cases when the main reinforcement has different yield strengths, the yield strength \( f_{yy} \) shall be taken as the lowest yield strength of any of the reinforcement.

Cracking in reinforced slabs subject to flexure shall be deemed to be controlled if the appropriate requirements specified in Items (a), (b), (c), and (f) and either the requirement specified in Item (d) for slabs primarily in tension or Item (e) for slabs primarily in flexure are satisfied.

For the purpose of this Clause, the resultant action is considered to be primarily tension when the whole of the section is in tension, or primarily flexure when the tensile stress distribution within the section prior to cracking is triangular with some part of the section in compression. These resultant actions are referred to as tension and flexure, respectively.

\[ \text{(a) The minimum area of reinforcement in a tensile zone of a slab shall comply with Clause 9.1.1.} \]

\[ \text{(b) The centre-to-centre spacing of bars in each direction shall be not greater than the lesser of 2.0D_s or 300 mm where } D_s \text{ is the overall depth of the slab. Bars with a diameter less than half the diameter of the largest bar in the cross-section shall be ignored.} \]

\[ \text{(c) Load effects shall be considered for the following two cases:} \]

\[ \text{(i) Serviceability limit state load combinations.} \]

\[ \text{(ii) For bridges designed for exposure classifications B2, C and U with slabs of depths greater than 300 mm or for slabs of any depth composite with a beam, permanent effects at the serviceability limit state.} \]

\[ \text{(d) For slabs subject to tension, the steel stress } (\sigma_{scr}) \text{ calculated assuming the section is cracked shall not exceed the maximum steel stress given in Table 9.4.1(A) for the largest nominal diameter } (d_b) \text{ of the bars in the section.} \]

\[ \text{(e) For slabs subject to flexure, the steel stress } (\sigma_{scr}) \text{ calculated assuming the section is cracked shall not exceed the maximum steel stress given in Table 9.4.1(A) for the largest nominal diameter of bars in the tensile zone. Alternatively, the steel stress shall not exceed the maximum stress determined from Table 9.4.1(B) for the centre-to-centre spacing of adjacent parallel bars in the tensile zone. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored when determining spacing.} \]

\[ \text{(f) The calculated steel stress } (\sigma_{scr}) \text{ shall not exceed } 0.8f_{yy}. \]

\[ \text{NOTE: In comparing the calculated steel stress } (\sigma_{scr}) \text{ with the maximum steel stresses determined from Tables 9.4.1(A) and 9.4.1(B) depending on the bar diameter and bar spacing, respectively, the greater of these two maximum steel stresses may be used.} \]
TABLE 9.4.1(A)
MAXIMUM STEEL STRESS FOR FLEXURE IN SLABS

<table>
<thead>
<tr>
<th>Nominal bar diameter, $d_b$ mm</th>
<th>Loading case specified in Item (c)(i)</th>
<th>Loading case specified in Item (c)(ii)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Overall depth $D_s \leq 300$</td>
<td>Overall depth $D_s &gt; 300$</td>
</tr>
<tr>
<td>6</td>
<td>375</td>
<td>450</td>
</tr>
<tr>
<td>8</td>
<td>345</td>
<td>400</td>
</tr>
<tr>
<td>10</td>
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<td>265</td>
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<td>20</td>
<td></td>
<td>240</td>
</tr>
<tr>
<td>24</td>
<td></td>
<td>210</td>
</tr>
</tbody>
</table>

NOTE: Values for other bar diameters may be calculated using the appropriate equation, as follows:

Maximum steel stress equals:
(a) $[760 - 173 \log_e (d_b)]$ MPa for $d_b \geq 20$ mm for loading case specified in Item (c)(i).
(b) $[760 - 173 \log_e (d_b)]$ MPa for $d_b < 20$ mm and $D_s > 300$ mm for loading case specified in Item (c)(i).
(c) $[580 - 114 \log_e (d_b)]$ MPa for $d_b < 20$ mm and $D_s \leq 300$ mm for loading case specified in Item (c)(i).
(d) $[575 - 130 \log_e (d_b)]$ MPa for loading case specified in Item (c)(ii).

TABLE 9.4.1(B)
MAXIMUM STEEL STRESS FOR FLEXURE IN SLABS

<table>
<thead>
<tr>
<th>Centre-to-centre spacing mm</th>
<th>Loading case specified in Item (c)(i)</th>
<th>Loading case specified in Item (c)(ii)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>360</td>
<td>280</td>
</tr>
<tr>
<td>100</td>
<td>320</td>
<td>240</td>
</tr>
<tr>
<td>150</td>
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<tr>
<td>200</td>
<td>240</td>
<td>160</td>
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<td>250</td>
<td>200</td>
<td>120</td>
</tr>
<tr>
<td>300</td>
<td>160</td>
<td>80</td>
</tr>
</tbody>
</table>

NOTE: Values for other centre-to-centre spacings may be calculated using the appropriate equations, as follows:

Maximum steel stress equals:
(a) $[400 - (0.8 \times \text{centre-to-centre spacing})]$ MPa for loading case specified in Item (c)(i).
(b) $[320 - (0.8 \times \text{centre-to-centre spacing})]$ MPa for loading case specified in Item (c)(ii).

9.4.2 Crack control for flexure in prestressed slabs

Flexural cracking in a prestressed slab shall be deemed to be controlled if, under the serviceability limit state load combination, the resulting maximum tensile stress in the concrete is not greater than $0.25 \sqrt{f' C}$. If this stress is exceeded, reinforcement or bonded tendons near the tensile face shall be provided and either—

(a) the calculated maximum flexural tensile stress in the concrete under the serviceability limit state load combinations, including transfer, shall be limited to—

$0.5 \sqrt{f' C}$; or
(b) the increment in steel stress near the tension face shall be limited to 150 MPa as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the serviceability limit state load combination values. The centre-to-centre spacing of reinforcement including bonded tendons shall be limited to the lesser of $D$ and 300 mm.

For prestressed members in exposure classifications B2, C or U, the concrete at the level of each tendon shall be in compression under a serviceability limit state load combination that comprises permanent effects plus 50% of the serviceability live load.

### 9.4.3 Crack control for shrinkage and temperature effects

Provided the minimum reinforcement provisions of Clause 2.8 are complied with, no additional reinforcement is required for shrinkage and temperature control of unrestrained slabs.

#### 9.4.4 Reinforcement for fully restrained slabs

The minimum area of reinforcement in slabs in the restrained direction shall be not less than—

(a) \[ (6.0 - 2.5\sigma_{cp})bD \times 10^{-3} \] for reinforcement bars of 16 mm diameter or less \ldots 9.4.4(1)

(b) \[ (8.0 - 2.5\sigma_{cp})bD \times 10^{-3} \] for reinforcement bars of 20 mm diameter \ldots 9.4.4(2)

The reinforcement shall be placed equally on each face of the slab and located as close to each face as cover and detailing permit. $D$ need not be taken as greater than 500 mm.

NOTE: Reinforcement provided for structural reasons can be considered as contributing to this requirement.

#### 9.4.5 Crack control at openings and discontinuities

For crack control at openings and discontinuities in a slab, additional, properly anchored, reinforcement shall be provided.

#### 9.4.6 Crack control in the vicinity of restraints

In the vicinity of restraints, special attention shall be paid to the internal forces and cracks which may be induced by prestressing, shrinkage or temperature.

### 9.5 VIBRATION OF SLABS

All slabs intended for pedestrian access, including bridge walkways, pedestrian bridges, access routes to platforms or similar shall comply with the vibration requirements of AS 5100.2, to ensure that vibrations induced by vehicular or pedestrian traffic shall not adversely affect the serviceability of the structure.

### 9.6 MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS

In the absence of more exact calculations, the width ($b$) of a solid one-way simply supported or continuous slab, deemed to resist the moments caused by a concentrated load, may be taken as follows:

(a) Where the load is not near an unsupported edge—

\[ b_{ef} = \text{the load width} + 2.4a \left( 1.0 - \frac{a}{L_a} \right) \] \ldots 9.6

where

\[ a = \text{perpendicular distance from the nearer support to the section under consideration} \]

(b) Where the load is near an unsupported edge, not greater than the lesser of—
(i) the value given in Item (a); and
(ii) half the value given in Item (a) plus the distance from the centre of the load to the unsupported edge.

9.7 LONGITUDINAL SHEAR IN SLABS

Composite slab systems shall be checked for longitudinal shear at the interfaces between components in accordance with Clause 8.4.

9.8 FATIGUE OF SLABS

The tensile stress range of steel in slabs and the concrete compressive stresses in slabs shall comply with Clause 2.5.
SECTION 10 DESIGN OF COLUMNS AND TENSION MEMBERS FOR STRENGTH AND SERVICEABILITY

10.1 GENERAL

10.1.1 Design strength

The design strength of a column shall be determined by its ability to resist the axial forces and bending moments caused by the design load for strength and any additional bending moments produced by slenderness effects.

10.1.2 Minimum bending moment

At any cross-section of a column, the design bending moment about each principal axis shall be taken to be not less than \( N^* \times 0.05D \), where \( D \) is the overall depth of the column in the plane of the bending moment.

10.1.3 Definitions

For the purpose of this Section, the definitions below apply.

10.1.3.1 Braced columns

Members for which the lateral load on the structure in the direction under consideration is resisted by lateral bracing.

10.1.3.2 Short columns

Columns in which the additional bending moments due to slenderness can be taken as zero.

10.1.3.3 Slender columns

Columns that do not satisfy the requirements for short columns.

10.1.4 Crack control in columns and tension members

In addition to strength requirements, columns and tension members shall also comply with Clause 8.6, unless otherwise approved by the authority.

10.2 DESIGN PROCEDURES

10.2.1 Design procedure using linear elastic analysis

Where the axial forces and bending moments are determined by a linear elastic analysis as specified in Clause 7.2, a column shall be designed as follows:

(a) For a short column, in accordance with Clauses 10.3, 10.6 and 10.7.

(b) For a slender column, in accordance with Clauses 10.4 to 10.7.

10.2.2 Design procedure incorporating secondary bending moments

Where the axial forces and bending moments are determined by an elastic analysis incorporating secondary bending moments due to lateral joint displacements, as specified in Clause 7.3, a column shall be designed in accordance with Clauses 10.6 and 10.7. The bending moments in slender columns shall be further increased by applying the moment magnifier for a braced column \( (\delta_b) \) calculated in accordance with Clause 10.4.2 with \( L_e \) taken as \( L_u \) in the determination of \( N_e \).

10.2.3 Design procedure using rigorous analysis

Where the axial forces and bending moments are determined by a rigorous analysis as specified in Clause 7.6, a column shall be designed in accordance with Clauses 10.6 and 10.7 without further consideration of additional moments due to slenderness.
10.3 DESIGN OF SHORT COLUMNS

10.3.1 General

Short columns shall be designed in accordance with Clauses 10.6 and 10.7, with additional bending moments due to slenderness taken to be zero. Alternatively, for short columns with small axial forces the design may be in accordance with Clause 10.3.2.

A column shall be deemed to be short where—

(a) for a braced column,

\[
\frac{L_e}{r} \leq 25; \text{ or } \\
A1 \quad \frac{L_e}{r} \leq 60 \left(1 + \frac{M_1^*}{M_2^*}\right) \left(1 - \frac{N^*}{0.6N_{uo}}\right)
\]

whichever is the greater; or

(b) for an unbraced column,

\[
\frac{L_e}{r} \leq 22
\]

where

- \(L_e\) = effective length of the column determined in accordance with Clause 10.5.3 or alternatively may be taken as—
  - \(0.9L_u\) for a braced column restrained by beams
  - \(L_u\) for a column designed in accordance with Clause 10.2.2
- \(r\) = radius of gyration of the cross-sections determined in accordance with Clause 10.5.2
- \(M_1^*/M_2^*\) = ratio of the smaller to the larger of the design bending moments at the ends of the column. The ratio is taken to be negative when the column is bent in single curvature and positive when the column is bent in double curvature. When the absolute value of \(M_2^*\) is less than or equal to \(0.05DN^*\), the ratio shall be taken as \(-1.0\)
- \(N_{uo}\) = ultimate strength in compression of an axially loaded cross-section without bending forces

10.3.2 Short column with small axial compressive force

Where the design axial compressive force (\(N^*\)) in a short column is less than \(0.1f'_cA_k\), the cross-section may be designed for bending only.

10.4 DESIGN OF SLENDER COLUMNS

10.4.1 General

Slender columns shall be designed in accordance with Clauses 10.6 and 10.7 with additional bending moments due to slenderness effects taken into account by multiplying the largest design bending moment, by the moment magnifier (\(\delta\)).

The moment magnifier (\(\delta\)) shall be calculated in accordance with Clause 10.4.2 for a braced column and Clause 10.4.3 for an unbraced column.

For columns subject to bending about both principal axes, the bending moment about each axis shall be magnified by \(\delta\), using the restraint conditions applicable to each plane of bending.
10.4.2 Moment magnifier for a braced column

The moment magnifier ($\delta$) for a braced column shall be taken to be equal to $\delta_b$, calculated as follows:

\[
\delta_b = \frac{k_m}{1 - \left(\frac{N^*}{N_c}\right)} \geq 1.0
\]

. . . 10.4.2(1)

where

- $k_m = \text{coefficient} = 0.6 - 0.4 \left(\frac{M_1^*}{M_2^*}\right)$ but shall be taken as not less than 0.4, except that if the column is subjected to significant transverse loading between its ends and in the absence of more exact calculations, $k_m$ shall be taken as 1.0
- $N_c = \text{buckling load specified in Clause 10.4.4}$

10.4.3 Moment magnifier for an unbraced column

The moment magnifier ($\delta$) for an unbraced column shall be taken as the larger value of $\delta_b$ or $\delta_s$ where—

(a) $\delta_b$ for an individual column is calculated in accordance with Clause 10.4.2 assuming the column is braced.

(b) $\delta_s$ for each column in a bent is calculated as follows:

\[
\delta_s = \frac{1}{1 - \frac{\Sigma N^*}{\Sigma N_c}}
\]

. . . 10.4.3(1)

where the summations include all columns at the same level and $N_c$ is calculated for each column in accordance with Clause 10.4.4.

(c) As an alternative to Item (b), $\delta_s$ may be calculated from a linear elastic critical buckling load analysis of the entire frame, where $\delta_s$ is taken as a constant value for all columns calculated as follows:

\[
\delta_s = \frac{1 + \beta_d}{\phi_s \lambda uc}
\]

. . . 10.4.3(2)

where

- $\beta_d = \text{ratio of dead load to total load at the ultimate limit state taken as zero if}$—
  - (i) $\frac{L_e}{r} \leq 40$; and
  - (ii) $N^* \leq \frac{M^*}{2D}$
- $\phi_s = \text{correlation factor taken as 0.6}$
- $\lambda uc = \text{ratio of the elastic critical buckling load of the entire frame to the design load for strength, calculated by taking the cross-sectional stiffness of the flexural members and columns as 0.4E_cI_f and 0.8E_cI_c, respectively}$

The frame shall be proportioned so that $\delta_s$ for any column is not greater than 1.5.
10.4.4 Buckling load

The buckling load \( N_c \) shall be taken as follows:

\[
N_c = \frac{\pi^2}{L_c^2} \left[ \frac{182d_o \phi M_{ub}}{1 + \beta_d} \right]
\]

where

\[\phi M_{ub} = \text{particular ultimate strength in bending of the cross-section assuming} \]

\[k_{uo} = 0.545, \text{ and } \phi = 0.6\]

10.5 SLENDERNESS

10.5.1 General

The slenderness ratio \( (L_c/r) \) of a column shall be not greater than 120, unless a rigorous analysis has been carried out in accordance with Clause 7.6 and the column is designed in accordance with Clause 10.2.3.

Where the forces and moments acting on a column have been obtained from a linear elastic analysis as specified in Clause 7.2, the influence of slenderness shall be taken into account using a radius of gyration \( (r) \) specified in Clause 10.5.2 and an effective length \( (L_e) \) in accordance with Clause 10.5.3.

10.5.2 Radius of gyration

The radius of gyration of the cross-section \( (r) \) shall be calculated for the gross concrete cross-section. For a rectangular cross-section, \( r \) may be taken as \( 0.3D \) where \( D \) is the overall dimension in the direction in which stability is being considered and for a circular cross-section, \( r \) may be taken as \( 0.25D \).

10.5.3 Effective length of a column

The effective length of a column \( (L_e) \) shall be taken as \( kL_u \) where the effective length factor \( (k) \) shall be determined as shown in Figure 10.5.3(A) for columns with simple end restraints, or more generally as shown in Figure 10.5.3(B) or Figure 10.5.3(C), as appropriate.

The end restraint coefficients \( \gamma_1 \) and \( \gamma_2 \) shall be determined as follows:

(a) For all structures, including structures where the axial forces in the restraining members are large, in accordance with Clause 10.5.4.

(b) Where the column ends at a footing, in accordance with Clause 10.5.5.

Alternatively, the effective length of a column may be determined from the elastic critical buckling load of the frame as calculated by analysis.
FIGURE 10.5.3(A) EFFECTIVE LENGTH FACTOR ($k$) FOR COLUMNS WITH SIMPLE END RESTRAINTS

FIGURE 10.5.3(B) EFFECTIVE LENGTH FACTOR ($k$) FOR A BRACED COLUMN
FIGURE 10.5.3(C) EFFECTIVE LENGTH FACTOR \((k)\) FOR AN UNBRACED COLUMN

10.5.4 End restraint coefficients for a framed structure

For any framed structure, the end restraint coefficient \((\gamma_1)\) at one end of a column and the end restraint coefficient \((\gamma_2)\) at the other end, may be calculated as the ratio of the column stiffness to the sum of the stiffnesses of all the members, except the column, meeting at the end under consideration. In the calculation of the stiffnesses of the members other than the column, due account shall be taken of the fixity conditions of each member at the end remote from the column-end being considered as well as any reduction in member stiffness due to axial compression.

\[
\gamma = \frac{(I/L)_c}{\sum \beta (I/L)_n}
\]

\[\ldots 10.5.4\]

where

\((I/L)_c\) = stiffness in the plane of bending of only the column under consideration

\(\sum \beta (I/L)_n\) = sum of the stiffnesses in the plane of bending of all the members except the column under consideration meeting at and rigidly connected to the column under consideration

\(\beta\) = fixity factor, given in Table 10.5.4, for fixity conditions at the end of a member opposite to the end connected to the column under consideration
TABLE 10.5.4

FIXITY FACTOR ($\beta$)

<table>
<thead>
<tr>
<th>Fixity conditions at far end of a beam, slab or column</th>
<th>Member in a braced frame</th>
<th>Member in an unbraced frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinned</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Rigidly connected to a column</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Fixed</td>
<td>2.0</td>
<td>0.67</td>
</tr>
</tbody>
</table>

10.5.5 End restraint coefficient at a footing

Where a footing provides negligible restraint to the rotation of the end of a column, $\gamma$ is theoretically infinite but may be taken as 10.

Where a footing is specifically designed to prevent rotation of the end of a column, $\gamma$ is theoretically zero but shall be taken as one unless analysis would justify a smaller value.

At a free end, $\gamma$ may be taken as infinite.

10.6 STRENGTH OF COLUMNS IN COMBINED BENDING AND COMPRESSION

10.6.1 Basis of strength calculations

Calculations for the strength of cross-sections in bending combined with axial forces shall incorporate equilibrium and strain-compatibility considerations and shall be consistent with the following assumptions:

(a) Plane sections normal to the axis remain plane after bending.
(b) The concrete has no tensile strength.
(c) The distribution of stress in the concrete and the steel is determined using a stress-strain relationship determined in accordance with Clauses 6.1.4 and 6.2.3 respectively. The simplified stress-strain relationship provided in Clause 10.6.2 may be used for concrete.
(d) The strain in the compressive reinforcement is not greater than 0.003.

Columns subject to axial force with bending moments about each principal axis may take into account the concessions specified in Clauses 10.6.4 and 10.6.5.

10.6.2 Rectangular stress block

Where the neutral axis lies within the cross-section and provided that the maximum strain in the extreme compression fibre of the concrete is taken as 0.003, Clause 10.6.1(c) shall be deemed to be satisfied by a uniform concrete compressive stress of $0.85f_c^\prime$ acting on an area bounded by—

(a) the edges of the cross-section; and
(b) a line parallel to the neutral axis under the load concerned, and located at a distance $\gamma k_u d$ from the extreme compressive fibre, where—

$$\gamma = \left[0.85 - 0.007(f_c^\prime - 28)\right]$$

within the limits 0.65 to 0.85.

