Limit states design of steel formwork shores

by

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PLEASE NOTE

The greatest amount of care has been taken while scanning this thesis,

and the best possible result has been obtained.
Abstract

Approximately half of all formwork collapses occur during concrete placement. Coincidentally, researchers who have measured the actual loads in formwork shores consistently found that they differ from those predicted. Indeed, there is some evidence that current methods tend towards underestimation.

In Australia, the current practice for designing steel formwork shores is set out in Australian Standard AS 3610 – 1990 Formwork for Concrete. When published, AS 3610 was the first national Standard to be published in limit states format. Significantly, AS 3610 departed from previous practice and introduced new methods for the design of formwork shores.

Prompted by the doubt cast on current practice and, in particular, the absence of any hard basis for the methods set out in AS 3610, the aim of this research is to develop new more reliable rules for the design of steel formwork shores. This is achieved using first-order probabilistic techniques to compare the reliability of new and reused shores designed in accordance with AS 3610 with current international permissible stress and limit states methods from British, American, Israeli and draft European formwork Standards.

For steel formwork shores in “new” condition, the AS 3610 limit states design load combinations proved to be less reliable than the design loads in other Standards and did not reach target reliability (safety) indices. Accordingly, new design load combinations are calibrated. However, unlike columns in normal steel construction formwork shores sustain damage from being dismantled and re-erected many times. In addition, accidental or unintentional end eccentricities are likely to occur due to the nature, conditions and erection procedures on construction sites. The presence of these additional imperfections introduces bending moments that reduce the axial capacity of shores. Therefore, the influence of additional imperfections is also examined and new design methods that explicitly take account of additional imperfections are presented.
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Preface

The author’s concerns about the reliability of the current Australian limit states methods for formwork design have arisen gradually since AS 3610 was first published. A preliminary investigation (Ferguson 2001) uncovered a theme of general concern about the adequacy of international formwork design loads in the literature. The same investigation found a disparity between Australian limit states design loads and the design loads in other recently published or soon to be published limit states Standards. Given this knowledge, ambivalence was not an option.

The results of this research have been published in two papers (Ferguson and Bridge 2001a) and (Ferguson and Bridge 2001b). In addition, Australian Standards Committee BD/43 has accepted the recommendations of the author and amended the design rules in AS 3610 – 1995 (SA 2003).
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Chapter 1  Introduction

1.1 Formwork

Formwork is a structure, usually temporary, erected to support and mould cast-in-situ concrete until it becomes self-supporting. Primarily, concrete is poured against a form face that bears directly on a framework of joists and bearers, which in turn distribute the load to supporting shores. Figure 1.1 shows the general arrangement of simple slab formwork.

![Diagram of formwork](image)

Figure 1.1  The general arrangement of simple slab formwork

Formwork is important because it has a major impact on the quality, cost and time to build concrete structures. In addition, the sound structural design of formwork is essential to ensure safety during construction.
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1.2 Problem

Unfortunately, the frequency of structural failure of formwork and the general risk of death during construction is much higher than later during the service life of the completed permanent structure.

Rightly or wrongly, in the past, structural engineers have focused more on the safety of permanent structures than the safety of temporary structures used during construction. The relative dearth of literature on the design of temporary structures is evidence of this imbalance and the potential for shortcomings. Of particular concern is the knowledge that researchers (Fattal 1983; Peng et al. 1994a, 1994b, 1996, 1997; Rosowsky et al. 1994a, 1994b, 1997; Kamala et al. 1996; Ikäheimonen 1997) who have measured the loads in formwork shores consistently report that, during concrete placement (when almost half of all formwork failures occur), the actual load in formwork shores differs from predictions and that current design methods tend to underestimation.

In Australia, the current practice for designing formwork was first published in Australian Standard AS 3610 – 1990 Formwork for concrete. Significantly, AS 3610 – 1990 was the first national Formwork Standard to be published in limit state format. Later in 1995, AS 3610 was revised but remained ostensibly unaltered. When AS 3610 was written there was little statistical data available on the action effects and resistance of formwork. Therefore, the design methods were specified based on experience and judgement.

1.3 Objective

Given the high frequency of formwork failure (especially during concrete placement), the doubt cast by recent research, the novelty, and absence of any hard basis for current practice, the aim of this research is to develop more reliable methods for the design of formwork, in particular, the design of steel shores that commonly support horizontal slab and beam formwork.
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(a) Adjustable steel prop

(b) Formwork frame

(c) Modular scaffold

Figure 1.2  Common types of formwork shores

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A wide range of proprietary formwork shores are available. Some of these are depicted in Figure 1.2, namely: adjustable steel props, formwork frames and modular scaffold.

In multistorey buildings, formwork shores are also used as backpropping to share the load of freshly cast concrete between previously poured lower floors. However, the design of shores for this situation is beyond the scope of this research.

1.4  Approach

To provide some background, Chapter 2 introduces the current Australian practice for designing steel formwork shores. A short history of Australian Standard AS 3610 – 1995 Formwork for Concrete is followed by an explanation of the limit states methods for determining design action effects and resistance. A design example illustrates how to determine the design action effects and select a suitable steel shore by using either standard steel sections or proprietary equipment.

Chapter 3 provides some perspective on the current Australian practice, as set in AS 3610 and explained in Chapter 2. This is achieved by first describing the methods for designing formwork shores set out in past Australian and other National Standards and Codes of Practice. In addition, recommendations from research that measured the actual load in formwork shores are considered.

Given the doubts raised about current practice, AS 3610 in particular, it is important to appreciate how the approach in AS 3610 differs from that in other Standards. Accordingly, Chapter 3 also presents a critique of aspects of the current practice set out AS 3610 – 1995. The criticism is supported by reference to the literature, past Australian Standards and other National Standards and Codes of Practice.

Chapter 4 explains why first-order probabilistic techniques were chosen to test reliability of formwork shores designed to the limit states permitted in AS 3610. The concept of reliability and reliability index are introduced. To select appropriate reliability targets, target indices chosen in the literature and compatible Standards are reviewed. In addition, Chapter 4 provides details of the action effect and
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resistance data chosen for analysis. Also included is a discussion of the reasons why this data was chosen and its limitations.

Chapter 5 addresses the relative reliability of steel formwork shores designed using limit states design loads in AS 3610 – 1995 Formwork for Concrete (SA 1995) and conforming to the requirements and tolerances specified in AS 4100 – 1998 Steel structures (SA 1998a). This is achieved by first developing a formwork specific expression for the reliability index $\beta$. This expression is then used to compare the relative reliability of the design load combinations from AS 3610 with past Australian and current international Standards. The results are presented in terms of reliability index $\beta$ curves, which are plotted over a range of concrete thickness. Two new design load combinations are introduced and calibrated to achieve target indices and a level of reliability similar to past Australian practice.

Chapter 6 investigates the influence of aspects of formwork and its construction that are not present in normal steel construction; thus, are not taken into account in the analysis in Chapter 5. Specifically, the influence of eccentric loading; and additional imperfections such as out-of-straightness and out-of-plumb are considered. Designers neglecting these effects might overestimate the member capacity. Accordingly, this Chapter investigates how they might be taken account of explicitly. This is achieved by deriving new column strength curves, which take account of the effects of additional out-of-straightness and eccentricity. In addition, the introduction of a partial resistance factor $\gamma$ proves a useful design concept. Using practical estimates for each type of imperfection, $\gamma$-curves are developed that demonstrate the influence of imperfections on the axial capacity of shores. Chapter 6 also includes a design example dealing with an out-of-straight and eccentrically loaded formwork shore.

Chapter 7 investigates whether, past and current national formwork Standards reliably address the effects of additional imperfections and unintentional end eccentricities. Some Standards deal with these issues explicitly, while others simply recommend larger factors of safety. Understanding how the different approaches compare is important. This is achieved by using the same first-order probability methods to determine the reliability of “new” formwork shores. In some cases, the
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resistance data for "new" steel columns is replaced by practical estimates of the strengths of formwork specified in the relevant Standard. Otherwise, the data for "new" columns is adjusted by specific resistance factors.

In a manner similar to Chapter 5, the results are presented in terms of reliability index $\beta$ curves, plotted over a range of concrete thickness. In addition, new capacity reduction factors are chosen by calibrating the relative reliability of the modified AS 3610 and new design loads (from Chapter 5) with past and current practice.

Chapter 8 presents a discussion of the research results and the conclusions drawn are presented in Chapter 9.

This is followed by a bibliography and three appendices. The appendices contain tables of action effect data, an example of the information provided by formwork shore manufacturers and tables of the data used in figures.

1.5 Definitions

**bearer** primary beam.

**falsework** the part of the formwork that supports the form and transfers all the loads to a stable surface.

**joist** secondary beam.

**shore** part of the falsework that carries the vertical loads, e.g. prop, scaffold or frame leg.

**simple method** a method for calculating the load in a shore that assumes the tributary area for the shore extends halfway to the adjacent shore on all sides.

**tributary area** the plan area of formwork contributing to the load in the formwork shore.
Chapter 2  Current Australian Practice

2.1  Introduction

Researchers suggest that there may be a problem with the current practice for designing formwork shores. As almost half of all formwork failures occur during concrete placement (Hadipriono and Wang 1986), it is most likely that the problem lies with estimates of the loads expected during concrete placement or the resistance of the formwork itself.

This Chapter introduces the current Australian practice for the design of formwork shores. Occupational Health and Safety Regulations (WorkCover NSW 2001) require the elimination or control of risks that arise at places of work. To this end, the regulations require that formwork must comply with AS 3610 – 1995 Formwork for Concrete (SA 1995). Accordingly, in Australia the methods set out in AS 3610 – 1995 represent current practice and are described herein.

A short account of the history of AS 3610 is followed by an explanation of the limit states methods for designing formwork shores, including the rules for determining the design action effects and design resistance. An example is used to illustrate the application of the methods to either: design a shore using standard steel sections; or select a proprietary shore based on information supplied by the manufacturer.

2.2  AS 3610 – 1995

In 1975, the Standards Association of Australia established a policy of a general unified approach for the design of all types of structures using limit states design (SAA 1975). In 1984, in keeping with that policy, Standards Australia Committee BD/43 set out to write a new Standard (AS 3610) that would include limit states design rules for formwork.

At that time, there was little guidance available on limit states design. No other national Formwork Standards were available in limit states format and there was
little data available on the action effects and resistance of formwork. Indeed, related limit states material structural Standards, although under development, were yet to be published. Thus, Committee BD/43 had to rely on experience and judgement to develop the limit states design rules in AS 3610 using an approach often referred to as a “soft conversion”.

In fact, AS 3610 – 1995 sets out both permissible stress and limit states methods. However, the permissible stress methods were only intended for use until other relevant material Standards were available in limit states format (SA 1996a). Now (2003) that all relevant material Standards are published in limit state format, and earlier permissible stress Standards have been withdrawn, it is appropriate to use limit states methods.

**Limits States**

AS 3610 requires that the formwork assembly and its components satisfy stability, strength and serviceability limit states. For steel formwork shores, strength limit state usually governs design and is satisfied if:

\[
S^* \leq \phi R_u
\]  

(2.1)

In Equation 2.1:

- \(S^*\) represents a design action effect due to the most adverse limit state design load combination given in AS 3610 Table 4.5.1; and

- \(\phi R_u\) represents the design resistance of the shore determined in accordance with limit state procedures, set out in the appropriate material structural design code or by testing in accordance with AS 3610 Clause 4.5.3.

**Design Action Effect, \(S^*\)**

For formwork shores, the most adverse vertical load combination in Table 4.5.1 might be either:
\[ S^* = 1.25G_F + 1.25G_c + 1.5Q_{uw} + 1.5M, \text{ kN/m}^2 \] (2.2)

or

\[ S^* = 1.25G_F + 1.25G_c + Q_c, \text{ kN/m}^2 \] (2.3)

In Equations 2.2 and 2.3:

- \( G_F \) represents the weight of the formwork;

- \( G_c \) represents the weight of the concrete (including an allowance for reinforcement);

- \( Q_{uw} \) is an imposed action of 1.0 kN/m\(^2\) for the weight of workmen and their equipment;

- \( M \) is an imposed action for the weight of stacked materials; and

- \( Q_c \) is an allowance for the localised mounding of concrete of 3.0 kN/m\(^2\) over a square area of 1.6 m x 1.6 m at any location and zero over the remainder.

As it is impractical to stack materials on wet concrete, during concrete placement it is assumed \( M = 0 \) kN/m\(^2\). In practice, it is common for formwork designers to completely neglect stacked materials or nominate only a small area where materials may be stacked after concrete placement.

In addition to axial compression, AS 3610 Clause 4.3.3 requires a nominal eccentricity (\( e \)) to be taken into account. For the design of formwork shores, the presence of eccentricities is important because they introduce bending moments, which reduce axial capacity. In AS 3610, the design eccentricity \( e \) is intended to take account of eccentricities arising from:

- eccentric vertical loading, see Figure 2.1; or
- irregular or variable stiffness bearing surfaces, see Figure 2.2;
the component of out-of-straightness permitted in AS 3610 Table 5.3.1 \( (L/300) \) and not taken into account under the relevant material Standard, e.g. the rules in AS 4100 take into account an initial out-of-straightness of \( L/1000 \), which leaves a nominal additional out-of-straightness \( (L/300 - L/1000 = L/400) \) to be considered, see Figure 2.3; and

- eccentricities arising from erecting shores (with base/end plates) out-of-plumb, see Figure 2.4.

(a) Continuous bearer positioned eccentric to shore centreline (McAdam 1993)

(b) Discontinuous bearers lapping on shore (SA 1996a)

Figure 2.1 Examples of eccentric loadings (McAdam 1993 and SA 1996a)
Figure 2.2 Examples of irregular or variable stiffness bearing surfaces (SA 1996a)
Figure 2.3  A representation of the additional out-of-straightness permitted in AS 3610

Figure 2.4  Examples of eccentric reactions (McAdam 1993)
The value of the design eccentricity $e$ is given by:

$$e = e'' + \frac{L}{200}$$

(2.4)

In Equation 2.4,

- $e''$ represents the expected eccentricity, which is the lesser of $0.25b_p$ ($b_p$ is the width of the stiff portion of bearing of an end plate), 0.25 times the bearer width or 40 mm; and

- $L$ represents the overall length of the strut, i.e. the distance from the base plate to the underside of the bearer.

The value of design eccentricity $e$ at each end of the strut may differ. For braced structures, it is recommended that the bending moment from the maximum eccentricity be assumed to apply over the full length of the member (SA 1996a).

**Design Resistance, $\phi R_u$**

AS 3610 Clause 4.5.1 permits the design resistance of formwork components or assemblies to be determined by calculation in accordance with the appropriate material structural design code or by testing in accordance with Appendix A.

**Calculation**

For steel shores, AS 4100 (SA 1998a) is the appropriate material structural design code. AS 3610 Clause 4.5.1 stipulates that where the requirements of AS 4100 and AS 3610 differ, the requirements of AS 3610 take precedence. For steel shores, examples of such differences include: loads; load combinations; minimum eccentricities; and column out-of-straightness and out-of-plumb tolerances.

For limit states design, it is necessary to take account of second-order effects. Second-order effects arise from the design loads acting on the structure in its displaced and deformed configuration. It is common practice to assume formwork
structures act as top restrained braced frames\textsuperscript{1}. Assuming an elastic analysis, AS 4100 permits taking account of second-order effects by either: amplifying first-order elastic moments; or performing a full second-order elastic analysis.

For a first order elastic analysis with moment amplification, AS 4100 Clause 8.4.2.2 requires the member to satisfy:

\[
M^* = \delta_b M_m^* \leq \phi M_s \left(1 - \frac{N^*}{\phi N_c}\right)
\]  

(2.5)

In Equation 2.5,

\(\delta_b\) is an amplification factor to take account of second order effects and is given in Equation 2.6;

\(M_m\) is the maximum first-order bending moment along the length of a member or segment;

\(\phi\) is the capacity reduction factor for a member subject to combined actions;

\(M_s\) is the nominal section moment capacity;

\(N^*\) is the design axial compressive force;

\(\phi\) is the capacity reduction factor for axial compression; and

\(N_c\) is the nominal member capacity in compression.

\textsuperscript{1} The connections in some proprietary formwork shores systems display semi-rigid behaviour. A more accurate assessment of the capacity of such systems can be made using more complex techniques (Godley and Beale 1997; and Vaux 2000).
\[ \delta_b = \frac{c_m}{1 - \left( \frac{N^*}{N_{omb}} \right)} \geq 1 \quad (2.6) \]

In Equation 2.6,

\( c_m \) is factor to take account of unequal moments and is given in Equation 2.7; and

\( N_{omb} \) is the elastic flexural buckling load.

\[ c_m = 0.6 - 0.4 \beta_m \leq 1.0 \quad (2.7) \]

In Equation 2.7,

\( \beta_m \) is the ratio of the smaller to the larger bending moment at each end of the member, taken as positive when the member is bent in reverse curvature; thus, for an eccentrically loaded member with equal and opposite end moments \( \beta_m = -1.0 \) and \( c_m = 1.0 \).

Importantly, the use of an amplified first order elastic analysis is only permitted if \( \delta_b < 1.4 \). This effectively limits the axial capacity of a shore to 28.5\% of its elastic flexural buckling load.

**Testing**

AS 3610 Appendix A describes the requirements for testing formwork. Compression members, such as shores, are tested in the least favourable configuration; i.e. out-of-plumb and loaded eccentrically. The out-of-plumb for testing is specified as \( L/400 \) and the value of eccentricity given by \( e'' \) in Equation 2.4.

After analysing the test results, the design resistance can be determined from:
In Equation 2.8,

\[ \phi R_u = \frac{\overline{x}}{k_s} \]  

\( \overline{x} \) is the mean value of test data; and

\( k_s \) is a sampling factor, given in AS 3610 Table A1, to take account of variability of results and confidence in the reliability of sample size.

In practice, there is a wide range of proprietary equipment available from which an appropriate formwork shore can be selected. To help with selection, manufacturers publish product capacities. To avoid possible confusion between permissible stress and limit states design resistance values, AS 3610 requires manufacturers publish a safe working load (SWL) and a limit states conversion factor (LSCF). The product of the SWL and LSCF equates to the limit state design resistance \( \phi R_u \), as shown in Equation 2.9.

\[ \phi R_u = (LSCF)_{SWL} \]  

(2.9)

For manufacturers, guidance on choosing an appropriate value of the conversion factor is given in the Commentary to AS 3610 (SA 1996a), such that for formwork shores:

\[ LSCF \geq \frac{1.25G_f + 1.25G_c + 1.5Q_{cr} + 1.5M}{G_f + G_c + Q_{cr} + M} \]  

(2.10)

and

\[ LSCF \geq \frac{1.25G_f + 1.25G_c + Q_c}{G_f + G_c + Q_c} \]  

(2.11)

The following worked examples illustrate the procedure for determining the design action effects and resistance of formwork shores.
2.3 Example

This example follows the methods set out in the Commentary to AS 3610 (SA 1996a) and a technical paper by Gardiner (Gardiner 1989), both references were intended to provide authoritative guidance on the application of the design methods set out in AS 3610.

Example

Determine an appropriate shore to support formwork during placement of concrete for a 500 mm thick concrete slab. The formwork construction consists of joists spanning 1.83 m and bearers spanning 1.22 m between shores, as shown in Figure 2.5. The shores themselves are required to support the formwork 4.0 m above the foundation.

\[ G = 0.3 \text{ kN/m}^2, \]
\[ G_c = 0.5 \times 25.0 = 12.5 \text{ kN/m}^2, \]
\[ Q_{uw} = 1.0 \text{ kN/m}^2, \]
\[ Q_c = 3.0 \text{ kN/m}^2 \]
over (1.6 m x 1.6 m), and
\[ M = 0.0 \text{ kN/m}^2. \]

![Figure 2.5 Formwork layout plan](image)

Assumptions

The design will be based on the following assumptions:
the formwork is laterally braced in the plane of the deck such that the shores are effectively top restrained;

- environmental actions can be safely neglected; and

- accidental actions can be safely neglected.

**Action effects**

To calculate the axial force in the shores, first consider the combined action of the weight of formwork and concrete plus an allowance for workmen and equipment:

\[
S_1^* = \{1.25(0.3) + 1.25(12.5) + 1.5(1.0) + 1.5(0)\} = 17.5 \text{ kN/m}^2
\]

\[
N_1^* = \{17.5 \times 1.83 \times 1.22\} = 39.1 \text{ kN}
\]

(2.12)

(2.13)

Next, consider the combined action of the weight of formwork and concrete plus an allowance for localised mounding of concrete:

\[
S_{2A}^* = \{1.25(0.3) + 1.25(12.5)\} = 16.0 \text{ kN/m}^2
\]

\[
S_{2B}^* = \{1.0(3.0)\} = 3.0 \text{ kN/m}^2
\]

\[
N_2^* = \{(16.0 \times 1.83 \times 1.22) + (3.0 \times 1.6 \times 1.22)\} = 41.6 \text{ kN}
\]

(2.14)

(2.15)

(2.16)

Comparing Equations 2.13 and 2.16 shows that the most adverse action effect is \(N_2^* = 41.6 \text{ kN}\).

To determine the design eccentricity \(e\), first determine the value of \(e''\), which is the lesser of:

- 0.25\(b_p\) = 19 mm, based on an 8 mm base plate thickness and 60 mm diameter shore section;

- 0.25 x bearer width = 25 mm, based on a 100 mm wide bearer; or

40 mm.