10.6.3 Calculation of the ultimate strength in compression ($N_{uo}$)

The ultimate strength in compression ($N_{uo}$) of an axially loaded cross-section without bending forces shall be calculated by assuming—

(a) a uniform concrete compressive stress of $0.85f_c^\prime$; and
(b) a maximum strain in the steel and the concrete of 0.0025.
10.6.4 Design based on each bending moment acting separately
For a rectangular cross-section, where the ratio of the larger to the smaller cross-sectional dimension is not greater than 3.0, which is subjected simultaneously to an axial force and bending moment about each principal axis, the cross-section may be designed for the axial force with each bending moment considered separately, provided that the line of the resultant axial force falls within the shaded area of the cross-section shown in Figure 10.6.4.

![Figure 10.6.4: Limitation for line of action of the axial force](image)

Shaded areas symmetrical about column centre-line

**FIGURE 10.6.4 LIMITATION FOR LINE OF ACTION OF THE AXIAL FORCE**

10.6.5 Design for biaxial bending and compression
A rectangular cross-section subject to axial force and bending moment acting simultaneously about each principal axis may be designed such that—

\[
\left[ \frac{M_{x}^{*}}{\phi M_{ux}} \right]^{\alpha_{n}} + \left[ \frac{M_{y}^{*}}{\phi M_{uy}} \right]^{\alpha_{n}} \leq 1.0 \quad \ldots 10.6.5(1)
\]

where

- \( \phi M_{ux} \) = design strength in bending, calculated separately, about the major and minor axes respectively under the design axial force \( N^{*} \)
- \( \phi M_{uy} \) = design bending moment about the major x-axis and minor y-axis respectively, magnified, if applicable
- \( M_{x}^{*}, M_{y}^{*} \) = design bending moment about the major x-axis and minor y-axis respectively
- \( \alpha_{n} \) = a coefficient

\[
\alpha_{n} = 0.7 + \frac{1.7N^{*}}{\phi N_{w}} \text{ within the limits } 1 \leq \alpha_{n} \leq 2 \quad \ldots 10.6.5(2)
\]

10.7 REINFORCEMENT FOR COLUMNS
10.7.1 Limitations on longitudinal steel
The cross-sectional area of the longitudinal reinforcement in a column shall—

(a) be not less than 0.01\(A_{g}\) except that in a column that has a larger area than that required for strength, a reduced value of \(A_{sc}\) may be used if—

\[
A_{scf_{w}} > 0.15N^{*}; \text{ and} \quad \ldots 10.7.1
\]

(b) not be greater than 0.04\(A_{g}\) unless the amount and disposition of the reinforcement will not prevent the proper placing and compaction of the concrete at splices and at junctions of the members.
10.7.2 Bundled bars
Groups of parallel longitudinal bars bundled to act as a unit shall have not more than 4 bars in any one bundle and be tied together in contact.

10.7.3 Restraint of longitudinal reinforcement

10.7.3.1 General
The following longitudinal bars in columns shall be laterally restrained in accordance with Clause 10.7.3.2:
(a) Single bars—
   (i) each corner bar;
   (ii) all bars where bars are spaced at centres of more than 150 mm; or
   (iii) at least every alternate bar where bars are spaced at 150 mm or less except that full restraint need not be provided in a column if—

\[
N^* \leq 0.5 \phi N_u \quad \ldots 10.7.3.1
\]

where the restraint provided shall be not less than the following:
(A) in horizontal sections, at least every sixth longitudinal (main) reinforcing bar shall be restrained but the spacing of restraints shall not exceed 1000 mm; and
(B) in the vertical direction the spacing between restraints shall not exceed 600 mm.

(b) Bundled bars, each bundle.
For columns with \( f'_c > 50 \text{ MPa} \), and where the design action effects satisfy the following:
   (i) \( N^* \geq 0.75 \phi N_u \); or
   (ii) \( N^* \geq 0.3 \phi f'_c A_g \) and \( M^* \geq 0.6 \phi M_u \)
core confinement shall be in accordance with Clauses 10.7.3.1 of AS 3600-2009 for special confinement regions.

10.7.3.2 Lateral restraint
Lateral restraint shall be deemed to be provided if the longitudinal column reinforcement is placed within and in contact with—
(a) a non-circular tie—
   (i) at a bend in the tie, where the bend has an included angle of 135° or less;
   (ii) between two 135° fitment hooks; or
   (iii) inside a single 135° fitment hook of a fitment that is approximately perpendicular to the column face.
(b) a circular tie or helix and the longitudinal reinforcement is equally spaced around the circumference.

10.7.3.3 Diameter and spacing of ties and helices
The diameter and spacing of ties and helices shall comply with the following:
(a) The bar diameter of the tie or helix shall be not less than that given in Table 10.7.3.
(b) The spacing of ties or the pitch of a helix shall be not greater than the smaller of—
   (i) \( D_c \) or \( 15d_b \) for single bars;
   (ii) \( 0.5D_c \) or \( 7.5d_b \) for bundled bars; or
   (iii) 300 mm.
where
\[ D_c = \text{smaller column dimension if rectangular or the column diameter if circular} \]
\[ d_b = \text{diameter of the smallest bar in the column} \]

(c) One tie or the first turn of a helix shall be located not more than 100 mm vertically above the top of a footing or the top of a slab. Another tie or the final turn of a helix shall be located not more than 50 mm vertically below the highest soffit of the members that frame into the column except that in a column with a capital, the tie or turn of the helix shall be located at a level at which the area of the cross-section of the capital is not less than twice that of the column.

(d) Welded wire fabric, having strength and anchorage equivalent to that required for bars, may be used.

### TABLE 10.7.3

<table>
<thead>
<tr>
<th>Longitudinal bar diameter mm</th>
<th>Minimum bar diameter of tie or helix mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single bars up to 20</td>
<td>6</td>
</tr>
<tr>
<td>Single bars 24 to 28</td>
<td>10</td>
</tr>
<tr>
<td>Single bars 32 to 36</td>
<td>12</td>
</tr>
<tr>
<td>Single bars 40</td>
<td>16</td>
</tr>
<tr>
<td>Bundled bars</td>
<td>12</td>
</tr>
</tbody>
</table>

#### 10.7.3.4 Detailing of lateral ties and helices

Detailing of ties and helices shall be as follows:

(a) A rectangular tie shall be spliced by welding, or by fixing two 135° fitment hooks around a bar or a bundle at a fitment corner. Internal ties may be spliced by lapping within the column core.

(b) A circular shaped tie shall be spliced either by welding, or by overlapping and fixing two 135° fitment hooks around adjacent longitudinal bars or bundles as shown in Figure 10.7.3.

(c) A helix shall be anchored at its end by one-and-one-half extra turns of the helix. It may be spliced within its length either by welding or by mechanical means.

(d) Where hooks or cogs are specified in combination with bundled bars, the internal diameter of the bend shall be increased sufficiently to readily accommodate the bundle.
10.7.3.5 Requirements for earthquake resistance

For bridge structures in BEDC-2, BEDC-3 and BEDC-4, special consideration shall be given to the detailing of concrete compression members, bearing in mind the manner in which earthquake-induced energy will be dissipated and the desirability of avoiding brittle failures, especially in shear. In particular, the ultimate shear capacity shall be assessed and additional capacity provided, where necessary, to ensure that premature failure does not occur.

A1 NOTE: The Clause does not apply to bridge structures in BEDC-1.

In reinforced and prestressed concrete compression members, the longitudinal reinforcement shall be restrained by lateral reinforcement in the potential plastic hinge regions as follows:

(a) Where helices are used, the area of the reinforcement in the helix, per unit length of member, shall be—

\[
\frac{A_s}{s} \geq \frac{0.03 f'_c D_c}{f_{y,t}} \quad \ldots 10.7.3.5(1)
\]

(b) Where closed ties are used (see Figure 10.7.3.2), the total cross-sectional area of the ties \((A_{sv})\), including supplementary cross-ties, shall be not less than—

(i) \(0.30 s y_1 \left( \frac{A_g}{A_c} - 1 \right) \left( f'_c / f_{y,t} \right)\); or \(\ldots 10.7.3.5(2)\)

(ii) \(0.09 s y_1 \left( f'_c / f_{y,t} \right)\), whichever is greater \(\ldots 10.7.3.5(3)\)

Where

\[
\begin{align*}
A_s &= \text{area of the reinforcement forming the helix} \\
A_g &= \text{area of the gross cross-section of the member} \\
A_c &= \text{area of the cross-section of the core measured over the outside of the ties} \\
s &= \text{centre-to-centre vertical spacing of ties} \\
f'_c &= \text{characteristic compressive cylinder strength of concrete at 28 days} \\
D_c &= \text{diameter of the inside face of the helix} \\
f_{y,t} &= \text{yield strength of the reinforcement used as fitments} \\
y_1 &= \text{core dimension of closed rectangular tie in the direction under consideration} \\
\end{align*}
\]
except that Item (b)(i) does not apply if $\phi N_{uo}$ for the core (concrete + reinforcement) is greater than $N'$.

**FIGURE 10.7.3.2  RECTANGULAR COLUMN TIE TERMS**

(c) Closed ties in accordance with Item (b) shall be used singly or in sets spaced vertically at not more than 150 mm centres, or one-quarter of the minimum cross-section dimension, whichever is smaller.

Supplementary ties, of the same diameter as the closed ties, consisting of a straight bar with a 135° minimum hook at each end, may be considered as part of a closed tie if they are spaced horizontally at not more than 350 mm centres and secured with the closed tie to the longitudinal bars.

(d) The lateral reinforcement in accordance with Item (a) or Item (b) shall extend into the footing, pilecap or deck, as applicable, over a length not less than half the maximum dimension of the compression member or 400 mm, whichever is greater.

The lateral reinforcement shall extend for a minimum distance of twice the maximum dimension of the compression member from the top and bottom of framed piers, or from the bottom of cantilever piers.

Piles may have potential plastic hinge positions at the top of the piles and at locations down the pile where there is an abrupt change in soil stiffness. The lateral reinforcement shall extend for a minimum distance of twice the maximum dimension of the pile from the bottom of the pile cap, or four times the maximum pile dimension centred about the hinge location.

**10.7.4  Splicing of longitudinal reinforcement**

10.7.4.1  General

Longitudinal reinforcement in columns shall be spliced in accordance with Clauses 10.7.4.2 to 10.7.4.5 and the splices shall comply with Clause 13.2.

10.7.4.2  Minimum tensile strength

At any splice in a column, a tensile strength in each face of the column of not less than $0.25 f_{yy} A_s$ shall be provided, where $A_s$ is the cross-section of longitudinal reinforcement at that face.

10.7.4.3  Where tensile force is greater than the minimum tensile strength

At any splice in a column where tensile stress exists and the tensile force in the longitudinal bars at any face of the column, due to strength design load effects, is greater than the minimum strength specified in Clause 10.7.4.2, the tensile force in the bars shall be transmitted by—
(a) a welded or mechanical splice (see also Clause 13.2.6); or
(b) a lap-splice in tension in accordance with Clause 13.2.2 or Clause 13.2.5.

10.7.4.4 End-bearing splice in compression

Where the splice is always in compression, the force in the longitudinal bar may be transmitted by the bearing of square-cut mating ends held in concentric contact by a sleeve, provided that an additional tie that complies with Clause 10.7.3 is placed above and below each sleeve. The bars shall be rotated to achieve the maximum possible area of contact between the ends of the bars, and the requirements of Clause 10.7.4.2 shall be met.

10.7.4.5 Offset bars

Where a longitudinal bar is offset to form a lap splice, the following shall be complied with:

(a) The slope of the inclined part of the bar in relation to the axis shall be not greater than 1 in 6.
(b) The portions of the bar on either side of the offset shall be parallel.
(c) Adequate lateral support shall be provided at the offset.

Where a column face is offset 75 mm or greater, longitudinal bars shall not be offset by bending but shall be lap-spliced with separate splicing bars placed adjacent to the offset column faces.

10.8 DESIGN OF TENSION MEMBERS

10.8.1 General

Tension members are members designed primarily to resist tensile axial loads or tensile axial loads combined with bending. They may occur in prestressed trusses, rigid frames and ties for various structures.

The stress of a tension member shall be such that—

\[ \phi N_u \geq N^* \]  

The strength and serviceability of a tension member shall be determined in accordance with—

(a) the basic principles specified in Clause 10.8.2; and
(b) the material properties specified in Section 6.

10.8.2 Basic principles

Calculations for strength and serviceability of cross-sections with tensile force, or with bending combined with tensile force, shall incorporate equilibrium and strain-compatibility considerations and shall be consistent with the following assumptions:

(a) Plane sections normal to the axis remain plane after bending.
(b) The concrete has no tensile strength except in the evaluation of tension stiffening effects for deflection calculations.
(c) The distribution of compressive stress in the concrete is determined by a recognized stress-strain relationship for the concrete in compression.
(d) Time-dependent deformation (creep and shrinkage) shall be considered in the calculation of deformation of a tension member.

NOTE: This total deformation may cause significant secondary moments in statically indeterminate structures.
11.1 APPLICATION

This Section applies to the design of planar walls, such as retaining walls, abutment walls and crash-resistant walls adjacent to both roads and railways.

11.2 DESIGN PROCEDURES

11.2.1 General

Planar walls shall be designed in accordance with Clauses 11.2.2 to 11.2.6, as appropriate. The reinforcement provided shall comply with Clause 11.6.

11.2.2 Walls subject only to in-plane vertical forces

Walls subject only to in-plane vertical forces shall be designed as columns in accordance with Section 10 if vertical reinforcement is provided in each face.

11.2.3 Walls subject to in-plane vertical and horizontal forces

Walls subject to vertical and horizontal forces in the plane of the wall shall be designed for vertical action effects in accordance with Clause 11.2.2 and horizontal action effects in accordance with AS 3600.

11.2.4 Walls subject principally to horizontal forces perpendicular to the wall

Walls subject to horizontal forces perpendicular to the plane of the wall and for which the design vertical force \( (V) \) at mid-height is not greater than \( 0.03 f'_{ck} A_g \) shall be designed as slabs in accordance with the appropriate clauses of Section 9, except that the ratio of effective height to thickness shall be not greater than 50, where the effective height shall be determined in accordance with Clause 11.4.

11.2.5 Walls subject to in-plane vertical forces and horizontal forces perpendicular to the wall

Walls subject to in-plane vertical forces and horizontal forces perpendicular to the plane of the wall shall be designed as columns in accordance with Section 10.

11.2.6 Walls forming part of a framed structure

Walls subject to axial forces, bending moments and shear forces arising from forces acting on the frame, shall be designed in accordance with Sections 9 and 10, as appropriate.

11.3 BRACING OF WALLS

Walls shall be assumed to be braced if they are laterally supported by a structure in which all of the following apply:

(a) Walls or vertical braced elements are arranged in two directions so as to provide lateral stability to the structure as a whole.

(b) Lateral forces are resisted by shear in the planes of these walls or by braced elements.

(c) Superstructures are designed to transfer lateral forces.

(d) Connections between the wall and the lateral supports are designed to resist a horizontal force not less than—

(i) the simple static reactions to the total applied horizontal forces at the level of lateral support; and

(ii) 2.5% of the total vertical load that the wall is designed to carry at the level of lateral support, but not less than 2 kN per metre length of wall.
11.4 SIMPLIFIED DESIGN METHOD FOR BRACED WALLS SUBJECT TO VERTICAL IN-PLANE LOADS ONLY

Braced walls that comply with Clause 11.3 and that are subject to vertical loads only shall be designed in accordance with AS 3600.

11.5 DESIGN OF WALLS FOR IN-PLANE HORIZONTAL FORCES

Where appropriate, walls subject only to in-plane horizontal forces in conjunction with vertical forces shall be designed in accordance with AS 3600.

11.6 REINFORCEMENT FOR WALLS

11.6.1 Minimum reinforcement

Walls shall have reinforcement not less than that specified in Clause 2.8.

11.6.2 Horizontal reinforcement for crack control

Where a wall is fully restrained from expanding or contracting horizontally due to shrinkage or temperature, the horizontal reinforcement ratio (As/bD) shall be not less than the following, as appropriate, where b is the wall height and D is the wall thickness:

- (a) For exposure classification A ................................................................. 0.0035.
- (b) For exposure classifications B1, B2 and C with reinforcement bars of—
  - (i) 16 mm or less .................................................................................... 0.006; and
  - (ii) 20 mm ............................................................................................ 0.008,

except that in no case shall the reinforcement ratio be less than that specified in Clause 11.6.1.

The reinforcement shall be placed equally on each face of the slab and located as close to each face as cover and detailing permit. D need not be taken as greater than 500 mm. Reinforcement provided for structural reasons can be considered as contributing to this requirement.

In walls and wall type piers, where the spacing of vertical contraction or expansion joints exceeds 8.0 m, the area of horizontal reinforcement for crack control shall be increased near the base of the wall. The area of horizontal reinforcement for crack control additional to that specified above shall be determined by the designer. As a minimum, the reinforcement ratio (As/bD) near the base of the wall up to a height equal to the thickness of the wall shall not be less than 1.33 times the values in Items (a) and (b) above.

NOTE: The designer should consider the timing of the stripping of the formwork, the autogenous and drying shrinkage of the concrete, and the differential shrinkage effects arising from the restraint of the wall at its base by the previously cast concrete footing.

Where vertical contraction or expansion joints are provided in the footing of the wall or in the pile cap in accordance with Clause 13.4.2 of AS 5100.3, matching vertical contraction joints or expansion joints at the same location shall also be provided in the wall.

11.6.3 Spacing of reinforcement

The minimum clear distance between parallel bars, ducts and tendons shall be sufficient to ensure that the concrete can be properly placed and compacted but shall be not less than 3db.

The maximum centre-to-centre spacing of parallel vertical bars shall be 1.5tw or 300 mm, whichever is the lesser, where tw is the thickness of the wall.

The vertical spacing of horizontal reinforcement shall not exceed 150 mm.

For walls greater than 200 mm thick, the vertical and horizontal reinforcement shall be provided in two grids, one near each face of the wall except that reinforcement need not be provided in a direction where it can be demonstrated that the face will always be in compression.
SECTION 12  DESIGN OF NON-FLEXURAL MEMBERS, END ZONES AND BEARING SURFACES

12.1  DESIGN OF NON-FLEXURAL MEMBERS

12.1.1  General

12.1.1.1  Application

This Section applies to the design of non-flexural members including deep beams, footings, pile caps, corbels, continuous nibs and stepped joints where the ratio of the clear span or projection to the overall depth is less than—

(a) for cantilevers ........................................................................................................ 1.5;
(b) for simply supported members ........................................................................ 3; and
(c) for continuous members .................................................................................... 4.

12.1.1.2  Design basis

The design for strength shall be carried out using one of the following:

(a) Linear elastic stress analysis and the checking procedure given in Clause 2.2.3.
(b) Strut-and-tie analysis, and the checking procedure given in Clause 2.2.4.

The value of the capacity reduction factor shall be determined from Clause 2.2, as appropriate for the analysis and checking procedure adopted.

12.1.1.3  Spacing of reinforcement

The clear distance between parallel bars, including bundled bars, ducts or tendons shall comply with Clause 8.1.7.

12.1.2  Strut-and-tie models for the design of non-flexural members

12.1.2.1  Design models

Design models are distinguished by the method in which the forces are transferred from the point of loading to the supports. The models are identified as Types I, II and III. These are shown in Figure 12.1.2.1 for the specific case of deep beams, and are defined as follows:

(a) Type I  The load is carried to the supports directly by major struts.

(b) Type II  The load is taken to the supports by a combination of primary (major) and secondary (minor) struts. Hanger reinforcement is required to return the vertical components of forces developed in the secondary struts to the top of the member.

(c) Type III  The load is carried to the supports via a series of minor struts with hanger reinforcement used to return the vertical components of the strut forces to the top of the member.

For Type II models, the force carried by the secondary struts shall be within the limits $0 \leq T_w \leq F$, where $T_w$ is the vertical component of the force carried by the secondary struts and $F$ is the total vertical component of the external load carried through the shear span.
12.1.2.2 Strut bursting reinforcement

Strut bursting reinforcement shall be provided in accordance with Appendix 12.4.

12.1.2.3 Additional requirements for continuous concrete nibs and corbels

Corbels and continuous nibs that support other members shall be also designed to comply with the following:
The tensile reinforcement shall be anchored at the free end of the nib or corbel, either by a welded or mechanical anchorage, or by a loop in either the vertical or horizontal plane. Where the main reinforcement is looped, the loaded area shall not project beyond the straight portion of this reinforcement.

Horizontal forces resulting from the supported member, because of factors, such as movement, shrinkage, temperature and prestress, shall be assessed but shall not be taken as less than 20% of the vertical force.

The line of action of the load shall be taken at the outside edge of a bearing pad for continuous nibs and at one third the width of the bearing from the free end for a corbel. Where no bearing pad is provided, the line of action may be taken at the commencement of any edge chamfer, or at the outside face of the nib or corbel as appropriate.

Where a flexural member is being supported, the outside face of a nib shall be protected against spalling.

Additional requirements for stepped joints in beams and slabs

The design of stepped joints shall take into account the horizontal forces and movements from the supported members and shall comply with the following:

Horizontal forces resulting from movement, shrinkage, temperature, prestress and other factors in the supported member shall be assessed but shall not be taken as less than 20% of the vertical force.

In prestressed members, the vertical component of the force from the prestressing steel shall be ignored.

The horizontal reinforcement shall extend at least a distance equal to the beam depth \((D)\) beyond the step and shall be provided with anchorage beyond the plane of any potential shear crack.

Hanger reinforcement shall be placed as close as possible to the vertical face of the step.

Empirical design methods

The ultimate strength of a non-flexural member may be based on design methods derived from test results, provided that the proportions of the member and the configurations of the reinforcement are within the range tested.

PRESTRESSING ANCHORAGE ZONES

Application

This Clause applies to the design of anchorage zones in prestressed concrete members.

An anchorage zone is the zone between the loaded face and the cross-section at which a linear distribution of stress due to prestress is achieved.

NOTE: Design procedures should be as described in Appendix G, which will clarify and supplement the requirements of Clause 12.2.

General

Reinforcement shall be provided to carry the tensile forces that arise from the action and dispersal of the prestressing forces in anchorage zones. In general, the dispersal occurs through both the depth and the width of the anchorage zone, and reinforcement shall therefore be provided in planes parallel to the end faces in two orthogonal directions. A two-dimensional analysis for each loading case shall be carried out in each direction in turn. The tensile forces shall be calculated on longitudinal sections through anchorages and on longitudinal sections where peak values of transverse moments occur.
The transverse moment on a longitudinal section shall be the moment acting on the free body bounded by the longitudinal section, a free surface parallel to it, the loaded face, and a plane parallel to the loaded face at the inner end of the anchorage zone.

The transverse moment shall be defined as positive if the resultant of the transverse compressive stresses is closer to the loaded face than the resultant of the transverse tensile stresses. A positive moment is one that produces bursting forces, that is, tensile forces are within the member. A negative moment is that which produces spalling forces, that is, tensile forces on the loaded face.

12.2.3 Loading cases

Loading cases to be considered shall include—

(a) all anchorages loaded; and

(b) critical loadings during the stressing operation.

Where the distance between two anchorages is less than 0.3 times the total depth or breadth of the member, consideration shall be given to the effects of the pair acting in a manner similar to a single anchorage subject to the combined forces.

12.2.4 Calculation of tensile forces along the line of an anchorage force

The bursting force resultant \( T \) of transverse tensile stresses induced along the line of action of an anchorage force shall be taken as follows:

\[
T = 0.33P(1 - k_r)
\]

where

\[
P = \text{maximum force occurring at the anchorage during jacking}
\]

\[
k_r = \text{ratio of the depth or breadth of an anchorage bearing plate to the corresponding depth or breadth of the symmetrical prism}
\]

The symmetrical prism is defined as a notional prism with an anchorage at the centre of its end face and with depth or breadth taken as twice the distance from the centre of an anchorage to the nearer concrete face or the distance from the centre of the anchorage to the centre of the nearest adjacent anchorage, whichever is the lesser.

12.2.5 Calculation of tensile forces induced near the loaded face

At longitudinal sections remote from a single eccentric anchorage, or between widely spaced anchorages, where the sense of the transverse moment indicates that the tensile stress resultant acts near the loaded face, i.e., spalling forces, the tensile force shall be calculated as follows:

(a) For a single eccentric anchorage The peak transverse moment shall be divided by a lever arm assumed to be one half the overall depth of the member.

(b) Between pairs of anchorages The peak transverse moment shall be divided by a lever arm assumed to be 0.6 times the spacing of the anchorages.

12.2.6 Quantity and distribution of reinforcement

Reinforcement shall be provided for the tensile forces derived in accordance with Clauses 12.2.4 and 12.2.5.

For bursting forces, where reinforcement is not near the concrete surface and there is additional surface reinforcement, the stress in the reinforcement shall be not greater than 200 MPa.

For spalling forces, where the reinforcement forms the surface layer of reinforcement on any face, the stress for this surface reinforcement shall be not greater than 150 MPa to control cracking. Reinforcement shall be adequately anchored to develop this stress.

This reinforcement shall be distributed as follows:
(a) Bursting reinforcement to resist the forces calculated in accordance with Clause 12.2.4 shall be distributed from $0.1d_{ef}$ to $1.0d_{ef}$ from the loaded face.

NOTE: The design of end zones for prestressing anchorages is explained in Appendix G.

Similar reinforcement shall be placed from the plane at $0.1d_{ef}$ to as near as practicable to the loaded face where $d_{ef}$ shall be equal to the depth or breadth of the symmetrical prism, as appropriate. Reinforcement provided for bursting may also be used for spalling reinforcement provided its position is suitable and it is adequately anchored.

(b) Spalling reinforcement to resist the forces calculated in accordance with Clause 12.2.5 shall be placed as close to the loaded face as is consistent with cover and compaction requirements.

At any plane parallel to the loaded face, the reinforcement shall be determined from the longitudinal section with the greatest reinforcement requirements at that plane, and shall extend over the full depth or breadth of the end zone.

12.2.7 Anchorage zones in pretensioned members

Bursting reinforcement is not generally required in pretensioned members.

To control horizontal cracking sufficient vertical stirrups shall be provided to resist at least 4% of the total prestressing force at transfer. To control vertical cracking the same area of steel shall be provided as horizontal stirrups, which shall be in addition to the vertical stirrups if control of both horizontal and vertical cracking is required. These stirrups shall be placed as spalling reinforcement in a length of 0.25 times the depth (width) of the member from the end face, with the last stirrup placed as close to the end face as practicable. Reinforcement shall be designed for a stress of 150 MPa.

Where tendons are grouped or where groups of tendons are widely spaced in the vertical (or horizontal) direction at the ends of a member, additional reinforcement determined in accordance with Clauses 12.2.4 to 12.2.6 shall be added to control horizontal or vertical cracking in the member.

Reinforcement shall be adequately anchored to develop the stress of 150 MPa in the reinforcement at any critical section.

NOTE: Critical sections are likely to be midway between groups of tendons, or where there is an abrupt reduction in cross-section, or between the tendon groups and the remaining tendon-free area of the cross-section.

12.2.8 Special reinforcement details in anchorage zones

In addition to the reinforcement required to resist bursting and spalling tensile forces, the following consideration shall also be given to the reinforcement required in other local zones of tensile stresses that may exist in the region of anchorages:

(a) Unstressed corners Corners that remain unstressed after stressing due to the gradual dispersion of the concentrated prestressing force from the anchor plate shall be adequately anchored to the prestressed member. These unstressed corners include those regions beyond the anchor plates around anchorage recesses, and the outer corners of cantilever slabs at the ends of post-tensioned members. Nominal longitudinal or diagonal reinforcement detailed to cross the planes of potential cracking shall be provided to secure these corners to the member.

(b) Internal anchorages Where internal anchorages (either dead end or stressing end) are cast into a member at intermediate locations (see Figure 12.2.8), tensile zones develop behind the anchorage with tensile stresses parallel to the tendons.

These stresses depend on the following:

(i) The magnitude of the anchored prestress forces.

(ii) The magnitude of the compressive stress in the longitudinal direction.
(iii) The ratio of the area of the anchorage to the total cross-sectional area of the prestressed member.

As a general rule, special reinforcement designed to resist from 20% to 50% of the prestress force in the tendon, depending on the influence of the factors specified in Items (i), (ii) and (iii), shall be provided as shown in Figure 12.2.8. Such reinforcement shall extend at least over a length of $2D$ as shown in Figure 12.2.8 and have sufficient length to develop the yield stress ($f_{sy}$) of the reinforcing bar, as specified in Section 13, at the anchorage.

(c) **External anchorages** Where external anchorages, that is, anchorages located on a protruding bracket on the member, are used, reinforcement, in addition to that provided to resist the bursting tensile forces, shall be designed, where applicable to—

(i) resist tension caused by curvature of tendons;

(ii) provide shear connection to the main member and cater for the distribution of the prestress force into the main member;

(iii) resist the forces as described in Item (b); and

(iv) resist tension caused by local eccentricity of prestress force.

---

**FIGURE 12.2.8 TYPICAL REINFORCEMENT DETAILS AT AN INTERNAL ANCHORAGE**

**12.3 BEARING SURFACES**

Unless special confinement reinforcement is provided, the design bearing stress at a concrete surface shall be not greater than—

(a) $0.85 f'_c \sqrt{\frac{A_4}{A_3}}$; or

(b) $2 f'_c$, whichever is the lesser

where

$A_4 = \text{largest area of the supporting surface that is geometrically similar to and concentric with } A_3$

$A_3 = \text{bearing area}$

In the case of a bearing surface where the supporting structure is sloped or stepped, it shall be permissible to take $A_4$ as the area of the base of the largest frustum of a right pyramid or cone—
(i) having for its opposite end the bearing area $A_3$;

(ii) having side slopes of 1 longitudinally to 2 transversely, with respect to the direction of the load; and

(iii) contained wholly within the supporting structure.

Where bearing areas are subject to high edge loading by the bearing plate, the design bearing stress shall be not greater than 0.7 times the values specified in Items (a) and (b) (see also Section 8).

12.4 CRACK CONTROL

The requirements of crack control may be deemed to be satisfied if the stress in the reinforcement is not greater than the following:

(a) Where a minor degree of control over cracking is required............................250 MPa.

(b) Where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view.............................................................200 MPa.

(c) Where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes..........................................................................150 MPa.

For prestressed concrete, the change in stress in the tendons after the point of decompression shall not exceed the limits given by Items (a), (b) or (c), as appropriate.
SECTION 13 STRESS DEVELOPMENT AND SPLICING OF REINFORCEMENT AND TENDONS

13.1 STRESS DEVELOPMENT IN REINFORCEMENT

13.1.1 General

The calculated force in reinforcing steel at any cross-section shall be developed on each side of that cross-section in accordance with Clauses 13.1.2 to 13.1.8, as appropriate.

13.1.2 Development length for a deformed bar in tension

13.1.2.1 Development length to develop yield strength

The development length ($L_{sy,t}$) to develop the characteristic yield strength ($f_{sy}$) of a deformed bar in tension shall be calculated from either Clause 13.1.2.2 or 13.1.2.3.

13.1.2.2 Basic development length

The development length ($L_{sy,t}$) shall be taken as the basic development length of a deformed bar in tension ($L_{sy, tb}$), calculated from—

$$L_{sy, tb} = \frac{0.5k_1k_2f_{sy}d_b}{k_2\sqrt{f'\varepsilon}} \geq 29k_1d_b$$  

where

$$k_1 = \begin{cases} 1.3 & \text{for a horizontal bar with more than 300 mm of concrete cast below the bar;} \\ 1.0 & \text{otherwise} \end{cases}$$

$$k_2 = \frac{(132 - d_b)}{100}, \text{and}$$

$$k_3 = 1.0 - 0.15(c_d - d_b) / d_b \text{ (within the limits 0.7 \leq k_3 \leq 1.0); where}$$

$c_d$ = a dimension (in millimetres), as shown in Figure 13.1.2.3(A).

The value of $f'\varepsilon$ used in Equation 13.1.2.2 shall not be taken to exceed 65 MPa; and the bar diameter ($d_b$) is in millimetres.

The value of $L_{sy, tb}$ calculated as above shall be—

(a) multiplied by 1.5 for epoxy-coated bars; and

(b) multiplied by 1.3 when lightweight concrete is used; and

(c) multiplied by 1.3 for all structural elements built with slip forms.

NOTE: A smaller value of $L_{sy,t}$ may be possible using the provisions of Clause 13.1.2.3.
13.1.2.3 Refined development length

Where a refined development length is required, the development length in tension \( (L_{sy,t}) \) shall be calculated from—

\[
L_{sy,t} = k_4 k_5 L_{sy,tb}
\]

where

\[
k_4 = 1.0 - K \lambda \quad \text{(within the limits } 0.7 \leq k_4 \leq 1.0)\]

\[
k_5 = 1.0 - 0.04 \rho_p \quad \text{(within the limits } 0.7 \leq k_5 \leq 1.0)\]

\[
K = \text{a factor that accounts for the position of the bars being anchored with respect to the transverse reinforcement, with values as shown in Figure 13.1.2.3(B)}
\]

\[
\lambda = \frac{(\Sigma A_{tr} - \Sigma A_{tr.min})}{A_s}
\]

\[
\Sigma A_{tr} = \text{sum of cross-sectional area of the transverse reinforcement along the development length } (L_{sy,t})
\]

\[
\Sigma A_{tr.min} = \text{sum of cross-sectional area of the minimum transverse reinforcement, which may be taken as } 0.25 A_s \text{ for beams or columns and } 0 \text{ for slabs or walls}
\]

\[
A_s = \text{cross-sectional area of a single bar of diameter } (d_b) \text{ being anchored}
\]

\[
\rho_p = \text{transverse compressive pressure (in MPa), at the ultimate limit state along the development length and perpendicular to the plane of splitting}
\]

The product \( k_3 k_4 k_5 \) shall be not taken as less than 0.7.
13.1.2.4 Development length to develop less than the yield strength

The development length ($L_{st}$) to develop a tensile stress ($\sigma_{st}$), less than the yield strength ($f_{sy}$), shall be calculated from—

$$L_{st} = L_{st} \frac{\sigma_{st}}{f_{sy}}$$

but shall be not less than—

(a) $12d_b$; or

(b) for slabs, as permitted by Clause 9.1.1.
13.1.2.5 Development length around a curve

Tensile stress may be considered to be developed around a curve if the internal diameter of the curve is $10d_b$ or greater.

13.1.2.6 Development length of a deformed bar with a standard hook or cog

Where a deformed bar ends in a standard hook or cog complying with Clause 13.1.2.7, the tensile development length of that end of the bar, measured from the outside of the hook/cog, shall be taken as $0.5L_{sy,t}$ or $0.5L_{st}$ as applicable (as shown in Figure 13.1.2.6).

**FIGURE 13.1.2.6 DEVELOPMENT LENGTH OF A DEFORMED BAR WITH A STANDARD HOOK OR COG**

13.1.2.7 Standard hooks and cogs

The standard hook or cog referred to in Clause 13.1.2.6 shall be one of the following:

(a) A hook consisting of a 180° bend with a nominal internal diameter complying with Clause 16.2.3.2 plus a straight extension of $4d_b$ or 70 mm, whichever is greater.

(b) A hook consisting of a 135° bend with the same internal diameter and length as Item (a).

(c) A cog, consisting of a 90° bend with a nominal internal diameter complying with Clause 16.2.3.2 but not greater than $8d_b$ and having the same total length as required for a 180° hook of the same diameter bar.

13.1.3 Development length of plain bars in tension

The development length ($L_{sy,t}$) to develop the yield strength ($f_{sy}$) of a plain bar in tension shall be taken as the basic development length calculated in accordance with Clause 13.1.2.2 multiplied by 1.5, but $L_{sy,t}$ shall be not less than 300 mm. Where a plain bar ends in a standard hook or cog complying with Clause 13.1.2.7, the tensile development length of that end of the bar, measured from the outside of the hook/cog, shall be taken as $0.5L_{sy,t}$ or $0.5L_{st}$ as applicable (as shown in Figure 13.1.2.6).

13.1.4 Development length of headed reinforcement in tension

A head used to develop a deformed bar in tension shall consist of a nut or plate, having either a round, elliptical or rectangular shape, attached to the end(s) of the bar by welding, threading or swaging of suitable strength to avoid failure of the steel connection at ultimate load.
In addition—

(a) the bar diameter \( (d_b) \) shall not exceed 40 mm;
(b) the density of the concrete shall be greater than 2100 kg/m\(^3\);
(c) the net bearing area of head shall be not less than 4 times the cross-sectional area of the bar;
(d) the clear cover for the bar shall not be less than 2\(d_b\); and
(e) the clear spacing between bars shall be not less than 4\(d_b\).

If the cross-sectional area of the head of the headed reinforcement, or the area of the end plate for deformed bars mechanically anchored with an end plate in the plane perpendicular to the axis of the bar, is at least 10 times the cross-sectional area of the bar, the bar shall be considered to have a development length \( (L_{sy,t}) \) measured from the inside face of the head equal to 0.4 \( L_{sy,t} \) of a bar of the same diameter.

### 13.1.5 Development length of deformed bars in compression

#### 13.1.5.1 Development length to develop yield strength

The development length \( (L_{sy,c}) \) to develop the characteristic yield strength \( (f_{sy}) \) of a deformed bar in compression shall be calculated from either Clause 13.1.5.2 or Clause 13.1.5.3, but shall be not less than 200 mm.

#### 13.1.5.2 Basic development length

The development length \( (L_{sy,c}) \) shall be taken as the basic development length of a deformed bar in compression \( (L_{sy,cb}) \) calculated from—

\[
L_{sy,cb} = \frac{0.22 f_{sy}}{\sqrt{f_c'}} d_b \geq 0.0435 f_{sy} d_b \text{ or } 200 \text{ mm, whichever is the greater} \quad . . . 13.1.5.2
\]

NOTE: A smaller value of \( L_{sy,c} \) may be obtained using the provisions of Clause 13.1.5.3.

#### 13.1.5.3 Refined development length

Where a refined development length is required, the development length in compression \( (L_{sy,c}) \) shall be calculated from—

\[
L_{sy,c} = k_6 L_{sy,cb} \quad . . . 13.1.5.3
\]

Where transverse reinforcement with at least 3 bars, transverse to and outside the bar being developed is provided within \( L_{sy,cb} \) and when \( \Sigma A_{tr} / s \geq A_s / 600 \),

\[
k_6 = 0.75
\]

where \( \Sigma A_{tr} \) and \( A_s \) are defined in Clause 13.1.2.3.

In all other cases, \( k_6 = 1.0 \).

#### 13.1.5.4 Development length to develop less than the yield strength

The development length \( (L_{sc}) \) to develop a compressive stress \( (\sigma_{sc}) \), less than the yield strength \( (f_{sy}) \), shall be calculated from—

\[
L_{sc} = L_{sy,c} \frac{\sigma_{sc}}{f_{sy}} \quad \text{(but not less than 200 mm)} \quad . . . 13.1.5.4
\]

A bend or a standard hook shall not be considered effective in developing stress in reinforcement in compression.
13.1.6 Development length of plain bars in compression

The development length for plain bars in compression shall be twice the calculated value of $L_{sy,c}$ or $L_{sy,ch}$ for a deformed bar.

13.1.7 Development length of bundled bars

The development length of a unit of bundled bars shall be based on the development length required for the largest bar within the bundle increased by—

(a) for a 3-bar bundle ......................................................... 20%; and

(b) for a 4-bar bundle ......................................................... 33%.

13.1.8 Development length of welded plain or deformed mesh in tension

13.1.8.1 Development length to develop yield strength

The development length ($L_{sy,t}$) of welded plain or deformed mesh, measured from the critical section to the end of the bar or wire, shall be calculated in accordance with Clause 13.1.8.2, Clause 13.1.8.3 or 13.1.8.4, as appropriate.

13.1.8.2 Two or more cross-bars within development length

The yield strength of plain or deformed bars of welded mesh shall be considered to be developed by embedding at least 2 cross-bars spaced at not less than 100 mm or 50 mm apart within the development length for plain or deformed bars respectively, with the first one not less than 50 mm from the critical section.

13.1.8.3 One cross-bar within development length

When only one cross-bar is located within the development length, the minimum length measured from the critical section to the outermost cross-bar shall be not less than $L_{sy, tb}$ calculated from—

$$L_{sy, tb} = 3.25 \frac{A_b}{s_m} \frac{f_{yw}}{f_c}$$  \hspace{1cm} \ldots 13.1.8.3

but not less than 150 mm for plain mesh and not less than 100 mm for deformed mesh, where

$A_b =$ area of the individual bar being developed in square millimetres

$s_m =$ spacing of bars being developed, in millimetres

13.1.8.4 No cross-bars within development length

When no cross-bars are located within the development length, the development length of welded mesh shall be determined by Clauses 13.1.2 and 13.1.3, as appropriate.

13.1.8.5 Development length to develop less than the yield strength

The development length ($L_{st}$) to develop a tensile stress ($\sigma_{st}$) less than the yield strength ($f_{sy}$) shall be calculated from the development length of Clauses 13.1.8.3 or 13.1.8.4 using the following equation:

$$L_{st} = L_{sy, tb} \frac{\sigma_{st}}{f_{sy}}$$  \hspace{1cm} \ldots 13.1.8.5

but not less than 150 mm for plain mesh and not less than 100 mm for deformed mesh.
13.2 SPlicing OF REINFORCEMENT

13.2.1 General

The following general requirements shall apply to the splicing of reinforcement:

(a) Splices of reinforcement shall be made only as required or permitted on the design drawings or in specifications.