Therefore, \(e'' = 19 \text{ mm}\).
By substituting, \( e'' = 19 \text{ mm} \) and an overall length of the shore \((L)\) of 4000 mm into Equation 2.4, the design eccentricity can be determined as:

\[
e = 19 + \frac{4000}{200} = 39 \text{ mm}
\]

(2.17)

In this example, the limit state design action effects are an axial force \( N^* = 41.6 \text{ kN} \) and a bending moment of \( M^* = N^* e = 41.6(0.039) = 1.62 \text{ kNm} \).

**Resistance**

To determine a suitable section for the shore, a designer might:

- Design a purpose made shore using standard steel sections; or
- Choose a proprietary shore by referring to manufacturers data sheets.

To design a purpose made shore using standard steel sections, assume the shore is braced at mid-height and eccentrically loaded. This means it will be subject to the combined effects of axial compression and bending.

As recommended in (SA 1996a), assume that the eccentricities cause a uniform moment along the full length of the shore. In this case, \( \beta_m = -1.0 \) and from Equation 2.7, \( c_m = 1.0 \).

Try a common formwork shore section, namely: 60.3CHS3.6 G250. Substitute into Equation 2.5 and 2.6, for an effective length of 2.0 m, \( N_{omb} = 128 \text{ kN} \) (AISC 1992), \( \phi N_c = 87.1 \text{ kN} \), \( \phi M_c = 2.61 \text{ kNm} \) (AISC 1999) and \( c_m = 1.0 \).

\[
\delta_h = \frac{1.0}{1 - \left( \frac{41.6}{128} \right)} = 1.48
\]

(2.18)

In this example, \( \delta_h > 1.4 \); thus, to check this section further, a full second-order analysis is required.
Try a stronger section, namely: 60.3CHS5.4 G250. Substitute into Equation 2.5 and 2.6, an effective length of 2.0 m, \( N_{omb} = 175 \text{kN} \) (AISC 1992), \( \phi N_c = 122 \text{kN} \), \( \phi M_s = 3.67 \text{kNm} \) (AISC 1999) and \( c_m = 1.0 \).

\[
\delta_b = \frac{1.0}{1 - \left( \frac{41.6}{175} \right)} = 1.31
\]

In this case, \( \delta_b < 1.4 \); therefore,

\[
M' = 1.31(1.62) = 2.12 \text{kNm}
\]

\[
\phi M_s \left( 1 - \frac{N^*}{\phi N_c} \right) = 0.9(3.67) \left( 1 - \frac{41.6}{122} \right) = 2.18 \text{kNm}
\]

\[2.12 \leq 2.18, \text{ OK}\]

Alternatively a proprietary shore might be chosen by referring to manufacturers information. For the purposes of this example a fictitious example of such information is included in Appendix B.

In this case, the shore capacities given in Figure B.1 are appropriate to use because the expected eccentricity \( e'' = 19 \text{ mm} \) and does not exceed the published test eccentricity (19 mm).

To achieve a strutting height of 4.0 m, try 2/1829 frames with top and bottom jacks extended 171 mm. From Figure B.1, for a jack extension of 170 mm, the maximum allowable leg load is 45 kN. To convert the allowable loads to limit states capacities, Appendix B specifies that the limit state conversion factor LSCF = 1.31. Substituting into Equation 2.9 gives the limit states capacity as:

\[
\phi R_u = (1.31)45 = 58.9 \text{kN}
\]

\[41.6 \leq 58.9, \text{ OK}\]
Chapter 2  Current Australian Practice

Use either: 2/1829 proprietary formwork shore frames with top and bottom jacks extended 171 mm; or 60.3 x 5.4 CHS G250 with 8 mm base plate and U-head braced at mid height in both directions (assume adjustment for stripping is achieved by wedges under the base plate).

2.4 Conclusions

In Australia, the current practice for designing formwork shores is set out in AS 3610 – 1995. Since all relevant permissible stress material Standards have been replaced with limit states versions, the limit states procedures in AS 3610 are the recommended current practice.

To determine the design load in a formwork shore during concrete placement, AS 3610 requires consideration of the most adverse of two situations: the effects of the weight of formwork and concrete plus an allowance for the weight of workmen and equipment; and the effects of the weight of formwork and concrete plus an allowance for the localised mounding of concrete.

When selecting suitable formwork shores, in addition to the axial force, AS 3610 requires that the effects of eccentric loading, out-of-plumb and out-of-straightness be taken into account.

The resistance of formwork shores may be determined by calculation or tests. Calculations must take account of second-order effects. Lightly loaded shores \(N^e < 0.286N_{om}\), may be designed by reiterative first-order elastic methods. To achieve economies, especially when using slender shores, it is most likely that a full second-order elastic analysis will be required. Alternatively, proprietary shores may be selected using information supplied by manufacturers, which has been derived from tests of eccentrically loaded specimens.
Chapter 3  A Critique of Current Australian Practice

3.1 Introduction

A critical review of the current Australian practice for the design of formwork shores is warranted because:

- the frequency of structural failure of formwork and the general risk of death during construction is much higher than later during the service life of completed permanent structures;
- almost half of all falsework failures in concrete structures occur during concrete placement (Hadipriono and Wang 1986);
- researchers who have measured the loads in formwork shores (Fattal 1983; Peng et al. 1994a, 1994b, 1996, 1997; Rosowsky et al. 1994a, 1994b, 1997; Kamala et al. 1996; Ikäheimonen 1997) consistently report that, during concrete placement, the actual load in formwork shores differs from predictions and that current design methods tend to underestimation; and
- without any “hard” basis, current Australian limit states methods depart from previous Australian and current international practice.

The previous chapter sets out the current Australian methods for designing steel shores that support formwork during concrete placement. It is important to understand how they differ from current international and past Australian practice, as well as how they compare with the results of recent research.

This chapter starts by presenting the methods for designing formwork shores as set out in previous Australian and other National Standards/Codes of Practice. This is followed by an overview of relevant research published in the literature. The methods for designing formwork shores are then compared with that in AS 3610 – 1995.
3.2 AS 1509 – 1974

From 1974, until superseded in 1990, AS 1509 – 1974 SAA Formwork Code (SAA 1974) sets out the rules for design and construction of formwork in Australia. AS 1509 permitted the design of steel formwork shores using permissible stress or ultimate strength design methods.

For formwork shores supporting vertical loads, AS 1509 required that both dead and superimposed loads be taken into account, specifically: the weight of formwork, freshly placed concrete, reinforcement and other embedded materials; and the superimposed load of workmen, equipment, runways, stacked materials as well as an allowance for impact. The value of the superimposed loads could not be less than the most severe of 1.9 kN/m² on the finished concrete plan area or a single isolated load of 2.3 kN at any point on the structure. Expressed as an equation, using similar notation to AS 3610, these requirements take the form:

\[ S = G_t + G_c + Q_v + M, \text{ kN/m}^2 \]  

(3.1)

The new terms in Equations 3.1 are:

\[ Q_v \] is a superimposed action of 1.9 kN/m² for the weight of workmen, equipment, runways and an allowance for impact; and

\[ M \] is a superimposed action for the weight of stacked materials such that the effect of the superimposed actions \( Q_v + M \) could not be less than that of a single point load of 2.3 kN located anywhere on the structure.

When the permissible stress method was used to design reusable formwork components, the maximum permissible stress was limited to 85% of that given in the applicable code, (SAA 1981). Effectively, this increased the nominal safety factor for permissible stress methods from 1.67 to approximately 2.0.

Significantly, AS 1509 required steel formwork shores to be straight and true within the tolerances of the relevant material Standard. AS 1509 provided little design
guidance on taking account of eccentric loading of formwork shores, other than to highlight that unsymmetrical or eccentric loading due to placement sequences may cause failure.

At the time, the SAA Formwork Committee recommended that all proprietary formwork be designed using the ultimate strength method. In this case, the allowable load was derived by dividing the ultimate strength (from testing) by a specified load factor (factor of safety), e.g. a load factor of 2.5 for formwork frames and 3.0 for single steel shores (props).

3.3 Other National Standards and Codes of Practice

Internationally, formwork design methods are in a state of transition from permissible stress to limit states. The American Standard ACI 347:2001 Guide to Formwork for Concrete (ACI 2001) and British Standard BS 5975:1996 Code of practice for Falsework (BS 1996) remain in permissible stress format. Despite the difficulty of attempting to compare permissible stress and limit states methods, both documents are authoritative references, often quoted in research, and warrant inclusion in this review. In addition, two limit states references are reviewed, namely: Israeli Standard SI 904: 1998 Part 1: Formwork for Concrete: Principles (SII 1998); and the draft European Standard prEN 12812: 2000 Falsework – Performance requirements and general design (ECS 2000). SI 904 is the only other National Standard (other than AS3610) published in limit states format. prEN 12812 is close to publication and when published will supersede much of BS 5975.

ACI 347

In the case of ACI 347, the design load for formwork shores is determined by considering the sum of the weight of the formwork \((G_f)\), the weight of the concrete \((G_c)\) plus a live load \((Q_v)\) of 2.4 kN/m\(^2\). The latter allows for the weight of workmen, equipment, material storage, runways and impact, but not motorised carts. The minimum permitted design load is 4.8 kN/m\(^2\). These rules expressed as an equation gives:
Chapter 3  A Critique of Current Practice

\[ S = G_t + G_c + Q_v \geq 4.8 \text{ kN/m}^2 \]  \hspace{1cm} (3.2)

Authoritative guidance on ACI 347 is found in Formwork for Concrete (Hurd 1995). Hurd suggests a “Simple Method” to determine the load in a shore, specifically:

It is generally sufficiently accurate to assume that each shore or scaffolding leg supports a formwork area extending halfway to the adjacent shore or leg on all sides.

In a similar manner to AS 1509, ACI 347 highlights that insufficient allowance for unsymmetrical or eccentric loading due to placement sequences is a design deficiency that might lead to failure. There is no specific guidance on taking account of eccentricities in shores in ACI 347, although, Hurd advises that it is important to avoid eccentric loading by centralising bearers on U-heads and top plates of frame legs.

ACI 347 suggests designers adopt the load-carrying capacity recommended by formwork shore manufacturers and supported by tests. In America, the Scaffolding Shoring and Forming Institute recommend a minimum safety factor of 2.5 based on the ultimate load capacity established by test procedures (SSFI 1999). The SSFI procedures recommend testing randomly selected specimens that exhibit variations in measurements compatible with mill tolerances. It is unclear whether this meant the only the selected specimens should comply with the tolerances in the relevant steel Standard. This might be the case, as Hurd reiterates the recommendation of Committee 347 to use reduced values for allowable loads for shoring components that experienced substantial reuse.