(b) The splice shall be made by welding, by mechanical means, by end-bearing, or by lapping.

(c) Splicing of reinforcement shall take into account the requirements of Clause 17.1.3 regarding the placement of concrete.

(d) Splices required in bars in tension-tie members shall be made only by welding or mechanical means.

(e) Lapped splices shall not be used for bars in compression or tension with diameter larger than 40 mm.

(f) Welding of reinforcing bars shall not be made less than 3\(d_b\) from that part of a bar that has been bent and re-straightened.

13.2.2 Lapped splices for bars in tension

In elements or members where the bars being lapped are in contact, the tensile lap length \((L_{sy,t,lap})\) shall be calculated from—

\[
L_{sy,t,lap} = k_7 L_{sy,t} \geq 29 k_1 d_b
\]

where

\(L_{sy,t}\) is calculated in accordance with Clause 13.1.2.1. (in the determination of \(L_{sy,t}\) for use in Equation 13.2.2, the lower limit of \(29k_1d_b\) in Equation 13.1.2.2 does not apply); and

\(k_7\) shall be taken as 1.25 unless \(A_s\) provided is at least twice \(A_s\) required and no more than half of the reinforcement at the section is spliced, in which case \(k_7\) may be taken as 1.0.

In elements or members where the bars are not in contact, the tensile lap length \((L_{sy,t,lap})\) shall be not less than the larger of \(29k_1d_b\), \(k_7 L_{sy,t}\) and \(L_{sy,t} + 1.5s_b\), where \(s_b\) is the clear distance between bars of the lapped splice as shown in Figure 13.2.2. However, if \(s_b\) does not exceed \(3d_b\), then \(s_b\) may be taken as zero for calculating \(L_{sy,t,lap}\).
13.2.3 Lapped splices for mesh in tension

A lapped splice for welded mesh in tension shall be made so the two outermost cross-bars spaced at not less than 100 mm or 50 mm apart for plain or deformed bars, respectively, of one sheet of mesh overlap the two outermost cross-bars of the sheet being lapped as shown in Figure 13.2.3. The minimum length of the overlap shall equal 100 mm.

A lapped splice for welded deformed and plain meshes, with no cross-bars within the splice length shall be determined in accordance with Clause 13.2.2.

13.2.4 Lapped splices for bars in compression

The minimum length of a lapped splice for deformed bars in compression shall be the development length in compression ($L_{sy,c}$) given in Items (a), (b) or (c), as appropriate, but shall be not less than 300 mm:

(a) The development length in compression shall be in accordance with Clause 13.1.5 but not less than $40d_b$.

(b) In compressive members with stirrups or fitments where at least 3 sets of fitments are present over the length of the lap and $A_u/s \geq A_b/1000$, a lap length of 0.8 times the value given in Item (a).
(c) In helically tied compressive members, if at least 3 turns of helical reinforcement are present over the length of the lap and \( \frac{A_t}{s} \geq n \frac{A_{b}}{6000} \), a lap length of 0.8 times the value given in Item (a), where \( n \) = the number of bars uniformly spaced around the helix.

In this Clause, \( A_{b} \) is defined as the area of the bar being spliced.

13.2.5 Lapped splices for bundled bars

Lapped splices for a unit of bundled bars shall be based on the lap splice length required for the largest bar within the bundle increased by—

(a) for a three bar bundle .................................................................................... 20%; and
(b) for a four bar bundle ............................................................................................ 33%.

Individual bar splices within a bundle shall not overlap.

13.2.6 Welded or mechanical splices

Welded or mechanical splices formed between Ductility Class N reinforcing bars shall not fail prematurely in tension or compression before the reinforcing bars, unless it can be shown that the strength and ductility of the concrete member meets the design requirements.

When control of cracking or vertical deflection are relevant serviceability design criteria, the potentially detrimental effects of excessive longitudinal slip between spliced Ductility Class N bars and a proprietary mechanical connector shall be considered if tests show the effective slip in the assemblage could exceed 0.1 mm at a tensile stress of 300 MPa. The effective slip shall be taken as the overall deformation of a spliced pair of reinforcing bars measured over a gauge length of \( 12d_{b} \), less the elongation of the bars assuming they are unspliced over the same gauge length.

13.3 STRESS DEVELOPMENT IN TENDONS

13.3.1 General

In the absence of substantiated test data, the length to develop the calculated force in a pretensioned tendon shall be taken to be a bi-linear relationship defined by the transmission length (\( L_{pt} \)) in Clause 13.3.2.1 and the total development length (\( L_{p} \)) in Clause 13.3.2.2.

13.3.2 Transmission lengths of pretensioned tendons

13.3.2.1 Transmission lengths of pretensioned tendons

The transmission length required to develop the effective prestress in pretensioned tendons shall be taken as the length given in Table 13.3.2, as appropriate to type of tendon and strength of concrete at transfer. The transmission length shall be taken to be independent of the effective prestress in the tendon.

It shall be assumed that no change in the position of the inner end of the transmission length occurs with time but that a completely unstressed zone of length \( 0.1L_{pt} \) develops at the end of the tendon.
TABLE 13.3.2
MINIMUM TRANSMISSION LENGTH FOR PRETENSIONED TENDONS

<table>
<thead>
<tr>
<th>Type of tendon</th>
<th>( L_p ) for gradual release</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( f_{tp} \geq 32 \text{ MPa} )</td>
</tr>
<tr>
<td>Indented wire</td>
<td>100 ( d_b )</td>
</tr>
<tr>
<td>Crimped wire</td>
<td>70 ( d_b )</td>
</tr>
<tr>
<td>Ordinary and compact strand</td>
<td>60 ( d_b )</td>
</tr>
</tbody>
</table>

13.3.2.2 Development length of pretensioned strand

In absence of test data, the bonded length to develop the stress in seven wire pretensioned strand at ultimate strength shall be taken as not less than—

\[
L_p = 0.145(\sigma_{pu} - 0.67\sigma_{p,ef})d_b \geq 60d_b
\]

where \( \sigma_{p,ef} \) is the effective stress in the tendon after allowing for all losses.

Both \( \sigma_{pu} \) and \( \sigma_{p,ef} \) are in megapascals, and the expression in parenthesis is used without units.

Embedment less than the development length is permitted at a section of a member provided the design stress in the strand at that section does not exceed the values obtained from the bi-linear relationship defined by this Clause and Clause 13.3.2.1.

The development length of de-bonded strand shall be taken to be 2\( L_p \) where the design includes tension in accordance with Clauses 8.6.2 and 9.4.2 in the development length.

13.3.2.3 Development length of pretensioned wire

Pretensioned indented and crimped wire tendons shall be bonded beyond the critical section for a length sufficient to develop the design stress in the wire but not less than 2.25 times the value for the transmission length in Table 13.3.2 as appropriate.

13.3.2.4 Development length of untensioned strand or wire

Where strand or wire is untensioned, the development length shall be taken as not less than 2.5 times the value of the appropriate transmission length of a stressed tendon given in Table 13.3.2 for a tendon stressed to the tensile strength \( f_{pb} \) in Table 3.3.1.

13.3.3 Stress development in post-tensioned tendons by anchorages

Anchorages for tendons shall be capable of developing in the tendon the minimum tensile strength \( f_{pb} \).

In addition, anchorages for unbonded tendons shall be capable of sustaining cyclic loading conditions.

13.4 COUPLING OF TENDONS

Coupling of tendons shall comply with the following:

(a) Couplers shall be capable of developing at least 95% of the tendon characteristic minimum breaking force specified.

(b) Couplers shall be enclosed in grout-tight housings to facilitate grouting of the duct.
SECTION 14 JOINTS, EMBEDDED ITEMS, FIXING AND CONNECTIONS

14.1 DESIGN OF JOINTS

14.1.1 Construction joints

A construction joint, including a joint between precast segments, in a part of a structure or member shall be designed and constructed so that the load-carrying capacity, serviceability and durability of the structure or member will be satisfactory.

14.1.2 Movement joints

A movement joint in a part of a structure or member shall be designed and constructed in accordance with AS 5100.4, so that the assessed relative movement or rotation, between the parts of the structure or member on either side of the joint, can occur without impairing the load-carrying capacity and serviceability of the structure or member.

14.2 EMBEDDED ITEMS AND HOLES IN CONCRETE

14.2.1 General

Embedded items and holes shall be permitted in concrete members provided that the member has the required strength, serviceability and durability.

For the purpose of this Section, embedded items shall include pipes and conduits with their associated fittings, sleeves, permanent inserts for fixings and other purposes, anchor bolts, bar chairs and other supports.

For the purpose of this Section, holes shall include holes through a member, holes along the length of a member, rebates and penetrations.

14.2.2 Limitations of materials

The materials to be embedded shall comply with the following, as appropriate:

(a) Conduits and pipes used for electrical purposes shall comply with the relevant requirements of AS/NZS 3000.

(b) Other embedded items shall be protected from corrosion or deterioration.

(c) Metals such as aluminium shall not be embedded in structural concrete unless effectively coated, covered or treated to prevent chemical action between the metal and concrete, and electrolytic action between the metal and steel.

14.2.3 Pipes containing liquid, gas or vapour

Pipes that are intended to contain liquid, gas or vapour under pressure and extremes of temperature may be embedded in structural concrete provided that—

(a) the pressure to which any piping or fitting is intended to be subjected shall not exceed 2 000 kPa; and

(b) the effect that inclusion of the pipe has on the behaviour of the member is taken into account.

NOTE: AS 5100.1 provides requirements for the accommodation of public utilities on bridges.

14.2.4 Spacing and cover

The clear distance between embedded items and between embedded items and bars, including bundled bars, tendons or ducts, shall be sufficient to ensure that the concrete can be placed to comply with Clause 8.1.7.
The cover to embedded items that are not corrosion resistant shall comply with Clause 4.10.3.

**14.3 REQUIREMENTS FOR FIXINGS**

Fixings shall be designed to have adequate strength, serviceability, and durability.

Fixings, including anchor bolts, inserts, and ferrules shall comply with the following:

(a) A fixing shall be designed to transmit all forces, acting or likely to act on it.

(b) Forces on fixings used for lifting purposes shall include an impact factor for assessing the load.

(c) Fixings shall be designed to yield before ultimate failure in the event of overload.

(d) The anchorage of any fixings shall be designed in accordance with Section 13 as appropriate. The design strength of this anchorage shall be taken as $\phi$ times the ultimate strength where $\phi$ equals 0.5. In the case of shallow anchorages, cone-type failure in the concrete surrounding the fixing shall be investigated and shall take into account edge distance, spacing, and the effect of reinforcement if any, and concrete strength at the time of loading.

(e) In the absence of calculations, the strength of a fixing shall be determined by load testing of a prototype to failure in accordance with Clause 17.2. The design strength of the fixing shall be taken as $\phi$ times the ultimate strength where the ultimate strength is taken as the average failure load divided by the appropriate factor for isolated elements given in Table 17.4.3 and $\phi$ equals 0.6.

(f) The spacing between and cover to fixings shall be in accordance with Clause 14.2.4.

**14.4 CONNECTIONS**

Monolithic connections between structural members shall be detailed to transmit the design action effects including allowances for any possible reversals of actions or action effects.
SECTION 15  PLAIN CONCRETE MEMBERS

15.1 APPLICATION
Plain concrete shall be used only for members in which cracks will not induce collapse. The requirements of this Section may be applied to plain concrete floors and pavements resting on the ground, footings, gravity retaining walls and bored piles.

15.2 DESIGN
15.2.1 Basic principles of strength design
Members shall be designed in accordance with the following:
(a) Design of members for flexure shall be based on a linear stress-strain relationship in both tension and compression.
(b) The tensile strength of concrete may be considered in the design.
(c) No tensile strength shall be assigned to reinforcement that may be present.

15.2.2 Section properties
In the calculation of strength, the entire cross-section of a member shall be considered except that for a member cast against soil, the overall relevant dimensions shall be taken as 50 mm less than the actual dimension.

15.3 STRENGTH IN BENDING
The design strength of a member in bending shall be taken as $\phi M_{uo}$, where $M_{uo}$ shall be calculated using the characteristic flexural tensile strength $(f'_{ct})$.

15.4 STRENGTH IN SHEAR
15.4.1 One-way action
Where the member acts essentially as a one-way member, and a shear failure can occur across the width ($b$) of the member, the design strength in shear shall be taken as $\phi V_u$ where—
$$V_u = 0.15bD(f'_{ct})^{1/3}$$ . . . 15.4.1

The maximum shear shall be taken to occur at a distance $0.5D$ from the face of a support.

15.4.2 Two-way action
Where a shear failure occurs locally around a support or loaded area, the design strength in shear shall be taken as follows:
$$\frac{\phi V_u}{1 + \left(\frac{u M^*}{8V^*aD}\right)}$$ . . . 15.4.2(1)

where
$$V_u = 0.1uD\left(1 + \frac{2}{\beta_h}\right)\sqrt{f'_{ct}}$$ . . . 15.4.2(2)
$$\leq 0.2udD\sqrt{f'_{ct}}$$ . . . 15.4.2(3)

$u =$ effective length of the critical shear perimeter (see Figure 9.2.3)
\( a = \) dimension of the critical shear perimeter, which is parallel to the direction of bending being considered (see Figure 9.2.3)

\( \beta_h = \) ratio of the longest overall dimension of the effective loaded area \((Y)\) to the shortest overall dimension \((X)\) measured perpendicular to \(Y\), as specified in Clause 9.2.3

15.5 **STRENGTH IN AXIAL COMPRESSION**

The design strength under axial compression of a member, other than a wall, shall be taken as \( \phi N_{uo} \) where—

\[
N_{uo} = 0.45 f'_c A_g
\]

provided that the unsupported length of the member shall be not greater than three times the least lateral dimension except that this restriction shall not apply to in-place bored piles.

15.6 **STRENGTH IN COMBINED BENDING AND COMPRESSION**

In the absence of more exact calculations, members subject to combined bending and axial load shall be designed so that—

\[
\frac{M_x^*}{\phi M_{ux}} + \frac{M_y^*}{\phi M_{uy}} + \frac{N_r^*}{\phi N_u} \leq 1
\]

15.7 **REINFORCEMENT AND EMBEDDED ITEMS**

The concrete cover to any reinforcement or embedded item, and the clear distance between these items shall comply with Sections 4 and 14.
SECTION 16  MATERIAL AND CONSTRUCTION REQUIREMENTS

16.1 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR CONCRETE AND GROUT

16.1.1 Materials and limitations on constituents

Materials for concrete and grout and limitations on their chemical content shall comply with the relevant requirements of AS 1379 and this Clause.

Where control of shrinkage or creep, or both, is a design requirement for concretes manufactured with type GP or HE cements, the acid-soluble sulfate content of the concrete from all mix sources, expressed as the proportion of SO₃ by mass, shall be not less than 20 g/kg of cement.

16.1.2 Specification and manufacture of concrete

Concrete to which this Standard applies shall be—

(a) specified as either normal-class or special-class and manufactured and supplied in accordance with AS 1379; and

(b) handled, placed, compacted, finished and cured in accordance with this Standard, so that the hardened concrete will satisfy the design requirements for strength, serviceability and durability.

Concrete to which this Standard applies shall not be subjected to its design load until the concrete has attained the 28 days strength nominated in the design.

Project assessment shall be specified for special-class concrete specified by strength grade and may be specified for normal-class concrete and other special-class concrete, all as defined in AS 1379.

16.1.3 Handling, placing and compacting of concrete

Concrete shall be handled, placed and compacted so as to—

(a) limit segregation or loss of materials;

(b) limit premature stiffening;

(c) produce a monolithic mass between planned joints or the extremities of members, or both;

(d) completely fill the formwork to the intended level, expel entrapped air, and closely surround all reinforcement, tendons, ducts, anchorages and embedments; and

(e) provide the specified finish to the formed surfaces of the member.

16.1.4 Finishing of unformed concrete surfaces

Unformed concrete surfaces shall be finished by appropriate methods, to achieve the specified—

(a) dimensions, falls, tolerances, or similar details relating to the shape and uniformity of the surfaces;

(b) cover from the surfaces to reinforcement, tendons, ducts and embedments; and

(c) texture of the surface.
16.1.5 Curing and protection of concrete

16.1.5.1 Curing

Concrete shall be cured continuously for a period of time that ensures that the design requirements for strength, serviceability and stripping are satisfied. To satisfy durability requirements, the initial curing periods shall be not less than those given in Clauses 4.4 to 4.6.

Curing shall be achieved by the application of water or steam to, or the retention of water in, the freshly cast concrete and shall commence as soon as practicable after finishing of any unformed surfaces has been completed. Where retention of water in the fresh concrete relies on the application to exposed surfaces of sprayed membrane-forming curing compounds, the compounds used shall comply with AS 3799.

Unless specified otherwise, curing shall continue for at least 7 days for concrete using type GP cement and 10 days for concrete using type GB cement.

16.1.5.2 Protection

Freshly cast concrete shall be protected from the effects of rain, running water and freezing or drying prior to hardening. During the initial curing period the concrete shall be protected from freezing or drying.

16.1.6 Sampling and testing for compliance

16.1.6.1 General

Concrete that is intended for use in structures designed in accordance with this Standard shall be assessed in accordance with AS 1379 for compliance with the specified parameters.

NOTE: When project assessment is required, the project specification should nominate responsibility for carrying out the relevant sampling, testing and assessment and, if these differ from or are not covered by AS 1379, should give details of how the assessment is to be made.

16.1.6.2 Concrete specified by strength grade

Concrete specified by strength grade shall satisfy the following criteria:

(a) For each strength grade of concrete supplied to a project, the mean grade strength, as defined in AS 1379, shall be maintained within the limits specified in that Standard.

(b) For concrete subject to project assessment—

(i) the slump of the supplied concrete shall be within the tolerance specified in AS 1379 for the relevant specified slump; and

(ii) in addition to Item (a), the mean compressive strength of the representative samples taken from the project shall be statistically consistent with the relevant specified characteristic strength.

NOTES:

1 ‘Strength grade’ is defined in AS 1379 as ‘the specified value of the characteristic compressive strength of the concrete at 28 days’.

2 The compressive strength of the concrete sampled, tested and assessed in accordance with AS 1379 indicates the potential strength of the supplied concrete, when placed, compacted and cured under optimum conditions; the responsibility of demonstrating resting on the supplier. The achievement of that potential on site is dependent upon the handling, placing, compacting and curing techniques actually used; the responsibility for which rests with the construction contractor (see Clauses 16.1.3 and 16.1.5). Information on appropriate site techniques may be found in HB 64 and HB 67.
16.1.6.3 Concrete specified by parameters other than strength grade

When concrete is specified by parameters other than strength grade, the method of production control and, if required, project control shall be specified together with the relevant compliance criteria.

The specified methods of control and assessment shall provide a reliable operating characteristic curve so that—

(a) concrete with a proportion defective of 0.05 has a probability of acceptance of not less than 50%; and

(b) concrete with a proportion defective of 0.30 has a probability of rejection of not less than 98%.

16.1.7 Rejection of concrete

16.1.7.1 Plastic concrete

Plastic concrete may be rejected if, after completion of mixing but prior to site handling—

(a) the slump, determined in accordance with AS 1012.3, differs from the specified slump by more than the tolerances permitted in AS 1379;

(b) the elapsed time from the first introduction of the mixing water is outside the time interval allowed in AS 1379; or

(c) the appearance and cohesiveness of a particular quantity is significantly different from previously supplied quantities of the same specification.

16.1.7.2 Hardened concrete

Hardened concrete shall be liable to rejection if—

(a) it does not satisfy the requirements of Clause 16.1.6;

(b) it is porous, segregated, or honeycombed, or contains surface defects; or

(c) it fails to comply with the other requirements of this Standard.

16.1.7.3 Action on hardened concrete liable to rejection

Where hardened concrete is liable to rejection in terms of Clause 16.1.7.2, the concrete may be accepted if it can be demonstrated, either by calculation or by testing, that the structural adequacy, intended use and design life of the affected members are not significantly impaired. Otherwise, the concrete shall be rejected.

For testing, reference shall be made to the appropriate clauses of Section 17.

16.1.8 Requirements for grout and grouting

16.1.8.1 Grout properties

Grout shall be proportioned to give the desired properties for its intended use. Grout to be used in grouting prestressing ducts shall have sufficient fluidity to enable it be pumped through the duct, have low sedimentation and shrinkage, and contain no more than 750 mg of chloride ions per litre or grout.