**BS 5975**

Designers complying with BS 5975 are required to consider the effects of a uniformly distributed load consisting of the weight of formwork \((G_t)\) and the weight of the concrete \((G_c)\) as well as a live load \((Q_v)\) of 1.5 kN/m² to allow for: construction operatives; hand tools and small equipment; materials for immediate use; and common situations of impact and heaping of concrete occurring during placing operations. In addition, provision must be made for the weight of stacked materials \((M)\) in excess of 1.5 kN/m². These rules expressed as an equation give:
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\[ S = G_t + G_c + Q_v + M_i \text{ in kN/m}^2 \]  \hspace{1cm} (3.3)

When determining the load in a vertical member, BS 5975 requires designers to consider the formwork design in more detail, as follows.

1  If the vertical member supports primary and secondary beams that are simply supported over a single span, the “Simple Method” (similar to that described by Hurd) is satisfactory; however

2  If the vertical member supports primary or secondary beams that are continuous over two or more spans then:

   (i) where random length beams are used, the load calculated using the “Simple Method” is increased by 10%; or

   (ii) where specific conditions can be identified, such as beams on three supports, a more “Detailed Method” that takes account of the beams continuity may be justified.

For steelwork, BS 5975 permits the capacity of a component to be determined using permissible stress methods. However, when calculations are impractical, testing is recommended. When testing falsework equipment, BS 5975 Clauses 3.9.3 and 3.9.4 require, for various configurations, a minimum safety factor of 2.0 against collapse in the worst acceptable condition. As an example, the worst acceptable condition for adjustable steel props is for props erected 1.5° off vertical and eccentrically loaded up to 25 mm. BS 5975 requires that designers and manufacturers specify tolerances, such as column inclination, out-of-straightness and eccentricity, that are appropriate for the formwork.

SI 904

During concrete placement, the Israeli Standard SI 904 requires designers to consider the permanent load from the weight of the formwork and the weight of the concrete, as well as the action effects arising from either:
a variable load \((Q_{v1})\) of at least 3.0 kN/m\(^2\), which includes the weight of workmen and equipment, storage of equipment for concrete placement, access and the dynamic effect of falling concrete during placement; or

a temporary point load \((Q_{v2})\) of 2.0 kN from the weight of piling fresh concrete during placement.

Except for special concrete, a weight of 26 kN/m\(^3\) for fresh concrete including reinforcement is recommended. The partial load factors for ultimate limit states are 1.4, 1.6 and 1.2 for permanent \((G_f, G_c)\), variable \((Q_{v1})\) and temporary loads \((Q_{v2})\), respectively. Thus, the ultimate design loads are given by the following equations:

\[
S^* = 1.4(G_f + G_c) + 1.6Q_{v1}, \text{ in kN/m}^2 \tag{3.4}
\]

\[
S^* = 1.4A(G_f + G_c) + 1.2Q_{v2}, \text{ in kN} \tag{3.5}
\]

In Equation 3.5, \(A\) represents the area of formwork and concrete supported by the shore.

SI 904 Clause 2.3.4 sets out methods for determining the design strength of formwork or its components by dividing their characteristic strength by a formwork specific material partial safety factor, i.e. \(\gamma_m = 1.15\) rather than \(\gamma_m = 1.08\) specified in Israeli Steel Standard SI 1225 Part 1. An additional safety factor is required in special risk conditions.

**prEN 12812**

Using notation similar to AS 3610, the draft prEN 12812 requires designers to consider the combination of: the permanent action of the formwork \((G_f)\); the imposed actions of the concrete \((G_c)\); a minimum action of workmen and equipment \((Q_e)\) equivalent to 0.75 kN/m\(^2\); and a transient in-situ concrete loading allowance \((Q_c)\) of 10% of the weight of the concrete. The in-situ loading allowance \((Q_c)\) shall not be less than 0.75 kN/m\(^2\) or more than 1.75 kN/m\(^2\) acting on a square area of 3 m by 3 m. This can be expressed by the following equation.
\[ S^* = 1.35G_f + 1.5G_c + 1.5Q_v + 1.5Q_c, \text{ in kN/m}^2 \]  

(3.6)

In prEN 12812, the design resistance is determined by dividing the characteristic strength given in the relevant material Standards by a partial safety factor for the material, nominally \( \gamma_M = 1.1. \) If the characteristic strength is unknown, it may be established by tests. In either case, account shall be taken of imperfections, such as out-of-straightness, unintentional end and joint eccentricities.

Significantly, except where the design and drawings will be fully detailed to the standard of permanent works construction and the formwork is erected to the level of workmanship appropriate for permanent construction, prEN12812 requires the design resistance value of formwork and its components (calculated in accordance with the relevant material Standards) to be reduced by dividing the design resistance by a factor \( \gamma_T = 1.15. \)

### 3.4 Research by Others

Until authorities (prompted by the high incidence of structural failures during construction) identified a need for reliable data on construction loads, there had been little research that measured the actual load in formwork shores. Those who have measured the loads in formwork shores during concrete placement include the following researchers.

**Fattal (1983)**

Among the first to collect such data, Fattal (1983) measured the load in the shores used in the construction of a six-storey building. The building floor slabs were 200 mm thick, constructed of regular weight (2400 kg/m\(^3\)) concrete. The slab soffit formwork consisted of 16 mm plywood spanning over aluminium joists spaced at 488 mm centres. The joists spanned 3.05 m between aluminium stringers supported by steel shores spaced at 1.2 m centres. A crane delivered concrete in 1.5 m\(^3\) skips to the formwork.
Fattal continuously measured shore loads during and after concrete placement. The loads measured were not uniform and did not correlate with an analysis that included consideration of the continuity of joists. He assumed that the disparity was a result of either residual forces in the shores, or lack of bearing when the shores were first installed. Fattal suggested that improved workmanship and consideration of overloads in design might be required.

In addition, he found that during concrete placement, the maximum dynamic effect of discharging concrete out of the skip was equivalent to a 1.6 kN/m² uniformly distributed load.

After concrete placement, Fattal observed the crane placing material and equipment on the newly poured slab. This was part of the construction activity for the next floor. The weight and dynamic effects of these super-imposed loads increased the load on the shores below. For the worst case, when the load was directly above a shore, the effect was equivalent to a uniform distributed load of 1.8 kN/m².

After analysing all load effects and comparing the maximum shore loads with the design loads, specified in ACI 347, he concluded that a design live load of 2.4 kN/m² was generally adequate to compensate for dynamic effects of concrete placement and the superimposed construction loads on the newly placed slab. He warned that the load in some shores exceeded the ACI 347 design load by up to 18%. However, he considered this tolerable in view of the overall safety margin of 2.5 applicable to shores.

**Ayoub and Karshenas (1994)**

Ayoub and Karshenas (1994) investigated probability models for live loads on newly poured concrete slabs during the construction of multistorey buildings. The objective of this research was to address concerns about the safety of buildings during construction. These concerns had arisen because of the lack of any apparent basis for the formwork design live loads specified in existing standards.
Ayoub and Karshenas collected load data from 22 buildings in 12 cities. The buildings included hotels, offices, apartments, car parks and a sports centre. The material surveyed included: portable toilets; carts; a carpenter’s table; metal column forms; metal containers; tool boxes; map stands; barrels of water; and stacks of reinforcement, metal frames, steel braces, timber, aluminium beams and scaffold. They weighed every piece of material and mapped its location.

The conclusion from a statistical analysis of the data (Karshenas and Ayoub 1994) was that the standard deviation was area dependent. Karshenas and Ayoub suggested a possible design live load specification, based on 0.99 fractiles, would be an equivalent uniformly distributed load, $Q_v$, where:

$$Q_v = 2.4 \text{ kN/m}^2, \text{ for } A \text{ less than } 15 \text{ m}^2$$

or

$$Q_v = 2.4 \left( \frac{0.04 + \frac{2.32}{\sqrt{A}}}{1.7} \right) \text{ kN/m}^2, \text{ for } A \text{ greater than } 15 \text{ m}^2$$

where $A$ is the influence area calculated considering the continuity of the formwork in two directions.

Karshenas and Ayoub (1994) considered the continuity of the formwork over two spans when calculating the influence area for the axial force in shores, and one span when calculating the influence area for bending and shear in stringers and joists. They observed that for influence areas greater than 15 m$^2$ the suggested design load is less than that specified in ACI 347. They warned that the reduced design load is only applicable when concrete placement impact loads are not very high (e.g. placing concrete with a pump) because when concrete is placed with buckets (skips) or motorised buggies, the combined static and dynamic loads may be substantially higher than equipment and material storage loads (Karshenas and Heinrich 1994).
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Peng et al. investigated the load effects of concrete placement patterns on high-clearance falsework. As before, concerns about the safety of falsework during concrete placement were the motivation for the research, in particular, concerns about the high incidence of accidents occurring in high-clearance scaffolds, particularly in Taiwan.

In a series of papers (Peng et al. 1994a, 1994b, 1996, 1997), Peng et al. investigated the load effects of influence surfaces associated with different concrete placement load paths. They tested their hypothesis, developed in early papers, by measuring the load in shores on a full-scale falsework model. Sand bags placed on the formwork simulated concrete placement. The falsework, light braced frames, was set out on a uniform 1.83 m by 0.76 m grid. The sandbags simulated the weight of a 200 mm thick slab. Their results demonstrated that the maximum axial force in the scaffold legs did not differ for different load patterns. They concluded that the effect of influence surfaces on scaffold frame shoring subjected to load patterns is not significant.


In two papers, (Rosowsky et al. 1994a, 1994b), Rosowsky and his associates reviewed research from Taiwan that had used containers of water, instead of sandbags, to simulate the weight of concrete. In this case, the water simulated the weight of a 130 mm thick slab and the falsework layout followed a uniform grid of 0.91 m by 0.76 m. Again, the aim of the research was to investigate the load effect of concrete placement patterns. Results, similar to Peng et al. (1997), demonstrated that the load distribution was non-uniform with a maximum shore load in the order of twice the calculated load.

Rosowsky et al (1997) suggested, for ultimate limit states design that a load factor of 1.4 was inadequate and possibly a factor in the order of 2.0 was warranted. They found there was a relatively high uncertainty associated with the resistance of the
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temporary structural system, despite the low degree of uncertainty in the actual concrete load (the coefficient of variation of concrete is in the order of 0.10).

In a later paper, Rosowsky et al (1997) analysed data on shore loads collected by Fattal (1983) and Huston et al (1995). The purpose of this paper was to develop a statistically based equivalent uniform design load.

Huston’s data related to two separate 200 mm thick slabs: a small pour with shores each supporting approximately 1.8 m² of slab; and a larger pour with each shore supporting approximately 3.2 m² of slab. The latter slab also incorporated a central band beam 410 mm thick and approximately 1.22 m wide. As with previous research, some shores carried more load than expected. Interestingly, when Rosowsky et al reviewed Fattal’s data, they dismissed a high reading from one shore and concluded that ACI 347 adequately predicted the load in all the other shores.

Rosowsky et al (1997) concluded that the ACI design load (the weight of formwork, concrete and a live load of 2.4 kPa) adequately estimated the maximum shore load. However, a magnification factor in the order of 2.0 might be appropriate to account for the spatial variability among groups of shores in a common pour area. As well, they found that the ACI 347 design loads became more conservative as the slab area increased. They suggested that the design load might be a function of pour area, slab thickness, formwork arrangement and concrete placement procedures.

Kamala et al (1996)

At the same time Peng, Rosowsky and others were measuring loads in formwork shores, Kamala et al (1996) measured the load in props forming the legs of a flying table used on a multistorey building. The prop loads, measured immediately after pouring, correlated well with the design loads, except for two (out of nine) that varied by +28% and −12%. The authors concluded that differential tightening of props just before the pour could induce additional loads in props in the order of 12 kN.
Ikäheimonen (1997) set out with the objective of measuring the load in formwork shores and then drawing up design methods in accordance with the method of partial factors. He measured the loads in formwork shores on nine construction sites: four bridges, four blocks of flats and a courtyard deck. Significantly, he measured the load in shores supporting concrete from 150 mm to 1310 mm thick. He compared the actual loads with the loads predicted by a “beam method” and a “simplified method”. The beam method attempts to accurately estimate the load in a support by taking into account the reactions of joists and stringers, splice locations, and lengths. The simplified method was the same as described by Hurd (1995).

Interestingly, his conclusions included a number of points.

- When concrete was delivered by pump, the variable loads had no significant effect on shore loads.
- When concrete was delivered in skips, there were short-term dynamic load effects in the order of 5 to 30%.
- Apart from shores affected by dynamic loads after the slab was up to full height, the maximum shore loads occurred when concreting was finished and the variable loads removed. This means when calculating shore loads only the weight of formwork and concrete need to be considered.
- The relative shore loads (measured loads/calculated loads) varied between 0.15 and 1.5. This implies that the way the soffit formwork, joists and stringers distribute the load to the shores has a critical effect on the magnitudes of the relative shore loads.
- It is a common conclusion in previous works, which measure shore loads, that the large variations in relative shore loads are due to the initial load on the shores, i.e. the load due to soffit formwork and reinforcement. However, since Kamala et al. (1996) did not measure initial shore loads, their conclusions could not be confirmed.
- A theoretical analysis shows that when there is an initial gap between shore and stringer, there are large variations in the relative shore load, which should confirm the conclusions of previous works, referred to above.
Chapter 3  A Critique of Current Practice

- Measurements showed that large initial shore loads need not at all times be followed by large relative total loads, nor should the reverse occur.
- In some cases, current design practice (in which the load consists of the weight of formwork and concrete plus a fixed variable load) may underestimate the shore load when the slab is very thick.
- The mean value of the weight of plain concrete came to 22.8 kN/m$^3$.
- The two methods of calculating shore loads, the “beam method” and simplified method, were largely comparable.
- In designing formwork beams and calculating the support reactions for beams acted upon by point loads, in most cases, a uniformly distributed load can replace point loads.

Ikäheimonen measured, calculated and plotted the relative shore loads in a diagram that is reproduced as Figure 3.1. The relative shore load is the measured shore load divided by the calculated shore load, based on weight of the formwork and concrete. He found a best-fit curve for the maximum relative shore load is defined by the equation:

$$S_{\text{max}} = a(G_t + G_c) + b, \text{ in kN/m}^2$$  \hspace{1cm} (3.7)$$

In this Equation, $a$ is a factor equal to 1.3, $G_t$ and $G_c$ are the weights of the formwork and concrete, respectively, and $b$ is a constant derived for the purposes of curve fitting, which is equal to 0.75 kN/m$^2$. Thus:

$$S_{\text{max}} = 1.3(G_t + G_c) + 0.75, \text{ in kN/m}^2$$  \hspace{1cm} (3.8)$$

He also combined the data from his study with previous research and replotted the relative shore loads. That diagram is reproduced as Figure 3.2. In this case, the best-fit curve for the maximum relative shore load was obtained when $a = 1.25$ and $b = 2.5$, thus:

$$S_{\text{max}} = 1.25(G_t + G_c) + 2.5, \text{ in kN/m}^2$$  \hspace{1cm} (3.9)$$
Chapter 3  A Critique of Current Practice

Figure 3.1: Relative shore loads measured by Ikäheimonen (1997)

Figure 3.2: Relative shore loads from various studies (Ikäheimonen 1997)

Kothekar et al (1998)

More recently, Kothekar et al (1998) revisited shore load measurements collected by Agarwal and Gardner (1974), Noble (1975), Fattal (1983), and Huston et al (1995) and compared the actual loads to the design loads in ACI 347, ANSI A10.9 (American National Standards Institute 1995) and ASCE (1996). The provisions of ANSI A10.9 are similar to ACI 347 except a load factor of 1.3 increases all construction loads. The authors commented that they were not clear if this reference
was in permissible stress or limit state format. The ASCE reference is a LFRD
document and requires dead and live load factors of 1.2 and 1.6, respectively.
Kothekar et al (1998) concluded that all three references adequately estimated the
maximum shore load measured, with ACI 347 being the least conservative.

3.5 Discussion

After reviewing each reference, the important aspects in formwork shore design
become clear, namely:

- The design load

- The method of calculating the axial load in a shore.

- The method of determining the shore capacity, specifically issues such
  as: the merits of calculation versus testing; how imperfections peculiar to
  formwork are taken into account and the factor of safety appropriate for
  formwork.

Accordingly, these aspects provide a framework for reviewing the methods in AS
3610.

Design Load

Prior to concrete placement, formwork shores are required to support the weight of
the formwork and reinforcement. In addition, the shores must support the imposed
loads from construction activity and stacked materials on top of the formwork.

During concrete placement, formwork shores are required to support the weight of
the formwork, reinforcement and concrete, including the extra weight of any
concrete that is mounded in one place. In addition, the shores must resist any
dynamic effects of placing (dropping) the concrete and the imposed loads from
workmen and equipment engaged in placing, compacting and finishing the concrete.
Chapter 3  A Critique of Current Practice

After concrete placement, before the concrete becomes self-supporting and the formwork is removed, the formwork shores might be required to support additional imposed loads from construction activity and stacked materials that are placed on top of the slab.

The focus of this research is on the loads that can be expected during concrete placement. In all the national Standards reviewed, the design load combinations for formwork during concrete placement follow a similar format, namely:

\[
S_d = \gamma_f G_f + \gamma_c G_c + \gamma_q Q, \text{ in kN/m}^2
\]  
(3.10)

In Equation 3.10, \(\gamma_f\), \(\gamma_c\) and \(\gamma_q\) are the partial load factors for the respective actions. For permissible stress design, \(\gamma_f = \gamma_c = \gamma_q = 1.0\).

Generally, the weights (\(G_f\) and \(G_c\)) can be accurately estimated. Usually, the weight of formwork \(G_f\) is much less than the weight of concrete \(G_c\). Recommended values for the weight of concrete vary from 24 to 26 kN/m\(^3\) depending on the reference.

The value of the imposed actions \(Q\) varied from 1.0 kN/m\(^2\) in AS 3610 to a maximum of 3.0 kN/m\(^2\) in SI 904. Measurements show that, during concrete placement, the actual load in formwork shores does not exceed the load arising from weight of \(s\) alone. Although the dynamic effects of placing concrete vary from 5 to 30\% of the load in the formwork shore, they can be safely neglected unless the concrete has already reached its full depth. Similarly, the load from stacked materials only affects the load in formwork shores after concrete placement, when an imposed action of 2.4 kN/m\(^2\) might be appropriate.

The most significant difference between the design loads in AS 3610 and other limit states Standards (SII 1998 and ECS 2000) arises from AS 3610’s relatively low value of 1.25 for the partial load factor for the weight of concrete \(G_c\). Compare this to the value of partial load factors of 1.4 and 1.5 in SI 904 and prEN 12812. Furthermore, a partial load factor of 1.25 appears at odds with thinking that a partial load factor of 1.4 was inadequate and possibly a factor in the order of 2.0 was warranted (Rosowsky et al. 1994).
Figures 3.3 and 3.4 demonstrate that the shore design load is sensitive to the value of the partial load factor for the weight of concrete. The graph in Figure 3.3 is a plot of the permissible stress load combinations for AS 1509, ACI 347 and BS 5975 for a range of slab concrete thickness. In permissible stress design, the partial load factor is unity and it is assumed that the weight of the formwork is 0.5 kN/m$^2$ and both the weight of the concrete and value of imposed action is that recommended in the respective reference.

Figure 3.3 demonstrates a general agreement between AS 1509, ACI 347 and BS 5975. Thus, it appears that the minor differences between the value of concrete density and imposed actions nominated in these references is not significant. In addition, Figure 3.3 confirms suggestions in the literature that current design practice might underestimate the maximum shore load when the slab is very thick.

Figure 3.4 is a graph of the limit states load combinations for AS 3610, SI 904 and prEN 12812 for a range of slab concrete thickness. As before, it is assumed that the formwork weighs 0.5 kN/m$^2$ and the weight of the concrete and value of imposed action is that recommended in the respective references. For simplification, where appropriate the limits on the application of imposed actions for mounding have been neglected.

Figure 3.4 demonstrates, that during concrete placement:

- the design loads specified in SI 904 and prEN 12812 are similar; and
- for thicker concrete, there is an increasing disparity between the design loads specified in AS 3610 and other national limit states Standards.
Figure 3.3: A comparison of permissible stress formwork design loads

Figure 3.4: A comparison of limit states formwork design loads
Furthermore, Figure 3.4 supports the previous propositions that the design load is sensitive to the value of the partial load factor for concrete (especially for thicker concrete), and that any minor differences between the nominal value of concrete density and imposed actions is not significant.

Interestingly, Figure 3.5 shows a plot of Equation 3.11 for various values of $\gamma$:

$$S = \gamma(G_t + G_c + Q_v + M), \text{ in kN/m}^2$$ \hspace{1cm} (3.11)

![Graph showing design action effect vs. concrete thickness for different load factors](image)

**Figure 3.5: A comparison of amplified permissible stress formwork design loads**

Using BS 5975 design load as an example, this demonstrates that by increasing the "hypothetical" partial load factor (unity) for permissible stress methods, the resulting design load more closely approximates the maximum shore load measured by Ikaheimonen (1997). In fact, a close correlation is achieved when permissible stress design loads are amplified by 17% ($\gamma = 1.17$).
Chapter 3  A Critique of Current Practice

Axial Load

The concept of a shore having a tributary area simplifies calculating axial loads. According to Hurd (1995), for designs complying with ACI 347, it is sufficiently accurate to adopt the simple method; effectively ignoring the effects of continuity. Similarly, the simple method is adopted in guidance on AS 3610 (SA 1996a; and Gardiner 1989).