16.1.8.2 Mixing and agitation

Grout shall be mixed in a mixer capable of producing a uniform grout of the specified fluidity and free from lumps or undispersed cement.

After mixing, grout shall be held in an agitation tank and kept in motion, to prevent settlement or segregation occurring, before it is pumped into its final position.
16.2 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR REINFORCING STEEL

16.2.1 Materials

16.2.1.1 Reinforcement

Reinforcement shall be deformed Class N bars, or Class L or Class N welded wire mesh (plain or deformed), with a yield strength of up to 500 MPa, except that fitments may be manufactured from Class L wire or bar, or plain Class N bar.

All reinforcement shall comply with AS/NZS 4671.

16.2.1.2 Protective coatings

A protective coating may be applied to reinforcement provided that such coating does not reduce the properties of the reinforcement below those assumed in the design.

16.2.2 Fabrication

Fabrication of reinforcing steel shall be as follows:

(a) Reinforcement shall be fabricated to the shape and dimensions shown in the drawings and within the following tolerances—

   (i) On any overall dimension for bars and mesh except where used as a fitment:
       (A) For lengths up to 600 mm......................................................−25, +0 mm.
       (B) For lengths over 600 mm.....................................................−40, +0 mm.

   (ii) On all overall dimension of bars or mesh used as a fitment:
       (A) For deformed bars and mesh..................................................−15, +0 mm.
       (B) for plain round bars and wire.................................................−10, +0 mm.

   (iii) On the overall offset dimension of a cranked column bar..............−0, +10 mm.

   (iv) For the sawn or machined end of a straight bar intended for use as an end-bearing splice, the angular deviation from square, measured in relation to the end 300 mm, shall be within 2°.

(b) Bending of reinforcement shall comply with Clause 16.2.3.

(c) Welding, if required, shall comply with AS 1554.3. Tack welding not complying with AS 1554.3 shall not be used.

16.2.3 Bending

16.2.3.1 General

Reinforcement may be bent either—

(a) cold, by the application of force, around a pin of diameter complying with Clause 16.2.3.2, so as to avoid impact loading of the bar and mechanical damage to the bar surface; or

(b) hot, provided that—

   (i) the steel is heated uniformly through and beyond the portion to be bent;
   (ii) the temperature of the steel does not exceed 600°C;
   (iii) the bar is not cooled by quenching; and
   (iv) if during heating the temperature of the bar exceeds 450°C, the design yield strength of the steel after bending shall be taken as 250 MPa.

Reinforcement that has been bent and subsequently straightened or bend in the reverse direction shall not be bent again within 20 bar diameters of the previous bend.
Reinforcement partially embedded in concrete may be field bend provided that the bending complies with Items (a) or (b), and the bond of the embedded portion is not impaired thereby.

16.2.3.2 Internal diameter of bends or hooks

The nominal internal diameter of a reinforcement bend or hook shall be taken as the external diameter of the pin around which the reinforcement is bent. The diameter of the pin shall be not less than the value determined from the following as appropriate:

(a) For fitments of—
   (i) 500L bars ................................................................. \(3d_b\);
   (ii) R250N bars ............................................................. \(3d_b\); and
   (iii) D500N bars .............................................................. \(4d_b\).

(b) For reinforcement other than that specified in Items (c) and (d), of any grade ...... \(5d_b\).

(c) For reinforcement, in which the bend is intended to be subsequently straightened or rebent, of—
   (i) 16 mm diameter or less.................................................. \(4d_b\);  
   (ii) 20 mm diameter and 24 mm....................................... \(5d_b\); and
   (iii) 28 mm diameter or greater........................................ \(6d_b\).

Any such straightening or rebending shall be clearly specified or shown in the drawings.

(d) For reinforcement that is epoxy-coated or galvanized, either before or after bending, of—
   (i) 16 mm diameter or less.................................................. \(5d_b\); and
   (ii) 20 mm diameter or greater........................................ \(8d_b\).

16.2.4 Surface condition

At the time concrete is placed, the surface condition of reinforcement shall be such as not to impair its bond to the concrete or its performance in the member. The presence of millscale or surface rust shall not be cause for rejection of reinforcement under this Clause.

16.2.5 Fixing

All reinforcement, including secondary reinforcement provided for the purpose of maintaining main reinforcement and tendons in position, shall be supported and maintained in position within the tolerances given in Clauses 16.5.2.4 until the concrete has hardened. Bar chairs, spacers and ties used for this purpose shall be made of concrete, steel or plastics, as appropriate.

16.2.6 Lightning protection by reinforcement

Where lightning protection is to be provided by the reinforcement, the reinforcement shall comply with the relevant requirements of AS/NZS 1768.

16.3 MATERIAL AND CONSTRUCTION REQUIREMENTS FOR PRESTRESSING DUCTS, ANCHORAGES AND TENDONS

16.3.1 Materials for ducts, anchorages and tendons

16.3.1.1 Ducts

Sheaths and removable formers used to form ducts shall be capable of maintaining their required cross-section and profile during construction.
16.3.1.2 **Anchorages**

Prestressing anchorages shall comply with AS/NZS 1314.

16.3.1.3 **Tendons**

Prestressing tendons shall comply with AS 1310, AS 1311 and AS 1313, as applicable.

Tendons shall not be galvanized.

In the absence of assurance, such as a manufacturer’s certificate, the quality of tendons shall be established by testing in accordance with the applicable Standards.

Hard-drawn, high tensile steel wire, which has not been stress-relieved, shall not be used for wire winding unless its elongation, tested in accordance with AS 1310, is 2% or greater.

Plain wire shall not be used for pretensioning.

16.3.2 **Construction requirements for ducts**

16.3.2.1 **Surface condition**

When concrete is placed, the outside surface of sheaths and formers for ducts shall be such as not to impair bond of the concrete to the duct. Immediately before grouting, the inside surfaces of sheaths shall be such as not to impair bond of the grout to the duct.

Where an extractable core is used, a suitable technique shall be chosen to ensure its withdrawal, without damage to the formed duct.

16.3.2.2 **Sealing**

Prior to the placing of concrete, ducts shall be sealed at the ends and all joints, to exclude concrete or other matter.

16.3.2.3 **Fixing**

Ducts shall be supported and fixed at regular intervals so that the required tendon profile will be maintained in accordance with Clause 16.5.2.4.

16.3.3 **Construction requirements for anchorages**

16.3.3.1 **Fixing**

Anchorages shall be fixed strictly in accordance with the supplier’s recommendations and the following:

(a) The anchorage shall be square to the line of the tendon.

(b) The duct shall be securely attached to the anchorage so that it provides a grout-tight joint between the duct and the anchorage.

(c) Where the anchorage is fixed to the formwork, the joint between the two parts shall be grout-tight.

16.3.3.2 **Surface condition**

At the time concrete is placed, the surface condition of the anchorage shall be such as not to impair its bond to the concrete.

16.3.4 **Construction requirements for tendons**

16.3.4.1 **Fabrication**

Tendons shall be fabricated in accordance with the following:

(a) Cutting of tendons shall be carried out so that damage to tendons, ducts and anchorages is avoided.

(b) Tendons shall not be welded.
(c) Prestressing bars shall be within manufacturing tolerances and not be bent in the threaded portion.

Small adjustments on site shall be carried out cold provided that when the bar temperature is lower than 10°C, the bar temperature shall be raised above this value by means of steam or hot water.

16.3.4.2 Protection

Before stressing, tendons shall be protected from stray current arcing and splashes from the cutting operation of an oxy-acetylene torch or an arc-welding process.

The threaded ends of prestressing bars shall be provided with suitable protection, at all times.

If tendons are to have a coating or wrapping, such coating or wrapping shall be inert with respect to both the steel and the concrete.

After stressing and anchoring, all tendons and anchorages shall be protected from physical damage and corrosion.

16.3.4.3 Surface condition

The surface condition of tendons shall be such as not to impair bond to the concrete or grout, or performance in the member.

The presence of a slight film of rust shall not be cause for rejection of ducts under this Clause unless the steel is visibly pitted.

16.3.4.4 Fixing

All tendons shall be supported and maintained in position within the permissible tolerances given in Clause 16.5.2.4 until the concrete has hardened.

16.3.4.5 Tensioning

Tensioning of tendons shall be carried out in a safe manner and in accordance with the following:

(a) The stressing procedure shall ensure that the force in a tendon increases at a uniform time rate and that the force is transferred gradually to the concrete.

(b) The prestressing force applied to the tendon shall be measured at the jack by measuring the jack pressure. The prestressing force shall be measured to an accuracy of ±3%.

(c) The tendon extension shall be measured.

(d) A check shall be made for each tendon, on the correlation between the measured extension and the calculated extension derived from the prestressing force, using the load-elongation curves for the tendons and assumed friction values for the cable. Any disparity between the two figures greater than 10% of the calculated extension shall be investigated.

(e) No stressing shall be carried out when the temperature of the surrounding air is lower than 0°C.

16.3.4.6 Maximum jacking forces

The maximum force to be applied to a tendon during the stressing operation shall not be greater than—

(a) for pretensioned tendons.................................................................0.80f_{pb}A_p;

(b) for stress-relieved post-tensioned tendons.........................................0.85f_{pb}A_p; or

(c) for post-tensioned tendons not stress-relieved..................................0.75f_{pb}A_p.
16.3.4.7 Grouting

Ducts containing post-tensioned tendons shall be completely filled with grout, complying with Clause 16.1.8, as soon as practicable after stressing. Grouting shall not be carried out when the temperature of the surrounding air is lower than 5°C.

Precautions shall be taken to prevent corrosion of the tendons if the elapsed period prior to grouting is likely to exceed 4 weeks.

16.3.5 Construction requirements for unbonded tendons

Where unbonded tendons are used, the requirements of Clauses 16.3.4.1 to 16.3.4.6 shall apply, and the tendons shall be adequately protected against corrosion.

16.4 CONSTRUCTION REQUIREMENTS FOR JOINTS AND EMBEDDED ITEMS

16.4.1 Location of construction joints

The following apply:

(a) Construction joints shall be located to facilitate the placement of concrete in accordance with Clause 16.1.3.

(b) Unless otherwise specified, a construction joint shall be made between the soffits of slabs or beams and their supporting columns or walls.

(c) Where an interruption to the placing of concrete occurs such that the requirements of Clause 16.1.3(c) or 16.1.3(d) cannot be fulfilled, a construction joint complying with Clause 14.1.1 shall be made at an appropriate location.

16.4.2 Embedded and other items not shown in the drawings

Where an embedded item, driven fixing device, or hole is required but is not specifically shown in the drawings, or included in the specification, it shall be located so that the behaviour of the members is not impaired.

16.5 TOLERANCES FOR STRUCTURES AND MEMBERS

16.5.1 General

For the purposes of the strength requirements of this Standard, the position of any point on the surface of a concrete member shall comply with Clause 16.5.2. More stringent tolerances may be required for reasons of serviceability, fit of components, or aesthetics of the structure.

For formed surfaces the tolerances given in AS 3610 take precedence, unless those in Clause 16.5.2 are more stringent. For unformed plane surfaces, the flatness tolerances and the methods for measuring them shall be detailed in the project specification, and shall be not greater than the relevant values given in Clause 16.5.2.

16.5.2 Tolerances for position and size of structures and members

16.5.2.1 Absolute position

The deviation from the specified position shall not be greater than the following:

(a) In plan, 25 mm horizontally.

(b) In elevation—

   (i) for footings.................................................................25 mm vertically; and

   (ii) other than footings....................................................... 10 mm vertically.
16.5.2.2 Deviation from specified dimensions

The deviation from any specified height, plan, or cross-sectional dimension, shall not be greater than 1/200 times, the specified dimension or 5 mm, whichever is the greater.

16.5.2.3 Deviation from surface alignment

The deviation of any point on a surface of a member, from a straight line joining any two points on the surface, shall not be greater than 1/250 times the length of the line or 10 mm, whichever is the greater.

16.5.2.4 Tolerance on position of reinforcement and tendons

The deviation from the specified position of reinforcement and tendons shall be not greater than the following:

(a) For positions controlled by cover—
   (i) in beams, slabs, columns and walls.........................................................−5, +10 mm;
   (ii) in slabs-on-ground ................................................................................. −10, +20 mm; and
   (iii) in footings cast in the ground.................................................................−20, + 40 mm,
        where a positive value indicates the amount the cover may increase and a negative value indicates the amount the cover may decrease.

(b) For positions not controlled by cover, namely—
   (i) the location of tendons on a profile.......................................................... 5 mm;
   (ii) the position of the ends of reinforcement ........................................ 50 mm; and
   (iii) the spacing of bars in walls and slabs and of fitments in beams and columns........... 10% of the specified spacing or 15 mm, whichever is greater.

For fitments that are nominally planar, the plane of the fitment may be skewed by not more than three bar diameters of the fitment. The spacing of fitments shall be measured between the same location on adjacent fitments.

16.6 FORMWORK

16.6.1 General

The materials, design and construction of formwork shall comply with AS 3610.

16.6.2 Stripping of forms and removal of formwork supports

16.6.2.1 General

The stripping of forms and the removal of formwork supports shall comply with the following:

(a) Forms shall not be stripped or any formwork supports removed until the part of the member that will be left unsupported has attained sufficient strength to support, with safety and without detriment to its intended use, its own weight and any superimposed loads due to concurrent or subsequent construction works.

(b) Removal of formwork supports shall be carried out in a planned sequence so that the concrete structure will not be subject to any unnecessary deformation, impact, or eccentric loading during the process.

16.6.2.2 Removal of formwork from vertical surfaces

Formwork shall not be removed from vertical surfaces unless the concrete in the member has achieved sufficient strength to withstand potential damage to its surfaces.
SECTION 17 TESTING OF MEMBERS AND STRUCTURES

17.1 GENERAL
This Section applies to the testing of a new structure or of a prototype to demonstrate compliance with the strength and serviceability requirements of this Standard. In addition, a procedure is set out to demonstrate routine compliance for similar units manufactured following prototype testing. Methods for testing hardened concrete in place are also detailed.

All testing shall be undertaken by persons competent, and with appropriate expertise in, performing such tests.

The capacity of an existing structure to carry repeated live loads shall be determined in accordance with AS 5100.7. For testing of culverts, the capacity shall be determined in accordance with AS 1597.2.

17.2 TESTING OF MEMBERS

17.2.1 Purpose of testing
Structures designed by calculation in accordance with other parts of this Standard are not required to be tested. Tests may be accepted as an alternative to calculation (prototype testing), or may become necessary in special circumstances (proof testing), in order to satisfy the requirements of Clause 2.2 with respect to strength and Clauses 2.7 to 2.9 with respect to serviceability.

Where testing is necessary, elements of structures or whole structures shall be either—

(a) proof tested in accordance with Clause 17.3 to ascertain the structural characteristics of an existing member or structure; or

(b) prototype tested in accordance with Clause 17.4 to ascertain the structural characteristics of a particular class of member, which are nominally identical to the elements tested.

17.2.2 Test set-up
All measuring equipment shall be chosen and calibrated to suit the range of measurements anticipated, in order to obtain measurements of the required precision. Care shall be exercised to ensure that no artificial restraints are applied to the test specimen. All necessary precautions shall be taken such that in the event of collapse of any part of a structure being tested, the risk to life is minimized and the collapse will not endanger the safety of the structure being tested (for tests on members) or adjacent structures, or both.

17.2.3 Test load
The test load shall be applied gradually at a rate as uniform as practicable and without impact. The distribution and duration of forces applied in the test shall be representative of those forces to which the structure is deemed to be subject under the requirements of this Standard.

17.2.4 Test deflections
The maximum vertical deflections of each test specimen shall be measured with respect to an appropriate datum. Deflections shall, as a minimum requirement, be recorded at the following times:

(a) Immediately prior to the application of the test load.
(b) Incrementally during the application of the test load.
(c) Immediately the full test load has been applied.
(d) Immediately prior to removing the test load.
(e) Immediately after the removal of the test load.

17.3 PROOF TESTING

17.3.1 Test load
The test load shall be determined by the authority for the strength and serviceability limit states, as appropriate.

17.3.2 Test procedure
A proof test shall be conducted according to the following procedure:
(a) Before applying any load, record the original position of the members involved.
(b) Apply the test load as determined in accordance with Clause 17.2.3, for the relevant limit state.
(c) Maintain the test load for the necessary period as stated in Clause 17.3.3.
(d) Remove the test load.

17.3.3 Criteria for acceptance
Criteria for acceptance shall be as follows:
(a) *Acceptance for strength* The test structure or element shall be deemed to comply with the requirements for strength if it is able to sustain the strength limit state test load for at least 24 h without incurring excessive damage.
(b) *Acceptance for serviceability* The test structure or element shall be deemed to comply with the requirements for serviceability if it is able to sustain the serviceability test load for a minimum of 24 h without exceeding the appropriate serviceability limits.

17.3.4 Test reports
A report shall be prepared which shall contain, in addition to the test load and serviceability criteria records, a clear description of the test set-up, including the methods of supporting and loading the members as appropriate, the method of measuring deflections, crack-widths, and so on, and any other relevant data. The report shall also contain a statement as to whether or not the structure, substructure or members tested satisfied the relevant acceptance criteria in Clause 17.3.3 as appropriate.

17.4 PROTOTYPE TESTING

17.4.1 Construction of prototypes
The prototype shall be constructed from materials that comply with this Standard in accordance with the requirements of the manufacturing specification for the element or member.

17.4.2 Number of prototypes
The number of prototypes to be tested shall be selected so that statistically reliable estimates of the behaviour of the member at relevant limit state values can be determined from the results of the testing. No fewer than two prototypes shall be tested. More than one loading combination and more than one limit state condition may be applied to a prototype.
17.4.3 Test load

The test load shall be applied gradually until the total load on the prototype is equal to the design load for the strength limit state as determined from Section 3, multiplied by the relevant factor given in Table 17.4.3. This factor shall be selected with respect to the expected coefficient of variation in the parameters that affect the strength and the sample size selected for the testing program unless a reliability analysis shows that a different value is appropriate.

The total load for each prototype used to assess serviceability shall be the design load for the serviceability limit state as determined from Section 3 multiplied by a factor of 1.2.

### TABLE 17.4.3

<table>
<thead>
<tr>
<th>Number of similar units to be tested</th>
<th>Expected coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10%</td>
</tr>
<tr>
<td>2</td>
<td>1.3</td>
</tr>
<tr>
<td>3</td>
<td>1.3</td>
</tr>
<tr>
<td>5</td>
<td>1.2</td>
</tr>
<tr>
<td>10</td>
<td>1.1</td>
</tr>
</tbody>
</table>

NOTE: Intermediate values may be obtained by linear interpolation. The values given in Table 17.4.3 are based on a target safety index of 3.0 for a confidence level of 90%.

17.4.4 Test procedure

The method of applying the test load to the prototype shall reflect the most adverse conditions expected to occur during construction and the in-service condition. A prototype test shall be conducted according to the following procedure:

(a) Before applying any load, record the original position of the members in the test specimen.

(b) Apply the test load for the relevant limit state as determined from Clause 17.4.3.

(c) Maintain the test load for the necessary period as stated in Clause 17.4.5.

(d) Remove the test load.

(e) Inspect and record the prototype for damage, spalling, cracking and any other relevant observations.

17.4.5 Criteria for acceptance

The units represented by the prototypes shall be deemed to comply with this Standard for serviceability and strength where Item (a) is satisfied, and Item (b) or Item (c) is satisfied, as follows:

(a) Production units are similar in all respects to the prototypes tested and variability of units is not greater than the expected variability selected in Table 17.4.3.

(b) Acceptance for strength The test prototype shall be deemed to comply with the requirements for strength if it is able to sustain the strength limit state test load for at least five minutes without incurring excessive damage.

(c) Acceptance for serviceability The test prototype shall be deemed to comply with the requirement for serviceability if it is able to sustain the serviceability test load for a minimum period of one hour without exceeding the serviceability limits appropriate to the member.
Qualitative indicators for the parameters affecting strength shall be determined for expected variability during production. These indicators shall be routinely monitored and measured in manufactured units and used to assess the expected coefficient of variation. Alternatively, manufactured units shall be routinely tested to failure to determine the coefficient of variation.

### 17.4.6 Test reports

A report shall be prepared in accordance with Clause 17.3.4. The report shall also contain a statement as to whether or not the prototypes tested satisfied the relevant acceptance criteria specified in Clause 17.4.5 as appropriate.

### 17.5 QUALITY CONTROL

#### 17.5.1 Application

This Clause applies to the assessment of a group of units that are part of a production run of similar units. Clauses 17.5.2, 17.5.3 and 17.5.4 identify three methods to routinely assess production. One of these methods shall be nominated by the manufacturer as the means of demonstrating that the manufactured group is similar to the tested prototypes. A routine examination shall include the determination of the variability in a production run by relating key indicators in the sample to the previously performed prototype testing and the application of a test load to each sample, as appropriate.