In contrast, BS 5975 requires the effects of beam continuity to be taken into account. When formwork shores support random length primary and secondary beams that are continuous over more than one span, BS 5975 recommends a continuity factor $\gamma = 1.1$. In practice, designers complying with BS 5975 tend to adopt a load factor of at least 1.1 in most situations. The effect of a continuity factor, $\gamma = 1.1$ can been seen in Figure 3.5.

Ilkäheimonen (1997) found the two methods (simple and detailed) were largely comparable. He thought this might be partly explained because, in practice, continuous formwork beams do not bear on all their supports. It is his proposition that before concrete placement small gaps might be present but close later under the weight of the concrete. If such gaps go unnoticed, the load distribution and action effects in formwork shores will differ from predictions. In addition, ignoring the differential stiffness amongst shores and their foundations might be another factor that influences the load distribution in shores (Liu and Chen 1987).

Shore Capacity

Significant economies arise when formwork components such as formwork shores can be reused. During their working life, formwork shores that are reused many times often sustain damage. The damage might occur: accidentally; from overload; and due to normal wear and tear during dismantling, handling, transportation and reerection. As it would be uneconomical to discard or continually repair shores with minor damage, it is often accepted practice to reuse them with imperfections greater than permitted in normal steel construction.
Typical imperfections affecting shores include: eccentricity in the application of the load and resulting reaction; out-of-plumb erection; out-of-straightness; joint eccentricities and angular changes. The presence of additional imperfections has a detrimental effect on the strength of the formwork.

In general, there appears to be two distinct approaches.

1. Explicitly take account of imperfections, as do AS 3610 and BS 5975; or

2. Attempt to minimise their occurrence and specify larger factors of safety to deal with any remaining unavoidable or unintentional imperfections. This is the approach followed in AS 1509, ACI 347 and SI 904.

The draft European Standard prEN 12812, has a “bet each way” requiring imperfections to be dealt with explicitly and increases the factor of safety for formwork in general.

In AS 1509 and ACI 347, shores are required to conform to the same material tolerances as permanent construction and eccentric loading is to be avoided. Although, both Standards highlight the need to make adequate allowance for unsymmetrical or eccentric loading due to placement procedures, the capacity of formwork shores is determined by testing concentrically loaded formwork shores. For AS 1509, the factor of safety against collapse is 2.5 or 3.0. ACI 347 follows SSFI recommendations for a factor of safety of 2.5, but further compensates for minor imperfections by recommending reduced allowable loads for reused formwork shores.

In contrast, AS 3610, BS 5975 and prEN 12812 recognise the presence of both eccentric loading and additional imperfections. Furthermore, testing or calculations must explicitly take into account such effects.

When testing is used to determine capacity, out-of-straight shores are selected and erected out-of-plumb and eccentrically loaded. The ultimate capacity of shores tested this way is lower than if tested concentrically and a relaxation in the factor of
safety might be expected. This explains the lower factor of safety of 2.0 specified in BS 5975 compared to 2.5 to 3.0 for AS 1509 and ACI 347. In AS 3610, the factor of safety against collapse is given by the product of a limit state conversion factor (LSCF) and a sampling factor ($k_s$). For example, a safety factor of 1.92 results by multiplying a LSCF = 1.37 (Clause CA4.4.4 of the Commentary to AS 3610) and $k_s = 1.4$ (AS 3610 Table A1 for a COV of 0.10 over 3 tests). For more tests, the value of the sampling factor reduces resulting in yet lower safety factors.

Although, the effects of imperfections are more easily and accurately accounted for by testing, AS 3610 (like BS 5975 and prEN 12812) permits the capacity of shores to be determined by calculation. Given the expense of testing and wide availability of computer software capable of non-linear analysis, there is an increasing tendency for manufacturers not to test their products.

Unfortunately, the limit states provisions of AS 3610 and prEN 12812 have been criticised as overly complex (SA 1997, Pallett 1994 and Pallet 2000). There is agreement that the effects of eccentricities and additional imperfections need to be taken into account, but there is wide disagreement of how this is best handled. There is support for a simple approach, such as in AS 1509 or SI 904. Where minor imperfections in reused components (other than shores) are accounted for by a simple factor of safety. For example, AS 1509 limits the maximum permissible stress to 85% of that given in the applicable design code. Similarly, SI 904 specifies a higher value for the partial material resistance factor for formwork than that for normal steel construction.

Whatever the approach, the crux of the matter is whether the formwork is structurally sound. Indeed, Fattal (1983) acknowledged the in some cases ACI 347 underestimated the shore load, but thought any minor overloading was tolerable given the factor of safety of 2.5 for shores in ACI 347. Was he right?

How does AS 3610 compare? The design loads are less conservative than other limit states Standards. Is the treatment of eccentric loading and imperfections sufficiently conservative? Or, should an additional formwork specific partial resistance factor, similar to the $\gamma_T = 1.15$ from prEN 12812, be introduced?
3.6 Conclusions

This chapter compares the formwork shore design methods in AS 3610 with past Australian, current international formwork Standards and Codes of Practice, and the results of recent research.

Formwork shore design loads from each reference were compared with predictions of the maximum load in formwork shores based on actual measurements. A simple comparison of the formwork shore design loads from past Australian and current international formwork Standards/Codes of Practice demonstrates that they are consistent. When compared with actual measurements of the load in shores, the comparison supports suggestions in the literature that, as concrete thickness increases, permissible stress methods increasingly underestimate the maximum load in shores. A similar comparison of limit states design loads suggests that, especially as concrete thicknesses increase, current Australian limit states practice appears less conservative than international practice.

The cause of this disparity is unclear, but unlikely to be the result of concrete placement patterns, initial preloading or neglecting the effects of continuous bearers. More likely, the presence of small initial gaps between shores and the bearers they support, combined with differences in shore and foundation stiffness cause a redistribution of the load.

Simple comparisons suggest that small variations in the density of concrete and imposed actions do not appear to have a significant effect on the design action. However, the value of partial load factor for the weight of the concrete is significant.

The strength of shores is influenced by aspects of formwork construction not present in normal steel construction. Consequently, shores may not conform to the material, fabrication and erection tolerances in the relevant material Standards. Neglecting these defects might result an over-estimation of shore capacity. Formwork Standards address this problem in different ways. Some simply increase the factor of safety for shores, while others (such as AS 3610) specify relaxed tolerances and
require that they be dealt with explicitly. This makes meaningful comparison difficult.

Any overloading of formwork shores might be tolerable if compensated given a sufficiently high factor of safety against collapse. However, it is unclear whether any shortcomings in the AS 3610 design loads are compensated by conservative estimates of shore capacity.
Chapter 4  Reliability

4.1 Introduction

In Chapter 3 the provisions of AS 3610 – 1995 Formwork for Concrete (SA 1995) were compared with past Australian practice, current international practice and recent research. Attention was confined to the rules for designing formwork shores that support formwork during concrete placement.

The comparison supported concerns in the literature that, as concrete thickness increases, current practice underestimates the maximum load in formwork shores. In particular, the limit states design loads in AS 3610 appear less conservative than those in other national limit states Standards. These observations might be somewhat academic if any overloading of formwork shores were compensated by a sufficiently high factor of safety against collapse. For AS 3610, this is unproven.

The proposition of this thesis is that, during concrete placement, formwork supported by steel shores designed to the limit states permitted in AS 3610 is unsatisfactory. To accurately compare the adequacy of current practice it is necessary to take account of both the action effects and resistance of formwork shores. Thus, to test the hypothesis, first-order probabilistic techniques were chosen. This chapter introduces the concept of reliability and calibration to achieve a target reliability index. Furthermore, it introduces and justifies the selection of target indices, as well as the action effect and resistance data used in the analysis.

4.2 Concept of Reliability

A structure is satisfactory if it is fit for purpose and consistently performs as intended. In other words, it is “structurally reliable”. Given sufficient data on structural failures, it would be possible to verify the reliability of a structure using purely probabilistic techniques. Figure 4.1 expresses the relationship between action effect $S$ and resistance $R$, in terms of probability density functions.
Figure 4.1: Probability distributions for design action effects and design resistance

The shaded area in Figure 4.1 illustrates the probability of failure, which can be expressed as:

$$p_f = P[(R - S) < 0]$$  \hspace{1cm} (4.1)

Figure 4.2: Definition of reliability index

An alternative representation of the probability of failure is:
Chapter 4  Reliability

\[ p_F = P\left[ \ln\left( \frac{R}{S} \right) < 0 \right] \]  

(4.2)

The relationship in Equation 4.2 is presented in Figure 4.2, where \( \ln(R/S)_m \) and \( \sigma_{\ln(R/S)} \) are the mean and standard deviation of the natural logarithm of the ratio \( (R/S) \).

Rarely is there sufficient data to know the probability distributions of \( R \) and \( S \). Normally, only the means \( (R_m \) and \( S_m \)) and the standard deviations \( (\sigma_R \) and \( \sigma_S \)) are known. However, by using first order probabilistic methods, a relative measure of the reliability (safety) of a structure can be calculated, such that:

\[ \beta = \frac{\ln\left( \frac{R_m}{S_m} \right)}{\sqrt{V_R^2 + V_S^2}} \]  

(4.3)

In Equation 4.3, \( \beta \) is called the reliability index, and \( V_R \) and \( V_S \) are the coefficients of variation of \( R \) and \( S \), respectively.

4.3 Calibration

Commonly, limit states design rules are developed using first-order probabilistic techniques or similar methods in a process called “calibration”. First, a target reliability index is chosen after considering the reliability of structural designs using existing methods, often permissible stress methods. Then, new design rules are calibrated to achieve the target index. This approach was first used by Ravindra and Galambos (1978) to develop LRFD load and resistance factors and more recently to determine resistance factors for use in design Standard AS/NZS 4600:1996 Cold-formed steel structures (SA 1996b).

Target Reliability Index

Normally, the responsibility of choosing the appropriate degree of reliability rests with Standards Committees. To provide some guidance on what might be an
appropriate degree of reliability for steel formwork shores, the target reliability indices suggested in the literature and those adopted in interacting Standards, such as AS 4100 and AS 4600, are considered.

There are few references in the literature that propose target reliability indices for formwork. One paper (Kennedy et al. 1991) describes the calibration of load and resistance factors for wooden wall formwork. Kennedy et al propose a target $\beta = 3.0$ as the basis for the calibration of load factors for the ACI and Gardner concrete pressure equations when used in conjunction with the Canadian Wood Standard CAN-CSA-086.1-M89 (CSA 1989).