#### 17.5.2 Statistical sampling

A sampling plan, in accordance with AS 1199 (all parts), shall be established for the routine inspection and testing of a produced batch. Sampling shall be undertaken in accordance with this plan and the selected specimens shall be routinely tested to ensure compliance with this Section is maintained.

For concrete specified by strength, the methods of production and assessment, taken together, shall provide a reliable operating characteristic curve so that—

(a) concrete with a proportion defective of 0.05 has a probability of acceptance of not less than 50%; and

(b) concrete with proportion defective of 0.30 has a probability of rejection of not less than 98%.

#### 17.5.3 Product certification

Independent assurance of the claim by a manufacturer or contractor of batch consistency shall be permitted, to ascertain whether a production run or application routinely complies with the requirements of this Section. The certification shall meet the criteria described in HB 18.28 in order that effective quality planning to control production is achieved.

#### 17.5.4 Quality system

Confidence in routine assessment of production shall be achieved where the manufacturer or contractor can demonstrate that an audited and registered quality management system complying with the requirements of the appropriate or stipulated Australian or international Standard for a quality system is in place.

Such a system shall include a quality or inspection and test plan to ensure product conformity.

### 17.6 TESTING FOR STRENGTH OF HARDENED CONCRETE IN PLACE

#### 17.6.1 Application

This Clause applies to the assessment of the strength of hardened concrete in place by non-destructive testing or coring.
17.6.2 **Non-destructive testing**

Non-destructive testing, including impact or rebound hammer and ultrasonic methods, or any combination of these, may be used to compare the strength of concrete under investigation with that of a comparable sample of known quality.

17.6.3 **Test on cores taken from the structure**

17.6.3.1 **Test requirements**

Taking and testing of cores shall comply with the following:

(a) Core locations shall be selected so as to minimize any consequent reductions of strength.

(b) The cores shall be representative of the whole of the concrete concerned and in no case shall fewer than three cores be tested.

(c) Cores shall be examined visually before and after testing, to assess the proportion and nature of any voids present. These factors shall be considered in the interpretation of the test results.

(d) Cores shall be taken and tested for compressive strength in accordance with AS 1012.14, and shall be tested dry unless the concrete concerned will be more than superficially wet in service. The density of cores shall be determined in accordance with AS 1012.12.

17.6.3.2 **Interpretation of results**

The strength of the concrete in part of a structure may be estimated—

(a) as 1.1 times the average strength of the cores with a diameter ideally 150 mm but not less than 100 mm and the length shall be not less than 1.5 times the diameter; or

(b) by comparison with cores from another part of the structure for which the strength of the concrete is known.
APPENDIX A

REFERENCED DOCUMENTS

AS

1012  Methods of testing concrete
1012.3 Method 3: Determination of properties related to the consistency of concrete
1012.3.4 Method 3.4: Compactibility index
1012.4 Method 4: Determination of air content of freshly mixed concrete
1012.4.1 Method 4.1: Measuring reduction in concrete volume with increased air pressure
1012.4.2 Method 4.2: Measuring reduction in air pressure in chamber above concrete
1012.9 Method 9: Determination of the compressive strength of concrete specimens
1012.10 Method 10: Determination of indirect tensile strength of concrete cylinders (‘Brasil’ or splitting test)
1012.11 Method 11: Determination of the modulus of rupture
1012.12 Method 12: Determination of mass per unit volume of hardened concrete
1012.12.1 Method 12.1: Rapid measuring method
1012.12.2 Method 12.2: Water displacement method
1012.13 Method 13: Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
1012.14 Method 14: Method for securing and testing cores from hardened concrete for compressive strength
1012.16 Method 16: Determination of creep of concrete cylinders in compression
1012.17 Method 17: Determination of the static chord modulus of elasticity and Poisson’s ratio of concrete specimens
1012.20 Method 20: Determination of chloride and sulfate in hardened concrete and concrete aggregates

1199  Sampling procedures for inspection by attributes
1199.0 Part 0: Introduction to the ISO 2859 attribute sampling system
1199.1 Part 1: Sampling schemes indexed by acceptance quality limit (AQL) for lot-by-lot inspection
1199.2 Part 2: Sampling plans indexed by limiting quality (LQ) for isolated lot inspection
1199.3 Part 3: Skip-lot sampling procedures
1199.4 Part 4: Procedures for assessment of declared quality levels

1310  Steel wire for tendons in prestressed concrete
1311  Steel tendons for prestressed concrete-7-wire stress-relieved steel strand for tendons in prestressed concrete
1313  Steel tendons for prestressed concrete—Cold-worked high-tensile alloy steel bars for prestressed concrete

1379  Specification and supply of concrete
1597  Precast reinforced concrete box culverts
1597.2 Part 2: Large culverts (from 1500 mm span and up to and including 4200 mm span and 4200 mm height)

2350  Methods of testing portland and blended cements (all parts)
AS
3582 Supplementary cementitious materials for use with portland and blended cement

3582.1 Part 1: Fly ash
3582.2 Part 2: Slag—Ground granulated iron blast-furnace

3600 Concrete structures

3610 Formwork for concrete

3799 Liquid membrane-forming curing compounds for concrete

3972 Portland and blended cements

5100 Bridge design

5100.1 Part 1: Scope and general principles
5100.2 Part 2: Design loads
5100.4 Part 4: Bearings and deck joints

5100.5 Supp 1 Supp 1: Bridge design—Concrete—Commentary
(Supplement to AS 5100.5)

5100.6 Part 6: Steel and composite construction
5100.7 Part 7: Rating of existing bridges

AS/NZS
1314 Prestressing anchorages

1554 Structural steel welding
1554.3 Part 3: Welding of reinforcing steel

1768(Int) Lightening protection

3000 Electrical installations (known as the Australian/New Zealand Wiring Rules)

3582 Supplementary cementitious materials for use with portland and blended cement

3582.3 Part 3: Amorphous silica

4671 Steel reinforcing materials

SAI
HB 18 Guidelines for third-party certification and accreditation
HB 18.28 Guide 28—General rules for a model third-party certification scheme for products

HB 64 Guide to concrete construction
HB 67 Concrete practice on building sites

BSI
BS 5400 Steel, concrete and composite bridges
BS 5400.4 Part 4: Code of practice for design of concrete bridges

Climate of Australia, 1982 Edition, Bureau of Meteorology Australia
APPENDIX B

DESIGN OF SEGMENTAL CONCRETE BRIDGES

(Normative)

B1 ANALYSIS

B1.1 Longitudinal analysis

Longitudinal analysis of segmental concrete bridges shall consider a specific construction method and construction schedule, as well as the time-related effects of concrete creep, shrinkage and prestress losses.

The effects of secondary moments due to prestress shall be included in stress calculations at serviceability limit states. In calculating flexural and shear resistance requirements at the strength ultimate limit states, the secondary moments or shears induced by prestress (with a load factor of 1.0) shall be added to the moments and shears due to factored dead and live loads.

B1.2 Transverse analysis

Consideration shall be given to the increase in web shear, transverse web flexure and other effects on the cross-section resulting from eccentric loading or asymmetry of the structural geometry.

B1.3 Deflection calculations

Prior to casting segments, deflections shall be calculated based on the anticipated casting and construction sequence and schedules. The calculated deflections shall be used as a guide for presetting the girders and for checking actual deflections during construction.

B2 LOADS

B2.1 Erection loads

Erection loads comprise all loadings arising from the anticipated system of temporary supporting works or special erection equipment, or both, to be used in accordance with the assumed construction sequence and schedule. The assumed erection loads and acceptable closure forces due to misalignment corrections shall be stated on the drawings. Allowance shall be made for all effects of changes to the statical structural scheme during construction and the application, changes or removal of the assumed temporary supports and special equipment, taking into account residual ‘built-in’ forces, moments, deformations, secondary post-tensioning effects, creep, shrinkage and any other strain-induced effects.

B2.2 Post-tensioning force

The structure shall be designed for both the initial and final post-tensioning forces. For determining the final post-tensioning forces, prestress losses shall be calculated for the construction schedule stated on the plans. The final post-tensioning forces used in serviceability limit state stress calculations shall be based on the most severe condition at each location along the structure.

B3 SHEAR AT JOINTS

Interfaces between elements such as webs and flanges, between dissimilar materials, between concretes cast at different times or at an existing or potential major crack shall be designed for shear transfer in accordance with Clause 8.4.
Shear keys in webs of precast segmental bridges shall extend for as much of the web height as is compatible with other detailing requirements. Alignment shear keys shall also be provided in top and bottom flanges.

For structures utilizing dry joints, the nominal shear resistance \( (V_{uj}) \) of the joint shall be calculated as follows:

\[
V_{uj} = 1.875A_k f'_{ct} \left( 1 + 0.205\sigma_{cp} \left( \frac{A_p}{A_g} \right) \right) + 0.45A_{sm} \sigma_{cp}
\]

where

- \( A_k \) = area of the base of all the keys in the failure plane
- \( f'_{ct} \) = characteristic uniaxial tensile strength of the concrete
- \( \sigma_{cp} \) = average intensity of effective prestress in concrete
- \( A_{sm} \) = area of contact between smooth surfaces on the failure plane

### B4 SEGMENTAL BRIDGE SUBSTRUCTURES

Consideration shall be given to erection loads, moments and shears imposed on piers and abutments by the construction method shown on the drawings. Auxiliary supports and bracing shall be shown on the drawings.

### B5 SPECIAL PROVISIONS

#### B5.1 Precast segmental construction

##### B5.1.1 Age of segments at erection

To limit construction deflections to values consistent with design calculations, precast segments shall be a minimum of 14 days old at the time of erection unless earlier erection is specifically approved.

##### B5.1.2 Temporary stress on epoxy joints

A minimum compressive stress of 0.28 MPa shall be provided for the closure stress on an epoxied joint until the epoxy has set.

##### B5.1.3 Dry joints

Dry joints shall not be used in conjunction with external post-tensioning tendons in areas with exposure classification B2 or C, or where freeze/thaw cycles occur.

Dry joints shall not be used for bridges with internal tendons.

##### B5.1.4 External tendons

External tendons shall be permanently protected against corrosion.

#### B5.2 Cast-in-place segmental construction

##### B5.2.1 General

Contact surfaces between cast-in-place segments shall be clean, free of laitance, and shall be intentionally roughened to expose coarse aggregate. The use of shear keys is optional.

##### B5.2.2 Diaphragms

Diaphragms shall be provided at abutments, piers, hinge joints, and at bottom flange angle points in structures with straight haunches. Diaphragms shall be substantially solid at piers and abutments except for access openings and utility holes.
B5.3 Incremental launching—Bridge design

Piers and superstructure diaphragms at piers shall be designed in such a way that during all launching stages, and after launching for the installation of the permanent bearings, the superstructure can be lifted with hydraulic jacks. Pier designs shall consider frictional forces during launching. Abutments shall be designed to resist the launching force where this is applied at the abutment.

High local stresses occur at the underside of the webs above the launching bearings, along the full length of the bridge. The design shall take into account the bearing pressures at the bottom edges of superstructures, and shall consider any eccentric location of the support reaction and any ungrouted ducts.

Design shall make allowance for the additional forces during launching due to the specified tolerances in the bearing levels, temporary bearing pad thickness, deck soffit profile and the like. This applies to the casting bay and temporary piers.

B6 SPECIFICATIONS

The method of construction shall be taken into consideration when designing the permanent works. Assumptions used in the design of the permanent works pertaining to the method of construction shall be included in the drawings or specification. Tolerances shall be provided regarding the construction equipment weights and variations in material properties to be used.

Allowances shall be shown for variations in construction loads and construction stages. The resultant camber information shall be given such that development of casting curves can be achieved.
APPENDIX C

BEAM STABILITY DURING ERECTION

(Normative)

A beam being lifted either by vertical or inclined slings may collapse or be damaged by excessive cracking due to tilting of the beam about a longitudinal axis through the lifting points. This initial tilting may be initiated by imperfections in the beam geometry and in the eccentric location of the lifting points.

The stability of a prestressed beam lifted at or near the ends by vertical slings, which allow rotation about the longitudinal axis through the lifting points (Figure C1), shall be determined as follows:

(a) Calculate the factor of safety against lateral buckling \( \psi_r \) as follows:

\[
\psi_r = \frac{e_o}{0.64\Delta_h} \quad \ldots \text{C1}
\]

where

\[
e_o = \text{vertical eccentricity between the centre of gravity of a beam and the longitudinal axis through the lifting points} = y_t - 0.67\Delta_v \quad \ldots \text{C2}
\]

\[
\Delta_h = \text{lateral deviation of a slender beam at mid-span from the specified datum line immediately after transfer}
\]

\[
y_t = \text{depth from the centroidal axis to the extreme fibre at the top of the section}
\]

\[
\Delta_v = \text{lateral deflection caused by the self weight of the beam due to bending about the } y-y \text{ axis}
\]

(b) Calculate the design lateral bending moment \( M_h^* \) as follows:

\[
M_h^* = \frac{M_g \beta_v}{\left( \frac{\psi_r - 1}{\psi_r} \right)} \quad \ldots \text{C3}
\]

where

\[
\beta_v = \frac{e_x + 0.67\Delta_h}{y_t} \quad \ldots \text{C4}
\]

\[
e_x = \text{eccentricity of the lifting point to the minor centroidal axis of a beam}
\]

The factor of safety \( \psi_r \) shall not be less than 2.0. In addition, stresses due to the combined effects of the lateral bending moment \( M_h^* \), the bending moment due to self-weight \( M_g \) and the prestress shall be assessed and, if cracking is possible, the lifting arrangements shall be changed or the beam shall be provided with adequate lateral support.
NOTE: The limiting stress in these calculations is the characteristic flexural tensile strength (see Clause 6.1.1). Consideration should also be given to increase the bending moment ($M_0$) to allow for dynamic and impact effects during handling of the beam.
APPENDIX  D
SUSPENSION REINFORCEMENT DESIGN PROCEDURES
(Normative)

D1  GENERAL

Some form of suspension reinforcement is required when a load is applied such as to cause direct vertical tension in the web of a member.

Three cases where this is met in bridge design, together with design procedures are described in Paragraphs D2, D3 and D4.

NOTES:

1. Refer also to Appendix I for modelling using Strut-and-Tie method.

D2  SUSPENDED LOADS ON MEMBERS

When loads act away from the centroidal axis of a member (suspended loads), as shown in Figure D1(a), suspension reinforcement shall be provided in the zones of the loads to transfer them to the upper (compression) face.

Suspension reinforcement in the form of stirrups or ties additional to other shear reinforcement to carry the shear forces generated by the suspended load shall be provided, except that shear reinforcement for this load, which falls inside the zone described in Paragraph D1(b), may be included as part of the suspension reinforcement. These ties shall extend over the full depth of the member and shall also comply with the following:

(a) \( F^* \leq \phi A_{sw} f_{sy} \) . . . D2

where
\( F^* \) = design suspended load at the strength ultimate limit state (action effect)
\( A_{sw} \) = area of the suspension reinforcement

(b) The ties shall be placed in the appropriate zones within a distance \( D/2 \) on either side of the suspended loads, as shown in Figure D1(b).

(c) Compression shall be determined in accordance with Clause 12.1.2 and Appendix I. The projecting part of the member, that is, the flange on which the suspended loads directly act, shall be designed in accordance with Paragraph D4, with steel distributed in the appropriate zones around the load.
The procedures described in this Paragraph apply to bridge girders supported indirectly, that is, outside the plane of the girder webs as shown in Figure D2(a). Suspension reinforcement shall be provided at the intersection of the web and diaphragm to transfer the load from the bottom of the web to the top of the diaphragm as follows:

\[ F_t^* = R_v^* \]  

where

- \( F_t^* \) = design tension force in a tensile tie of an analogous truss
- \( R_v^* \) = design vertical reaction at ultimate limit state

**FIGURE D1 MEMBER SUBJECTED TO SUSPENDED LOAD**

**D3 MEMBERS WITH INDIRECT SUPPORT**

The procedures described in this Paragraph apply to bridge girders supported indirectly, that is, outside the plane of the girder webs as shown in Figure D2(a). Suspension reinforcement shall be provided at the intersection of the web and diaphragm to transfer the load from the bottom of the web to the top of the diaphragm as follows:

\[ F_t^* = R_v^* \]  

where

- \( F_t^* \) = design tension force in a tensile tie of an analogous truss
- \( R_v^* \) = design vertical reaction at ultimate limit state
The girder web shall be stressed in a similar way to an inverted case of a load suspended from beneath the web at the diaphragm. Thus it becomes a similar case to Paragraph D2.

Suspension reinforcement shall be provided in addition to other shear reinforcement requirement, except that shear reinforcement placed inside the zone described in Paragraph D3(b) may be included as part of the suspension reinforcement. The suspension reinforcement shall include vertical or inclined prestressed tendons or vertical or inclined non-prestressed stirrups, or all or some of the above. These ties shall extend over the full depth of the girder, and shall also comply with the following:

(a) \[ F_t^* \leq \phi A_{sv} f_{sy} \text{ (or } f_{pb} \text{ as appropriate)} \]  

\[ \text{where } F_t^* \text{ is the design vertical reaction per web of the main girder.} \]

(b) The suspension reinforcement shall be distributed in the hatched area of the intersection region as shown in Figure D2(d). 70% of this reinforcement shall be located in the longitudinal girder.

This reinforcement shall extend over the full depth of the member and shall be adequately anchored at the top and bottom.

(c) Compression shall be determined in accordance with Clause 12.1.2 and Appendix I.

NOTES:

1. For the purposes of this Paragraph, at the strength ultimate limit state, \( R_v^* \) denotes the design vertical reaction per web of the main girder. Figure D2(b) shows that this implies a support reaction of \( 2R \) and from the analogous truss shown in Figure D2(c), the design tension tie transfer load.

2. For background information, see Ref. 1 in Appendix J.
FIGURE D2 MEMBERS WITH INDIRECT SUPPORT

D4 STEPPED JOINTS

The procedures described in this Paragraph apply to the type of member shown in Figure D3, although the joint may not necessarily contain prestressing tendons. Suspension
reinforcement in the form of inclined stirrups or ties shall be provided to transfer the reaction force \( R_v^* \) to the upper (compression) face. Also horizontal reinforcement shall be provided at the depth of the cantilever. (See Section 12.)

Suspension reinforcement in the form of inclined stirrups or ties shall be provided in addition to bursting or spalling reinforcement, which may be required for prestressing anchorages or bearings. These ties shall extend through the full depth of the member and shall also comply with the following:

(a) \[ R_v^* \leq \phi A_{sw} f_{sy} \sin \alpha_h \quad (\alpha_h < 60^\circ) \]  \ldots D4

where

- \( R_v^* \) = design vertical reaction at the strength ultimate limit state
- \( A_{sw} \) = area of reinforcement provided for suspension ties.
- The area of any prestress tendons shall be ignored
- \( \alpha_h \) = angle of the inclination of inclined bars in a stepped joint

NOTES:

1. For the purpose of these procedures, the vertical component of prestress is treated as a load, and it is, therefore, deducted from other reactions in determining \( R_v \). In this computation, only tendons crossing the potential crack may be included in calculating the vertical component of prestress, e.g., in Figure D3, the top tendon may be included, but not the bottom tendon.

2. In the case of pretensioned members, the vertical component of prestress should be ignored over the transmission length of the tendons.

(b) Horizontal reinforcement at the depth of the cantilever shall be provided such that when moments are taken about the centroid of the nib (point C in Figure D3) for the forces \( R_v^* \) and \( R_h^* \), the force in the horizontal reinforcement is less than or equal to—

\[ \phi A_k f_{sy} \]

where

- \( R_h^* \) = design horizontal reaction at the strength ultimate limit state
- \( \phi \) = strength reduction factor for shear
- \( A_k \) = area of the horizontal reinforcement at the cantilever depth

(c) The suspension ties shall be placed within the region extending a distance equal to the depth of the cantilever from the re-entrant corner as shown in Figure D3. The first stirrup shall be placed as close to the end face of the re-entrant corner as possible.

(d) The horizontal reinforcement shall extend at least \( D \) beyond the re-entrant corner.

(e) Figure D3, which shows suitable detailing but in general horizontal steel, shall be anchored in a compression region, suspension ties shall be closed loops and, where practicable, a fillet shall be provided at the re-entrant corner of the stepped joint to minimize stress concentration.

---

Interim

NOTE: For background information, see Ref. 2 in Appendix J.
FIGURE D3 DESIGN DETAILS FOR STEPPED JOINTS IN PRESTRESSED MEMBERS
APPENDIX E
COMPOSITE CONCRETE MEMBERS DESIGN PROCEDURES
(Normative)

E1 APPLICATION
Composite flexural members shall consist of precast prestressed concrete beams connected to cast-in-place reinforced concrete parts (see Figure E1) so that the two components function as a monolithic unit.