In another less closely related paper (El-Shahhat et al. 1994), the authors adopted a target reliability $\beta$ in the range 2.5 to 3.0 as the basis for developing load combinations for buildings during construction.

In Australia, the resistance design of steel formwork shores should comply with either: AS 4100:1998 Steel Structures (SA 1998a) or AS 4600:1996 Cold-formed Steel Structures (SA 1996b).

Generally, a target value for $\beta$ is set to give levels of safety consistent with past practice. Accordingly, the design rules in AS 4100 are intended to achieve a target reliability index $\beta = 3.5$, which was set after considering a statistical analysis of steel beams and columns designed to AS 1250 (Pham et al. 1986).

In contrast, the design rules for members in AS 4600 are based on a lower target index, $\beta = 2.5$ (SA 1998b). In this case, a lower target index is justified as being the same target index set for the AISC LRFD Specification (AISC 1993).

4.4 Action Effect and Resistance Data

In an ideal world, data for statistical analysis would be collected during full-scale destructive testing of formwork. For formwork, the task of full-scale testing presents serious practical difficulties that are amplified to impractical proportions by:
Chapter 4 Reliability

- the large range of different formwork systems available;
- the range of possible combinations of height and span of each system;
- the range of load configurations (concrete thickness); and
- the inherent danger in the collapse of such large and heavily loaded structures.

Examples of the practical difficulties of full-scale testing can be found in Rosowsky et al. (1994) and Peng et al. (1997). In each case, full-scale formwork structures were tested using containers of water and sand bags in lieu of concrete, respectively.

An alternative to testing, is to source data from the literature.

**Action Effect Data**

Ikäheimonen (1997) measured the load in 66 formwork shores before, during and after concrete placement. The shores were located on nine sites (four bridges and five apartment buildings) and supported concrete ranging from 0.15 m up to 1.31 m thick. His data is reproduced in Appendix A.

Statistically, the independence of this data might be questioned. As well, all measurements occurred at serviceability limit state and not ultimate limit state. However, in support:

- it is the most comprehensive single set of data available;
- it is the only source of load data for shores supporting concrete thicker than 0.30 m;
- Figure 3.2 demonstrates that it achieved a reasonably good, if not slightly conservative fit with the work of others; and
Chapter 4  Reliability

- the mean of the data is very close to 1.0, which would be expected for equilibrium to be maintained.

Ikäheimonen presented the measurements in terms of the relative shore load, being the ratio of the actual load to the estimated or nominal load. To do this he used two methods to estimate the load in the shores.

(a) A precise method that took into account: the continuity of secondary and primary beams; the exact weight of formwork, reinforcement and concrete; as well as deductions for voids and pipes.

(b) A simple method that did not account for continuity and assumed that the weight of formwork was 0.4 kN/m² and the weight of the concrete is based on a density of 2400 kg/m³, respectively.

Assuming a normal distribution, the ratio of mean to the nominal action effect ($\lambda_S = S_m/S_n$) and the coefficient of variation ($V_S$) is: $\lambda_S = 0.94$ and $V_S = 0.31$ for the precise method; and $\lambda_S = 1.03$ and $V_S = 0.31$ for the simple method. In practice, it is unlikely that designers would use a method as rigorous as the precise method; therefore, median values might be appropriate for this analysis.

Resistance Data

Traditionally, the capacity of formwork shores was established by component testing. Thus, it could be expected that there is a substantial body of data on the resistance of steel formwork shores. Unfortunately, due to the proprietary nature of this information, it is considered commercially sensitive and remains outside the public domain. In fact, there is little research reported in the literature that specifically measures the resistance of steel formwork shores.

Of a more general nature is work by Pham and Bridge (1985) who analysed experimental data on steel columns collected by Fukumoto and Itoh (1983). Their analysis is useful because it proposes a relationship between the mean and nominal design resistance of steel columns for both permissible stress and limit state design
methods, specifically, the permissible stress methods in AS 1250 (SAA 1981) and the limit states methods later used in AS 4100 (SA 1998a). This data is relevant because, in Australia, steel formwork shores can be designed using either: the permissible stress methods in AS 3610 and AS 1250; or the limit states methods in AS 3610 in conjunction with AS 4100.

Pham and Bridge proposed the following relationships between the mean and nominal resistance \( (\lambda_R = R_m/R_n) \), and the coefficient of variation \( (V_R) \) of steel columns.

For permissible stress methods to AS 1250, \( \lambda_R \) varies from 1.98 to 2.20 and \( V_R \) varies from 0.17 to 0.16.

For limit states methods to AS 4100, \( \lambda_R \) varies from 1.17 to 1.25 and \( V_R \) varies from 0.13 to 0.16.

At this point, it is important to recognise that the values of \( \lambda_R \) and \( V_R \) apply to steel columns in "new" condition, designed and constructed to the standard of normal permanent structures. Where the strength of shores is degraded by defects and construction techniques not permitted in normal steel construction, this resistance data is not appropriate.

To address this anomaly, Chapter 5 investigates the relative reliability of design loads for formwork shores in "new" condition. Chapters 6 and 7 then address the relaxed tolerances and construction techniques peculiar to formwork.

4.5 Conclusions

First order probabilistic techniques have been chosen to test the reliability of formwork limit state design rules. A review of the literature suggests that an appropriate target reliability index \( (\beta) \) for steel formwork shores should fall in the range of 2.5 to 3.5.
Chapter 4  Reliability

An analysis will be based on statistical data collected from actual measurements of shore loads and experimental data on the resistance of steel columns. For the purposes of this research median values for actions effect and resistance data have been chosen, specifically:

- $\lambda_S = 0.99$ and $V_S = 0.31$, because, in practice, it is unlikely that designers would use more rigorous methods; and
- for permissible stress design to AS 1250, $\lambda_R = 2.09$ and $V_R = 0.165$; and for limit states design to AS 4100, $\lambda_R = 1.21$ and $V_R = 0.145$.

The details and results of that analysis are set out in Chapter 5.

Significantly, the results of the analysis are only appropriate for steel shores in new condition that form part of formwork designed and constructed to a standard equivalent to normal permanent construction. Chapters 6 and 7 investigate the reliability of shores when this is not the case.
Chapter 5  The Reliability of AS 3610 Design Loads

5.1  Introduction

In Chapter 4, simple first-order probabilistic techniques, action effect and resistance data were selected to test the adequacy of formwork shores designed to the limit states permitted in AS 3610.

This chapter establishes the relative reliability of steel formwork shores designed using limit states design loads in AS 3610 – 1995 Formwork for Concrete (SA 1995). In addition, proposed new design loads are calibrated to achieve an equivalent reliability with past practice.

This is achieved by first developing a formwork specific expression for reliability index $\beta$. This expression is then used to compare the relative reliability of the design load combinations from AS 3610 to past Australian Standard AS 1509 – 1974 (SAA 1974) and current international Standards, specifically:

- American Standard ACI 347:2001 Guide to Formwork for Concrete (ACI 2001);

- British Standard BS 5975:1996 Code of practice for Falsework (BS 1996);

- Israeli Standard SI 904: 1998 Part 1: Formwork for Concrete: Principles (SII 1998); and

- the draft European Standard prEN 12812: 2000 Falsework – Performance requirements and general design (ECS 2000).

The results reported are valid for steel shores conforming to the requirements and tolerances specified in AS 4100 – 1998 Steel structures (SA 1998a).
5.2 Formwork Reliability Index $\beta$

From Chapter 4, a relative measure of the reliability (safety) of a structure can be calculated, such that:

$$
\beta = \frac{\ln \left( \frac{R_m}{S_m} \right)}{\sqrt{V_R^2 + V_S^2}}
$$

(4.3)

In Equation 4.3, $\beta$ is called the reliability index, and $V_R$ and $V_S$ are the coefficients of variation of $R$ and $S$, respectively.

In limit states design, separate partial factors for actions and resistance deal with the uncertainties and consequence of failure. Usually, this is expressed as:

$$
\gamma S_n \leq \phi R_n
$$

(5.1)

In Equation 5.1, $S_n$ is the nominal action effect, $R_n$ is the nominal resistance, and $\gamma$ and $\phi$ are partial factors. The values for each variable are commonly specified in the applicable structural Standard.

It can be shown (Ravindra and Galambos 1978) that approximate values of $\gamma$ and $\phi$ are given by the expressions:

$$
\gamma = \lambda_S e^{\alpha \beta V_S}
$$

(5.2)

and

$$
\phi = \lambda_R e^{-\alpha \beta V_R}
$$

(5.3)

where $\alpha$ is a numerical constant equal to 0.55, $\lambda_S = (S_m/S_n)$ and $\lambda_R = (R_m/R_n)$. 

- 55 -
Thus, knowing the mean and coefficient of variation of the action effect and resistance, a reliability index $\beta$ can be determined from Equation 5.4. Alternatively, given a target reliability index appropriate values for $\gamma$ and $\phi$ can be derived.

$$\beta = \ln \left[ \frac{\lambda y S_m}{\phi S_m} \right] \sqrt{\frac{\nu^2}{V^2_R + V^2_S}}$$  \hspace{1cm} (5.4)

**Formwork action effects**

It is proposed to base the first order probabilistic analysis on the action data sourced from Ikäheimonen (1997), which is reproduced in Appendix A. Ikäheimonen found that unless shores were loaded after the concrete had reached full height, only the combined weight of formwork, reinforcement and concrete need to be considered. Thus, the relationship between mean and nominal action effects can be expressed as:

$$S_m = \lambda_s (G_f + G_c)$$  \hspace{1cm} (5.5)

where $G_f$ is the weight of the formwork and $G_c$ is the weight of the reinforcement and concrete.

This differs slightly from the general format used in formwork Standards, namely:

$$\gamma S_u = \gamma_f G_f + \gamma_c G_c + \gamma_q Q$$  \hspace{1cm} (5.6)

In Equation 5.6: $\gamma_f$, $\gamma_c$ and $\gamma_q$ are load factors for formwork, concrete and imposed actions; $G_f$ is the weight of the formwork; $G_c$ is the weight of the concrete including an allowance for reinforcement; and $Q$ is an imposed action for the weight of workmen, equipment and the effects of concrete discharge methods (dynamic and mounding). For permissible stress design, the values of the load factors are usually 1.0.
Chapter 5  The Reliability of AS 3610

Generically, the reliability index \( \beta \) for formwork shores can be found by substituting the appropriate values of \( \lambda_R, \lambda_S, V_S, V_R, \) and Equations 5.5 and 5.6 into Equation 5.4, which gives:

\[
\beta = \sqrt{\frac{\ln \left( \frac{\lambda_R \left( \gamma_f G_f + \gamma_c G_c + \gamma_q Q \right)}{\phi \lambda_S (G_f + G_c)} \right)}{V_R^2 + V_S^2}} \tag{5.7}
\]

where \( \phi \) is the ultimate limit state capacity reduction factor for steel columns specified in AS 4100 (\( \phi = 0.90 \)). For permissible stress methods, \( \phi \) defaults to unity.