NOTE: For standard precast prestressed concrete beam sections, see Appendix H.

In this Appendix, a continuous composite member shall consist of a succession of simply supported prestressed concrete beams made continuous by the provision of non-prestressed reinforcement in the cast-in-place concrete over the intermediate supports.

Monolithic action of composite beams up to the strength ultimate limit state in bending of the member (see Clause 8.1) shall be ensured by the following:

(a) Transferring longitudinal shear at the contact surface without excessive slip.

(b) Preventing separation of the elements normal to the contact surface.

The transfer of shear shall be achieved by some combination of bond, roughness, steel ties and shear keys, and the separation shall be prevented by steel ties.

The requirements for shear connection shall be as set out in Clause 8.4.
E2 CONSIDERATIONS

E2.1 General

The following design requirements shall be met:

(a) The construction sequence influences the design of composite members and shall be indicated on the drawings.

(b) The individual elements of the composite member shall be investigated for any critical loads during construction, for example, handling and erection as well as for the loads applied after their interconnection.

(c) The effects of residual creep in the precast beam and differential shrinkage between the beam and the cast-in-place concrete shall be considered, and the member shall be designed for the following two extreme cases:

(i) Final residual creep acting with final differential shrinkage.

(ii) Zero residual creep acting with zero differential shrinkage (representing conditions at a time shortly after completion of construction).

Methods of calculating the effects of residual creep and differential shrinkage shall be as specified in Paragraph E3.2.

Residual creep and differential shrinkage in a composite member shall be regarded as always acting together.
E2.2 Analytical

The following assumptions shall be made for analysis:

(a) The effective width of the cast-in-place concrete slab shall be used in the design of a concrete member and shall be determined in accordance with Clause 8.8.

(b) The effective cross-sectional area of the cast-in-place concrete shall be transformed to an equivalent area of beam concrete by multiplying by the modular ratio \((\alpha_c)\) of cast-in-place concrete and the precast beam concrete in the composite member.

E3 Design for Applied Loads

E3.1 General

All components and composite members shall be designed in accordance with Section 8 for all loads to which they are subjected. Particular attention shall be given to the validity of any assumptions about concrete stress strain relationships being used while high compression stresses are occurring at the serviceability limit state.

E3.2 Effects due to residual creep and differential shrinkage

E3.2.1 General

Residual creep is that portion of the creep that occurs in the precast element after establishing composite action.

The following procedures may be used to determine the effects of residual creep in a composite member subject to dead load and prestress only, and the effects of differential shrinkage. The procedures shall be based on the rate of creep method, which has some theoretical flaws, and which therefore gives only approximate answers; however, for bridges with spans up to 30 m, problems have not been experienced using this method. Consideration shall be given to other methods, such as superposition or rate of flow method if a more refined solution is required.

NOTE: For background information, see Refs 3 and 4 in Appendix J.

E3.2.2 Effect of creep due to sustained loads

In a composite member, creep occurring in the precast beam results in a redistribution of stresses between the beam and the cast-in-place concrete. The magnitude of these stresses depends on the age of the precast beam when composite behaviour is established. If a large proportion of the creep in the beam has taken place by the time the slab is cast, the effect of subsequent creep will be small.

The calculation of stresses between the precast beam and the cast-in-place concrete shall be as follows:

(a) Simply supported members Calculation of stresses due to sustained loads shall be based on the assumption that the stresses in any cross-section lie between the following extreme distributions:

(i) The stress distribution due to dead load (beam and cast-in-place concrete) and the prestress after all losses, acting on the precast beam.

(ii) The stress distribution due to dead load (beam and cast-in-place concrete) and prestress after all losses acting on the composite section. The member shall be considered monolithic, and the eccentricity of the prestressing force shall be measured to the centroid of the composite section.
The stresses in the composite section caused only by residual creep in the precast beam shall be calculated as follows:

\[ \left[ 1 - e^{-\varphi_{cc,j}} \right] \times \text{[stresses in Item (ii) - stresses in Item (i)]} \]  

E3.2.2(1)

which \( \varphi_{cc,j} \) is the residual creep coefficient, which depends on the amount of creep strain that will occur after the precast beam and the cast-in-place concrete are made composite (see Table E1).

The final stresses in the composite section due to dead load (precast beam and cast-in-place concrete), prestress and creep shall be the sum of—

\[ \text{[stresses from Item (i)] + [stresses due to residual creep]} \]  

\[ \ldots \text{E3.2.2(2)} \]

(b) Continuous members Stresses in a continuous composite member due to dead load, prestress after all losses and creep shall be calculated by considering the continuous member separated into simply supported spans, and then restoring continuity by applying restraint moments at the supports. The final stresses at any section shall be the sum of the following:

\[ \text{[those occurring in each simply supported span calculated in accordance with Item (a) above]} + \text{[those caused by } \left[ 1 - e^{-\varphi_{cc,j}} \right] \times \text{the continuity restraint moments resulting from the application of both the dead load and prestress to the continuous composite section described in Item (a)(iii)}] \]

The restraint moments may be calculated by any method using elastic analysis. The restraint moment calculation shall be based on the assumption that continuity and composite action are established in all spans simultaneously at time \( t_j \). A minimum and maximum estimated value of \( t_j \) shall be used in the calculation of creep and shrinkage effects.

### TABLE E1

<table>
<thead>
<tr>
<th>( \varphi_{cc,j} )</th>
<th>0</th>
<th>0.5</th>
<th>1.0</th>
<th>2.0</th>
<th>3.0</th>
<th>4.0</th>
<th>5.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 1 - e^{-\varphi_{cc,j}} )</td>
<td>0</td>
<td>0.393</td>
<td>0.632</td>
<td>0.865</td>
<td>0.950</td>
<td>0.982</td>
<td>0.993</td>
</tr>
<tr>
<td>( \frac{1 - e^{-\varphi_{cc,j}}}{\varphi_{cc,j}} )</td>
<td>1.0</td>
<td>0.787</td>
<td>0.632</td>
<td>0.432</td>
<td>0.317</td>
<td>0.245</td>
<td>0.199</td>
</tr>
</tbody>
</table>

E3.2.3 Effect of differential shrinkage

Differential shrinkage between the precast beam and the cast-in-place concrete produces stresses and deformations in both simply supported and continuous members. Modification of these stresses due to the effect of residual creep in the precast beam shall be evaluated as follows:

(a) Simply supported members Stresses and deformations in the composite member, due to differential shrinkage, shall be evaluated assuming a uniform differential shrinkage force along the member calculated as follows:

\[ \text{Differential shrinkage force } = E_c A_{cs} \varphi_{es,j} \left( \frac{1 - e^{-\varphi_{es,j}}}{\varphi_{cc,j}} \right) \]  

\[ \ldots \text{E3.2.3} \]

where

\[ A_{cs} = \text{area of cast-in-place concrete} \]
Interim

\[ \varphi_{cs,j} = \text{differential shrinkage} \]

The term \[ \left( \frac{1-e^{-\varphi_{cs,j}}}{\varphi_{cc,j}} \right) \] accounts for the influence of residual creep in the beam, and some values of this coefficient are given in Table E1.

The stresses in the composite beam shall be obtained from the sum of the following:

\[ \text{[a direct tensile force equal to the differential shrinkage force acting at the centroid of the cast-in-place concrete only]} + \text{[a corresponding compressive force equal to the differential shrinkage force at the centroid of the cast-in-place concrete, and acting on the composite section]}. \]

(b) Continuous members Stresses and deformation in a continuous member due to differential shrinkage shall be calculated by considering the continuous member as separated into simply supported spans, and then restoring continuity by applying restraint moments at the supports. The final stresses at the cross-section shall be the sum of the following:

\[ \text{[those occurring in the section in the simply supported span, calculated in accordance with Item (a)]} + \text{[those caused by the continuity moments]}. \]

In the cast-in-place concrete directly over the piers, stresses are produced only by the continuity restraint moments.

The restraint moment calculation shall be based on the same time of establishment of continuity assumptions as in Paragraph E3.2.2(a).

E3.3 Design of continuous composite members

E3.3.1 General

This Paragraph applies to special aspects of the design of composite structures erected as single spans of precast prestressed concrete beams of uniform depth and made continuous afterwards for live load and superimposed dead load.

The analysis of the continuous member shall be based on the assumption of uniform moment of inertia using the uncracked cross-section including the actual width of the member.

The time-dependent effects of creep and shrinkage shall be calculated in accordance with Paragraph E3.2.

E3.3.2 Positive moment connection at supports

In addition to those due to live load, support settlement, thermal effects and the like, positive moments can develop due to the combined effects of differential creep and shrinkage. Where positive moments occur at supports, fully anchored non-prestressed longitudinal reinforcement shall be cast into the ends of the precast beams to enable subsequent connection of the bottom flanges of adjoining beams at supports when erected. This reinforcement shall be designed for the serviceability limit state in accordance with Clause 8.6.1.

The connection may be a welded splice or overlapping reinforcement. If overlapping caged bars or hooked bars are used, the distance between the end face of the beam and the inside edge of the leg of the bar projecting from the beam shall be not less than 12 times the bar diameter.
E3.3.3 Negative moment zones

The value of $f'_c$ for the beam concrete and the width of the bottom flange of the beam shall be used in the strength calculation for the cross-section directly over internal supports.

The negative moment reinforcement shall be distributed evenly within the effective width and extended at the same rate beyond that area.
APPENDIX F
BOX GIRDERS
(Normative)

F1 SHEAR LAG
Shear lag may be significant in any section of flange extending beyond the face of the web if for—
(a) positive bending moments, $L_1/b_{fc}$ is less than 5; and
(b) negative bending moments, $L_2/b_{fc}$ is less than 10.

The width ($b_{fc}$) of the portion of the flange and the lengths $L_1$ and $L_2$ shall be as shown in Figure F1(a) and (b) respectively. In Figure F1, the bending moments shall be the maximum design moments for the support in the case of $L_2$ and for midspan in the case of $L_1$.

In the absence of a more accurate calculation, the effective width of flange shall be determined as for integral beam and slab construction specified in Clause 8.8.2.

NOTE: Where shear lag effects are expected to be large, reference should be made to specialist literature.

F2 ADDITIONAL DETAILS FOR BOX GIRDERS
The following additional details for box girders shall be considered:
(a) **Fillet** Fillets or haunches shall be provided at the intersections of all surfaces within the cells of a box girder. If torsion is important in the design, the dimension of fillets should be not less than the least thickness of the members meeting at the joint.

(b) **Dimensions** The minimum top flange thickness shall be 1/30 times the clear distance between fillets but not less than 150 mm. The minimum bottom flange thickness shall be 1/30 times the clear distance between fillets or webs but not less than 140 mm.

(c) **Top flange reinforcement** Transverse reinforcement at the bottom of the top flange shall be anchored beyond the faces of webs.

(d) **Bottom flange reinforcement** Minimum reinforcement of 0.15% of the flange cross-sectional area shall be placed at each face of the bottom flange, in both the transverse and longitudinal directions. The spacings of bars shall not exceed either 300 mm, or twice the flange thickness.

(e) **Additional reinforcement** Consideration shall be given to providing additional reinforcement in webs, flanges or diaphragms to control cracking which may occur as a result of restraint to shrinkage, arising from the construction sequence and thermal movements during construction.

Consideration shall also be given to providing additional reinforcement in deck areas at the ends of box girders with wide cantilevers where shear lag effects may be significant.
FIGURE F1 TERMINOLOGY FOR SHEAR LAG EFFECT IN BOX GIRDERS
APPENDIX  G

END ZONES FOR PRESTRESSING ANCHORAGES

(Informative)

G1  INTRODUCTION

This Appendix sets out to explain and clarify the design of end zones for prestressing anchorages.

G2  GENERAL

The analysis of stresses in an end zone is a complex three-dimensional problem. For design, approximate methods are used, which involve carrying out two-dimensional analyses in each of the two longitudinal directions in turn.

The approach is that reinforcement should be provided to carry the entire transverse tensile force in each direction; no rules are given for calculating stresses.

G3  DEFINITIONS

For the purposes of this Appendix, the definitions below apply.

G3.1  Symmetrical prism

The concept of the symmetrical prism shown in Figure G1(A) may be used for estimating the magnitudes of the transverse tensile forces, and the lengths over which transverse tensile stresses occur, at sections immediately behind anchorages. Circular anchorages or anchor plates may be designed as square anchorages with the same area.

G3.2  Transverse moment

Figure G1(B) shows the anchorage zone of a post-tensioned beam with a single anchorage. At the end of the anchorage zone (section B-B) the stresses caused by prestress may be determined from simple beam theory. On any longitudinal section such as section C-D, there is a transverse bending moment ($M_t$) whose magnitude may be calculated from the equilibrium requirements for the free body ABCD.

This moment is a resultant of transverse tensile and compressive stresses acting across section C-D. Figure G1(B) also shows how the magnitude of $M_t$ varies throughout the depth of the beam.

The sense of the moment $M_t$ shown in Figure G1(B) indicates that the resultant of the transverse compressive stresses on section C-D acts closer to the loaded face A-A than does the tensile stress-resultant. A moment with this stress is defined as positive and is referred to as producing bursting forces.

The sense of the moment may be reversed at sections between anchorages in end zones with two or more widely spaced anchorages (see Figure G1(C)), or at sections remote from the anchorage in end zones with a single eccentric anchorage (see Figure G1(D)).

Transverse moments with this sense, implying that the resultant of the transverse tensile stresses is closer to the loaded face than is the compressive stress resultant, are defined as negative and are referred to as producing spalling forces.
FIGURE G1(A)  SYMMETRICAL PRISM

FIGURE G1(B)  TRANSVERSE MOMENT

FIGURE G1(C)  SYMMETRICAL ANCHORAGES

FIGURE G1(D)  ECCENTRIC ANCHORAGES
G4 END BLOCKS

In order to accommodate anchorages and to reduce congestion of anchorage zone reinforcement in prestressed concrete members, end blocks may be used. For end block dimensions of standard precast prestressed concrete beam section, see Appendix H.

Where end blocks are used, they should be at least as wide as the narrower flange of the beam and have a minimum length to depth ratio of 0.75.

G5 DESIGN OF REINFORCEMENT

G5.1 Bursting reinforcement

Bursting reinforcement should be determined as follows:

(a) **Amount of reinforcement** The design bursting tensile force in each of the two principal directions is to be entirely resisted by reinforcement.

Only rigid parts of the anchorage bearing plate should be considered as shown in Figure G2(A).

Where multiple anchorages with individual anchor plates occur, the end block is considered as subdivided into a series of symmetrically loaded prisms and each prism designed individually.

The reinforcement of adjacent prisms is to be interconnected and the complete multi-anchorage system tied together.

When calculating the bursting tensile forces, account is to be taken of the temporary conditions occurring during stressing and the prism size proportioned in accordance with the proposed stressing sequence, and suitable reinforcement designed.

Zones of secondary bursting can occur if the anchorages are closely arranged in groups. The bursting forces can be calculated in accordance with Clause 12.2.4 using the same expression for $T$, in which the appropriate group force and $k_r$ ratio values are inserted in the equation.

(b) **Type of reinforcement** The bursting reinforcement may consist of any, or a combination of, the following types of reinforcement placed perpendicular to the axis of the tendon and adequately anchored:

(i) Mats consisting of sets of parallel bars in two perpendicular directions either welded together or bent in a continuous hair-pin form.

(ii) Welded mesh.

(iii) Sets of closed ties.

(iv) Spiral reinforcement, which may be used for bursting forces for an individual anchorage.

Whichever type of reinforcement is used, the aim should be to cross the planes of potential cracking with as many bars as reasonably possible, particularly near the axis of the tendon.

(c) **Arrangement of reinforcement** Reinforcement required to resist bursting forces should be placed in a region extending from $0.1d_{et}$ to $d_{et}$ measured from the inside face of the anchor plate as shown in Figure G2(B), where $d_{et}$ is the effective depth of a hypothetical symmetrically loaded prism of a post-tensioned end block. The maximum bursting tensile stresses occur at a distance approximately $0.25d_{et}$ from the inside face of the anchor plate, and the bursting reinforcement should be more closely spaced in this region.
Reinforcement in anchorage zones can often be difficult to accommodate and it is important sometimes to modify the position requirement specified in Item (c) to place and adequately compact concrete in the anchorage zone.

G5.2 Spalling forces

The end face of the anchorage zone should be continuously reinforced to prevent edge spalling. Reinforcement should be placed as close to the end face as possible, and in any case between the end face and a plane at 0.25 times the depth of the member from the end face.

**FIGURE G2(A) WIDTH OF ANCHORAGE USED IN CALCULATING BURSTING FORCES**

**FIGURE G2(B) POSITIONING OF REINFORCEMENT IN POST-TENSIONED END BLOCKS**
APPENDIX  H

STANDARD PRECAST PRESTRESSED CONCRETE GIRDER

(Informative)

H1 GENERAL

The standard sections for precast, prestressed concrete bridge girders shown in Figures H1(A) for I-girders and in Figure H1(B) for Super T-girders, have been adopted. For the Super T-girder sections, the size of the internal void has not been detailed.

In addition, Figure H1(C) shows the earlier dimensions of Super T-girders used up until mid 2001 when the width of the bottom flange was increased to enable the addition of a deeper section, to a common mould shape.

Cover in excess of 25 mm required for durability may require increased width of the webs of standard sections.
### Table 1: Girders Properties

<table>
<thead>
<tr>
<th>Girder Type</th>
<th>$A_g$ (mm²)</th>
<th>$Z_{y, sup}$ (mm³)</th>
<th>$Z_{y, sub}$ (mm³)</th>
<th>$I$ (mm⁴)</th>
<th>$d_{y, sub}$ (mm)</th>
<th>Hypothetical thickness, $t_h$ (girders only)</th>
<th>* (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$126 \times 10^3$</td>
<td>$17.9 \times 10^6$</td>
<td>$22.0 \times 10^6$</td>
<td>$7400 \times 10^6$</td>
<td>337</td>
<td>120</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>$218 \times 10^3$</td>
<td>$41.1 \times 10^6$</td>
<td>$48.1 \times 10^6$</td>
<td>$19950 \times 10^6$</td>
<td>415</td>
<td>155</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>$317 \times 10^3$</td>
<td>$82.9 \times 10^6$</td>
<td>$91.1 \times 10^6$</td>
<td>$49900 \times 10^6$</td>
<td>548</td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>$443 \times 10^3$</td>
<td>$135.9 \times 10^6$</td>
<td>$168.6 \times 10^6$</td>
<td>$105330 \times 10^6$</td>
<td>625</td>
<td>205</td>
<td></td>
</tr>
</tbody>
</table>

*Refer to Articles 5.6.1.7(b) and 5.6.1.8(b)

### Figure H1(A): Standard Precast Prestressed Concrete I-Girder Sections

**Dimensions in millimetres**

**FIGURE H1(A) STANDARD PRECAST PRESTRESSED CONCRETE I-GIRDER SECTIONS**
DIMENSIONS IN MILLIMETRES

FIGURE H1(B) STANDARD PRECAST PRESTRESSED CONCRETE SUPER T-GIRDER SECTIONS
FIGURE H1(C)  PRECAST PRESTRESSED CONCRETE
SUPER T-GIRDER SECTIONS (PRE-2001)

DIMENSIONS IN MILLIMETRES

LEGEND:
* = Denotes dimension has to be increased if flange thickness > 75
\( \dagger \) = Denotes dimensions varies
H2  END BLOCK DIMENSIONS

The recommended dimensions for end blocks for post-tensioned I-girders are given in Table H1.

**TABLE H1**

END BLOCK DIMENSIONS FOR POST-TENSIONED I-GIRDERS

<table>
<thead>
<tr>
<th>Girder type</th>
<th>End block length (mm)</th>
<th>End block width (mm)</th>
<th>Taper length (see Note) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>750</td>
<td>200</td>
<td>70</td>
</tr>
<tr>
<td>2</td>
<td>900</td>
<td>350</td>
<td>170</td>
</tr>
<tr>
<td>3</td>
<td>1,150</td>
<td>450</td>
<td>260</td>
</tr>
<tr>
<td>4</td>
<td>1,400</td>
<td>500</td>
<td>300</td>
</tr>
</tbody>
</table>

**NOTE:** The taper length is the length of the tapered section between the end block and the web of the beam.

H3  FLEXURAL PROPERTIES

Flexural moduli about the major axis of bending for standard precast prestressed concrete I-girder sections are given in Table H2(A).

Flexural moduli about the major axis of bending are also given for a typical range of Super T-girder sections that conform to the standard sections shown in Figure H1(B), with the following dimensions:

(a) Width of webs: 100 mm (for girder types 1 to 4).
   120 mm (for girder type 5).

(b) Thickness of top flange: 75 mm.

(c) Width of top flange: 2100 mm.

(d) Thickness of bottom flange at centre-line: Dimension $t_b$ (see Tables H2(B)(1) and H2(B)(2)).

Flexural moduli for the open top flange case are given Table H2(B)(1) and for the closed top flange case are given in Table H2(B)(2).
### TABLE H2(A)
**FLEXURAL MODULI—PRECAST CONCRETE I-GIRDERS**

<table>
<thead>
<tr>
<th>Girder type</th>
<th>$A_g$</th>
<th>$Z_t$</th>
<th>$Z_b$</th>
<th>$I$</th>
<th>$y_b$</th>
<th>$t_h$ (girders only)*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm² × 10³</td>
<td>mm³ × 10⁶</td>
<td>mm³ × 10⁶</td>
<td>mm⁴ × 10⁶</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>1</td>
<td>125</td>
<td>17.9</td>
<td>22.0</td>
<td>7 400</td>
<td>337</td>
<td>120</td>
</tr>
<tr>
<td>2</td>
<td>218</td>
<td>41.1</td>
<td>48.1</td>
<td>19 950</td>
<td>415</td>
<td>155</td>
</tr>
<tr>
<td>3</td>
<td>317</td>
<td>82.9</td>
<td>91.1</td>
<td>49 900</td>
<td>548</td>
<td>180</td>
</tr>
<tr>
<td>4</td>
<td>443</td>
<td>135.9</td>
<td>158.5</td>
<td>105 330</td>
<td>625</td>
<td>205</td>
</tr>
</tbody>
</table>

* See Clauses 6.1.7(b) and 6.1.8.2

where

- $A_g$ = area of the gross cross-section of the member
- $Z_t$ = section modulus about the centroidal axis at the top of an uncracked cross-section
- $Z_b$ = section modulus about the centroidal axis at the bottom of an uncracked cross-section
- $I$ = second moment of area of the gross concrete cross-section
- $y_b$ = depth from the centroidal axis to the extreme fibre at the bottom of the section
- $t_h$ = hypothetical thickness of the member

### TABLE H2(B)(1)
**FLEXURAL MODULI—PRECAST CONCRETE SUPER T-GIRDERS—OPEN TOP FLANGE CASE**

<table>
<thead>
<tr>
<th>Girder type</th>
<th>$t_b$</th>
<th>$A_g$</th>
<th>$Z_t$</th>
<th>$Z_b$</th>
<th>$I$</th>
<th>$y_b$</th>
<th>$t_h$ (girders only)*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm</td>
<td>mm² × 10³</td>
<td>mm³ × 10⁶</td>
<td>mm³ × 10⁶</td>
<td>mm⁴ × 10⁶</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>T1</td>
<td>240</td>
<td>436.0</td>
<td>100.7</td>
<td>95.2</td>
<td>36 700</td>
<td>385</td>
<td>154</td>
</tr>
<tr>
<td>T2</td>
<td>240</td>
<td>472.1</td>
<td>152.5</td>
<td>146.9</td>
<td>74 840</td>
<td>509</td>
<td>150</td>
</tr>
<tr>
<td>T3</td>
<td>260</td>
<td>514.0</td>
<td>197.0</td>
<td>193.5</td>
<td>117 140</td>
<td>605</td>
<td>150</td>
</tr>
<tr>
<td>T4</td>
<td>260</td>
<td>555.8</td>
<td>268.6</td>
<td>257.8</td>
<td>197 320</td>
<td>765</td>
<td>146</td>
</tr>
<tr>
<td>T5</td>
<td>325</td>
<td>691.9</td>
<td>361.4</td>
<td>352.9</td>
<td>321 390</td>
<td>911</td>
<td>163</td>
</tr>
</tbody>
</table>

* See Clauses 6.1.7(b) and 6.1.8.2

### TABLE H2(B)(2)
**FLEXURAL MODULI—PRECAST CONCRETE SUPER T-GIRDERS—CLOSED TOP FLANGE CASE**

<table>
<thead>
<tr>
<th>Girder type</th>
<th>$t_b$</th>
<th>$A_g$</th>
<th>$Z_t$</th>
<th>$Z_b$</th>
<th>$I$</th>
<th>$y_b$</th>
<th>$t_h$ (girders only)*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm</td>
<td>mm² × 10³</td>
<td>mm³ × 10⁶</td>
<td>mm³ × 10⁶</td>
<td>mm⁴ × 10⁶</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>T1</td>
<td>240</td>
<td>507.3</td>
<td>100.7</td>
<td>95.2</td>
<td>36 700</td>
<td>385</td>
<td>154</td>
</tr>
<tr>
<td>T2</td>
<td>240</td>
<td>543.3</td>
<td>152.5</td>
<td>146.9</td>
<td>74 840</td>
<td>509</td>
<td>150</td>
</tr>
<tr>
<td>T3</td>
<td>260</td>
<td>585.2</td>
<td>197.0</td>
<td>193.5</td>
<td>117 140</td>
<td>605</td>
<td>150</td>
</tr>
<tr>
<td>T4</td>
<td>260</td>
<td>627.1</td>
<td>268.6</td>
<td>257.8</td>
<td>197 320</td>
<td>765</td>
<td>146</td>
</tr>
<tr>
<td>T5</td>
<td>325</td>
<td>760.1</td>
<td>361.4</td>
<td>352.9</td>
<td>321 390</td>
<td>911</td>
<td>163</td>
</tr>
</tbody>
</table>

* See Clauses 6.1.7(b) and 6.1.8.2
### H4 TORSIONAL PROPERTIES

Torsional moduli \( J_{tb} \) of the standard precast prestressed concrete I-girder sections, together with torsional moduli \( J_{tn} \) of the I-girder sections with a composite slab connected above, taking into account the difference in elastic moduli of the girder and slab concretes, as shown in Figure H2(A), are given in Table H3(A).

Torsional moduli are given for typical Super T-girder sections that conform to the standard sections shown in Figure H1(B), with the following dimensions:

(a) Width of webs: 100 mm (for girder types 1 to 4). 120 mm (for girder type 5).

(b) Thickness of top flange: 75 mm.

(c) Width of top flange: 2100 mm.

(d) Thickness of bottom flange at centre-line: Dimension \( t_b \) (see Tables H2(B)(1) and H2(B)(2)).

Torsional moduli \( J_{tb} \) of the standard precast prestressed concrete Super T-girder sections, together with torsional moduli \( J_{tn} \) of the Super T-girder sections with a composite slab connected above, taking into account the difference in elastic moduli of the girder and slab concretes, shown in Figures H2(B)(1) and H2(B)(2), are given in Table H3(B)(1) for the open top flange case and in Table H3(B)(2) for the closed top flange case.

For the application of the torsional moduli, the following considerations apply:

(i) Torsional moduli, given in Tables H3(A), H3(B)(1) and H3(B)(2) are based on elastic theory and are equivalent to the Saint-Venant’s torsional constants.

(ii) The value of torsional modulus \( J_{tn} \) for a composite section is the torsional modulus for the girder plus the slab together with the junction effect between the girder and the cast-in-place slab.

NOTE: The contribution to \( J_{tn} \) from the cast-in-place deck slab is reduced to one half of the full amount because the continuity of the slab removes the effect of the vertical shear stresses that would otherwise be present at the free ends of the slab.

(iii) Values of \( J_{tn} \) are given for modular ratios \( \alpha_c \) of 0.70 and 1.00 where \( \alpha_c \) is the modular ratio of the cast-in-place concrete to the precast beam concrete in the composite member. Intermediate values may be interpolated.

(iv) The width \( b_s \) is the width of the flange in a composite member.

(v) The full torsional moduli are suitable for determining distribution of forces at applied loads only, that is, while the section is uncracked. At ultimate load, considerable reduction in the torsional stiffness may occur and the effect of using a torsional modulus equal to 20% of the full value should be taken into consideration.
FIGURE H2(A) STANDARD PRECAST PRESTRESSED CONCRETE I-GIRDER WITH COMPOSITE SLAB

TABLE H3(A)

TORSIONAL MODULI ($J_{tb}$) AND ($J_{tn}$) FOR SECTIONS USING STANDARD PRECAST PRESTRESSED CONCRETE I-GIRDERS

<table>
<thead>
<tr>
<th>Girder type</th>
<th>Torsional moduli</th>
<th>For girders in composite section, $J_{tn}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$d_s = 150$ mm</td>
<td>$d_s = 175$ mm</td>
</tr>
<tr>
<td></td>
<td>$\alpha = 0.70$</td>
<td>$\alpha = 1.00$</td>
</tr>
<tr>
<td>1</td>
<td>800</td>
<td>1100</td>
</tr>
<tr>
<td>2</td>
<td>2400</td>
<td>3500</td>
</tr>
<tr>
<td>3</td>
<td>5000</td>
<td>7100</td>
</tr>
<tr>
<td>4</td>
<td>10000</td>
<td>13000</td>
</tr>
</tbody>
</table>
FIGURE H2(B)(1) STANDARD PRECAST PRESTRESSED CONCRETE SUPER T-GIRDER (OPEN TOP FLANGE) WITH COMPOSITE SLAB

FIGURE H2(B)(2) STANDARD PRECAST PRESTRESSED CONCRETE SUPER T-GIRDER (CLOSED TOP FLANGE) WITH COMPOSITE SLAB
### TABLE H3(B)(1)

**TORSIONAL MODULI (J_{tb}) AND (J_{tn}) FOR SECTIONS USING OPEN TOP FLANGE STANDARD PRECAST PRESTRESSED CONCRETE SUPER T-GIRDERS**

<table>
<thead>
<tr>
<th>Girder type</th>
<th>( t_b ) mm</th>
<th>For girder only, ( J_{tb} )</th>
<th>Torsional moduli ( mm^4 \times 10^6 )</th>
<th>For girders in composite section, ( J_{tn} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>For ( d_s = 150 ) mm</td>
<td>( \alpha_c = 0.70 )</td>
<td>( \alpha_c = 1.00 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>For ( d_s = 175 ) mm</td>
<td>( \alpha_c = 0.70 )</td>
<td>( \alpha_c = 1.00 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>For ( d_s = 200 ) mm</td>
<td>( \alpha_c = 0.70 )</td>
<td>( \alpha_c = 1.00 )</td>
</tr>
<tr>
<td>1</td>
<td>240</td>
<td>6 300</td>
<td>66 000</td>
<td>75 000</td>
</tr>
<tr>
<td>2</td>
<td>240</td>
<td>6 100</td>
<td>103 000</td>
<td>116 000</td>
</tr>
<tr>
<td>3</td>
<td>260</td>
<td>6 900</td>
<td>136 000</td>
<td>151 000</td>
</tr>
<tr>
<td>4</td>
<td>260</td>
<td>6 400</td>
<td>181 000</td>
<td>200 000</td>
</tr>
<tr>
<td>5</td>
<td>325</td>
<td>9 900</td>
<td>244 000</td>
<td>265 000</td>
</tr>
</tbody>
</table>

### TABLE H3(B)(2)

**TORSIONAL MODULI (J_{tb}) AND (J_{tn}) FOR SECTIONS USING CLOSED TOP FLANGE STANDARD PRECAST PRESTRESSED CONCRETE SUPER T-GIRDERS**

<table>
<thead>
<tr>
<th>Girder type</th>
<th>( t_b ) mm</th>
<th>For girder only, ( J_{tb} )</th>
<th>Torsional moduli ( mm^4 \times 10^6 )</th>
<th>For girders in composite section, ( J_{tn} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>For ( d_s = 150 ) mm</td>
<td>( \alpha_c = 0.70 )</td>
<td>( \alpha_c = 1.00 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>For ( d_s = 175 ) mm</td>
<td>( \alpha_c = 0.70 )</td>
<td>( \alpha_c = 1.00 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>For ( d_s = 200 ) mm</td>
<td>( \alpha_c = 0.70 )</td>
<td>( \alpha_c = 1.00 )</td>
</tr>
<tr>
<td>1</td>
<td>240</td>
<td>49 000</td>
<td>67 000</td>
<td>76 000</td>
</tr>
<tr>
<td>2</td>
<td>240</td>
<td>83 000</td>
<td>109 000</td>
<td>121 000</td>
</tr>
<tr>
<td>3</td>
<td>260</td>
<td>114 000</td>
<td>145 000</td>
<td>160 000</td>
</tr>
<tr>
<td>4</td>
<td>260</td>
<td>156 000</td>
<td>196 000</td>
<td>214 000</td>
</tr>
<tr>
<td>5</td>
<td>325</td>
<td>215 000</td>
<td>261 000</td>
<td>282 000</td>
</tr>
</tbody>
</table>
APPENDIX I

STRUT-AND-TIE MODELLING

(Normative)

11 GENERAL

It shall be permissible to use strut-and-tie models to represent the conditions at overload and at failure in non-flexural members and in non-flexural regions of members, as a basis both for strength design and for evaluating strength.

A strut-and-tie model shall consist of compression elements (struts) and tension elements (ties) that are connected together at nodes to form a load-resisting structural system.

Strut-and-tie models shall satisfy the following requirements:

(a) Loads shall be applied at nodes, and the struts and ties shall be subjected only to axial force.

(b) The model shall provide load paths to carry the loads and other actions to the supports or into adjacent regions.

(c) The model shall be in equilibrium with the applied loads and the reactions.

(d) In determining the geometry of the model, the dimensions of the struts, ties, and nodal zones shall be taken into account.

(e) Ties shall be permitted to cross struts.

(f) Struts shall cross or intersect only at nodes.

(g) For reinforced concrete members at a node point, the angle between the axes of any strut and any tie shall be not less than 30°.

(h) For prestressed concrete members at a node point, the angle between the axes of any strut and any tie with a tendon acting as the reinforcement shall be not less than 20°.

12 CONCRETE STRUTS

12.1 Types of struts

Struts shall be of prismatic, fan or bottle shape, depending on the geometry of the compression field, as shown in Figure I2.1. Prismatic struts shall be used only where the compressive stress field cannot diverge.

12.2 Strut efficiency factor

For prismatic struts, the strut efficiency factor ($\beta_s$) that is used to determine the design strength shall be taken as 1.0.

For fan- and bottle-shaped compression fields that are unconfined, the strut efficiency factor shall be taken as—

$$\beta_s = \frac{1}{1.0 + 0.66 \cot^2 \theta} \quad \text{within the limits } 0.3 \leq \beta_s \leq 1.0$$

... I2.2

The angle ($\theta$) is measured between the axis of the strut and the axis of a tie passing through a common node (see Figure I2.2). Where more than one tie passes through a node, or where the angle ($\theta$) is different for nodes at each end of a strut, the smallest value of $\theta$ shall be used in determining $\beta_s$. 
FIGURE 12.1 TYPES OF STRUTS
12.3 Design strength of struts

The design strength of a concrete strut shall be taken as—

\[ \varphi_s \beta_s \cdot 0.9 f'_c A_c \]  

where

- \( A_c \) = smallest cross-sectional area of the concrete strut at any point along its length and measured normal to the line of action of the strut
- \( \beta_s \) = an efficiency factor given in Clause I2.2

The value of the strength reduction factor (\( \varphi_{st} \)) shall be obtained from Table 2.2.4.

Longitudinal reinforcement may be used to increase the strength of a strut. Such reinforcement shall be placed parallel to the axis of the strut, located within the strut and enclosed in ties or spirals satisfying Clause 10.7. The longitudinal reinforcement shall be properly anchored. The strength of a longitudinally reinforced strut may be calculated as for a prismatic, pin-ended short column of similar geometry.

12.4 Bursting reinforcement in bottle-shaped struts

The design bursting force at both the serviceability limit state (\( T_{bs}^* \)) and ultimate limit state (\( T_b^* \)) shall be calculated using an equilibrium model consistent with the bottle shape shown in Figure I2.4(A). The divergence angle (\( \alpha \)) for the bottle-shaped strut shall be assessed for each situation but shall be not less than—

(a) \( \tan \alpha = 1/2 \) ................................................................. for serviceability; and

(b) \( \tan \alpha = 1/5 \) ................................................................. for strength.

The bursting force across the strut at cracking shall be taken as—

\[ T_{bcr} = 0.7 b l_b f'_c \]  

where

- \( b \) = width of rectangular cross section or member
- \( l_b \) = length of the bursting zone [see Figure I2.4(A)]
If the calculated bursting force \( T^*_{b} \) is greater than \( 0.5T_{b,cr} \), then transverse reinforcement shall be provided in either—

(i) two orthogonal directions at angles \( \gamma_1 \) and \( \gamma_2 \) to the axis of the strut [see Figure I2.4(B)]; or

(ii) one direction at an angle \( \gamma_1 \) to the axis of the strut, where \( \gamma_1 \) shall be not less than 40° and shall satisfy the following—

(A) for serviceability

\[
\sum A_i f_{si} \sin \gamma_i \geq \max \left( T^*_{b}, T_{b,cr} \right) \quad \ldots \text{I2.4(2)}
\]

(B) for strength

\[
\varphi \sum A_i f_{sy} \sin \gamma_i \geq T^*_{b} \quad \ldots \text{I2.4(3)}
\]

In the above expressions, \( A_i \) is the area of reinforcement in directions 1 and 2 crossing a strut at an angle \( \gamma_1 \) to the axis of the strut [see Figure I2.4(B)] and \( f_{si} \) is the serviceability limit stress in the reinforcement as specified in Clause 12.4.

The transverse reinforcement shall be evenly distributed throughout the length of the bursting zone \( (l_b) \), which is given by—

\[
l_b = \sqrt{z^2 + a^2 - d_c} \quad \ldots \text{I2.4(4)}
\]

and \( a, d_c \) and \( z \) are the shear span, the width of the idealized strut, and the projection of the inclined compressive strut normal to the shear span respectively [see Figure I2.4(A)].
FIGURE I2.4(A)   MODEL OF BURSTING FORCES IN BOTTLE-SHAPED STRUTS
## 13 TIES

### 13.1 Arrangement of ties

Ties shall consist of reinforcing steel and/or prestressing tendons. The reinforcement and/or tendons shall be evenly distributed across the nodal regions at each end of the tie, and arranged such that the resultant tensile force coincides with the axis of the tie in the strut-and-tie model.

### 13.2 Design strength of ties

The design strength of a tie shall be taken as \( \phi_{st} [A_{st} f_{sy} + A_p (\sigma_{p,ef} + \Delta \sigma_p)] \) where \((\sigma_{p,ef} + \Delta \sigma_p)\) shall not exceed \(f_{py}\). The value of \(\phi_{st}\) shall be obtained from Table 2.2.4.

### 13.3 Anchorage of ties

To provide adequate anchorage at each end of the tie, the reinforcement or tendon shall be extended beyond the node to achieve the design strength of the tie at the node and anchored in accordance with Clause 13.1. At least 50% of the development length shall extend beyond the nodal zone.

Alternatively, anchorage of reinforcement may be achieved by a welded or mechanical anchorage, located entirely beyond the nodal zone.

## 14 NODES

### 14.1 Types of nodes

Three types of node are distinguished by the arrangement of the entering struts and ties, and the confinement thus provided, as follows:

(i) **CCC**—there are only struts entering the node.
Interim

(j) CCT—there are two or more struts and a single tension tie entering the node.
(k) CTT—there are two or more tension ties entering the node.

14.2 Design strength of nodes

Where confinement is not provided to the nodal region, the design strength of the node shall be such that the principal compressive stress on any nodal face, determined from the normal and shear stresses on that face, is not greater than $\phi_{st} \beta_n 0.9 f'c$ where—

(l) for CCC nodes $\beta_n = 1.0$; or
(m) for CCT nodes $\beta_n = 0.8$; or
(n) for CTT nodes $\beta_n = 0.6$.

The value of the strength reduction factor ($\phi_{st}$) shall be taken from Table 2.2.4.

Where confinement is provided to the nodal region, the design strength of the node may be determined by tests or calculation, considering the confinement, but shall not exceed a value corresponding to a maximum compressive principal stress on any face of $\phi_{st} 1.8 f'c$.

15 ANALYSIS OF STRUT-AND-TIE MODELS

In the analysis of a strut-and-tie model to determine the internal forces in the struts and ties, the requirements of Clause 7.1.1 shall be satisfied, and Clauses 7.1.2 and 7.7.2 shall be complied with.

16 DESIGN BASED ON STRUT-AND-TIE MODELLING

16.1 Design for strength

When strut-and-tie modelling is used for strength design, the requirements of Clause 2.2.4 shall be satisfied.

16.2 Serviceability checks

When design for strength is based on strut-and-tie modelling, separate checks shall be undertaken to ensure that the design requirements for serviceability are satisfied.
APPENDIX J

REFERENCES

(Informative)