5.3 Design load combinations

For convenience, the design load combinations reported in Chapters 2 and 3 are reproduced in Table 5.1.

<table>
<thead>
<tr>
<th>Standard</th>
<th>Design load combination</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 3610'</td>
<td>( S^* = 1.25G_f + 1.25G_c + 1.5Q_{uv} + 1.5M, \text{kN/m}^2 ) (2.2)</td>
</tr>
<tr>
<td>AS 3610'</td>
<td>( S^* = 1.25G_f + 1.25G_c + Q_o, \text{kN/m}^2 ) (2.3)</td>
</tr>
<tr>
<td>AS 1509</td>
<td>( S = G_f + G_c + Q_v + M, \text{kN/m}^2 ) (3.1)</td>
</tr>
<tr>
<td>ACI 347</td>
<td>( S = G_f + G_c + Q_v \geq 4.8, \text{in kN/m}^2 ) (3.2)</td>
</tr>
<tr>
<td>BS 5975</td>
<td>( S = G_f + G_c + Q_v + M, \text{in kN/m}^2 ) (3.3)</td>
</tr>
<tr>
<td>SI 904'(^1)</td>
<td>( S^* = 1.4(G_f + G_c) + 1.6Q_{uv}, \text{in kN/m}^2 ) (3.4)</td>
</tr>
<tr>
<td>prEN 12812*</td>
<td>( S^* = 1.35G_f + 1.5G_c + 1.5Q_v + 1.5Q_o, \text{in kN/m}^2 ) (3.6)</td>
</tr>
</tbody>
</table>

* limit states load combinations  
1 by observation Equation 3.4 governs and Equation 3.5 can be neglected

To compare the relative reliability of the different design load combinations over a range of concrete thicknesses certain assumptions are made.
Chapter 5  The Reliability of AS 3610

- The weight of formwork $G_r$ is constant at 0.5 kN/m$^2$;

- The value for the weight of concrete $G_c$ is as recommended in each reference;

- The value for imposed actions $Q_i$ is as recommended in each reference;

- Limitations on the extent of imposed actions for concrete mounding $Q_c$ are ignored; and

- No stacked materials are present during concrete placement, i.e. $M = 0$ kN/m$^2$.

Consequently, by substitution, the design load combinations from Table 5.1 simplify to the equations in Table 5.2, where $t$ represents the thickness of concrete in metres.

### Table 5.2: Simplified design load combinations

<table>
<thead>
<tr>
<th>Standard</th>
<th>Simplified design load combination</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 3610'</td>
<td>$S^* = 1.25(0.5) + 1.25(24.5)t + 1.5(1.0) + 1.5(0)$, in kN/m$^2$</td>
</tr>
<tr>
<td>AS 3610'</td>
<td>$S^* = 1.25(0.5) + 1.25(24.5)t + 1.0(3.0)$, in kN/m$^2$</td>
</tr>
<tr>
<td>AS 1509</td>
<td>$S = 1.0(0.5) + 1.0(24.5)t + 1.0(1.9)$, in kN/m$^2$</td>
</tr>
<tr>
<td>ACI 347</td>
<td>$S = 1.0(0.5) + 1.0(23.5)t + 1.0(2.4) \geq 4.8$, in kN/m$^2$</td>
</tr>
<tr>
<td>BS 5975</td>
<td>$S = 1.0(0.5) + 1.0(24.5)t + 1.0(1.5)$, in kN/m$^2$</td>
</tr>
<tr>
<td>SI 904'</td>
<td>$S^* = 1.4(0.5) + 1.4(26)t + 1.6(3.0)$, in kN/m$^2$</td>
</tr>
<tr>
<td>prEN 12812'</td>
<td>$S^* = 1.35(0.5) + 1.5(24.5)t + 1.5(0.75) + 1.5(0.75)$, in kN/m$^2$</td>
</tr>
</tbody>
</table>

* limit states load combinations
5.4 Action and Resistance Data

In Chapter 4, it was assumed that the ratio of the mean to nominal action effect $\lambda_S = 0.99$ and the coefficient of variation $V_S = 0.31$. Similarly, the values selected for resistance are:

For permissible stress methods, median values of $\lambda_R = 2.09$ and $V_R = 0.165$; and

For limit states methods, $\lambda_R = 1.21$ and $V_R = 0.145$.

5.5 Relative Reliability of AS 3610

An appreciation of the relative reliability of AS 3610 is attained by reviewing Figures 5.1 to 5.3. Each Figure plots the relative reliability index $\beta$ of steel formwork shores designed using the design load combinations from different formwork Standards and the relevant structural steel Standard, AS 1250 or AS 4100. The reliability index $\beta$ is calculated for a range of concrete thickness from 0.10 m to 1.30 m thick.

Figure 5.1 compares the reliability of past Australian and current international practice; specifically, the limit states design load combinations in SI 904 and prEN 12812 with the equivalent permissible stress load combinations in AS 1509, ACI 347 and BS 5975.

Figure 5.2 compares the reliability of the current Australian limits states practice with past Australian and current international permissible stress practice; specifically, the limit states design load combinations in AS 3610 with the equivalent permissible stress load combinations in AS 1509, ACI 347 and BS 5975.

Figure 5.3 compares the reliability of the current Australian and international limit states practice; specifically, the design load combinations in AS 3610, SI 904 and prEN 12812.
Figure 5.1: A comparison of the reliability of past Australian and current international design practice.

Figure 5.2: A comparison of the reliability of current Australian limit states practice with past Australian and current international permissible stress practice.
Figure 5.3: A comparison of the reliability of current Australian and international limits states practice.

5.6 Analysis of Results

Figure 5.1 demonstrates that the reliability index for past Australian and current international practice is consistent and varies from a maximum of 4.3 to a minimum of 2.3. In addition, it appears that design loads for shores supporting thinner concrete are more reliable, with reliability decreasing with increasing concrete thickness.

Figures 5.2 and 5.3 clearly demonstrate a disparity between the limit states design loads in AS 3610 and all other formwork Standards. For AS 3610, the reliability index is significantly lower and varies from approximately 3.4 to 1.8.

Calibration

In an attempt to calibrate current with past practice, Figure 5.4 plots two new design load combinations, namely:
\[ S^* = 1.3(1.25G_t + 1.25G_c + 1.5Q_{uv} + 1.5M) \text{ kN/m}^2 \]  

\[ S^* = 1.6(G_t + G_c) + 2.5 \text{ kN/m}^2 \]  

The equations reflect two different approaches. Equation 5.8, amplifies the existing AS 3610 design load (Equation 2.2) by a factor of 1.3. Equation 5.9 reflects that the actual load in shores can be accurately estimated considering only combined weight of the formwork and concrete plus a constant for curve fitting.

![Graph](image-url)  

**Figure 5.4:** Calibration of proposed new and amplified AS 3610 design loads to past Australian practice (AS 1509).

### 5.7 Conclusions

A first order probability method of comparing the relative reliability of steel formwork shores is presented. Using this technique over a range of concrete thickness, the reliability of past Australian and current international practice appears consistent and compares favourably with a target reliability index in the range of 2.5 to 3.5.
Chapter 5  The Reliability of AS 3610

Except for shores supporting thin concrete, the reliability of current Australian limit states methods appears significantly less than other formwork Standards and do not reach target indices.

Accordingly, new limit states design load combinations are calibrated to achieve a target reliability index compatible with past practice, values recommended in the literature and adopted in compatible Standards.

Importantly, any detrimental effects from reusing steel shores and differences in the quality of formwork practices and normal permanent construction have been neglected. Thus, the results of this analysis are only valid for steel shores conforming to the requirements and tolerances of AS 4100.
Chapter 6  The Influence of Additional Imperfections

6.1 Introduction

Chapters 4 and 5 investigated the relative reliability of the AS 3610 limit states design rules for formwork shores compared to past Australian and current international practice. The analysis was based on data for “new” steel columns that conform to the tolerances permitted in AS 4100 (SA 1998a) and neglected the degrading effects of additional imperfections permitted in AS 3610.

Under load, the effect of additional imperfections is to introduce additional bending stresses that reduce the axial capacity of the member. Designers neglecting these bending stresses might overestimate the member capacity. At present, there is little guidance in the literature or current design Standards as to how the effects of additional imperfections can be taken into account. Accordingly, this chapter investigates the influence of these imperfections and how they might be taken account of explicitly.

This is achieved by deriving new column strength curves that take account of the effects of additional out-of-straightness and eccentricity. Although not supported by test data, it is shown that, for a normal out-of-straightness, the new curves closely approximate the appropriate column curves in AS 4100.

In addition, the partial resistance factor $\gamma$ is introduced as a useful design concept. Using practical estimates for each type of imperfection, $\gamma$-curves are developed that demonstrate the influence of imperfections on the axial capacity of columns. The chapter also includes a design example dealing with an out-of-straight and eccentrically loaded formwork shore.
6.2 Initial Imperfections

Small imperfections are present in all steel members.

- During manufacture, the cooling of hot-rolled or welded sections leaves residual stresses;

- Steel members may be fabricated with an initial curvature or in-built eccentricity; and

- When erected, members may be out-of-position and/or out-of-plumb.

In compression members, the presence of imperfections degrades the axial capacity of the member. Thus, it is important that they are taken into account.

Method of Analysis

For practical reasons of robustness and telescoping, compact circular hollow (CHS) and square hollow (SHS) sections are the most common steel column sections used in scaffold, formwork, tilt-up panel construction, etc. Subsequently, the effects of local, distortional and flexural-torsional buckling can be safely neglected.

For a slender member subject to an axial load, deformations increase instability. In this case, a first-order elastic analysis that ignores geometrical and material non-linearity will underestimate the actions. Although a first-order plastic analysis takes into account material non-linearity, it is more appropriate for sway structures with small axial loads that are not common to formwork.

A second-order elastic analysis accounts for geometrical non-linearity but not material non-linearity. This approach is appropriate for members subject to primarily axial loads, as it provides a conservative estimate of ultimate load. Computer programs (ITS 1997) that are capable of a second-order elastic analysis are readily available. Alternatively, an approximation of second-order effects is possible by amplifying first-order moments. A much more accurate analysis is
possible using an advanced analysis (Clarke et al. 1992) but is not yet in general use.

As formwork designs are often carried out on-site, a simple generic approach using amplified first-order elastic analysis has merit.

**Perfectly Straight Member**

Consider a perfectly straight member, pin-ended, fixed at the base and braced from moving horizontally at the top, but free to move vertically as shown in Figure 6.1(a). It is assumed that the member is made of a linear elastic material with a constant cross-sectional area, \( A \).

![Diagram of perfectly straight compression member](image)

(a) Unloaded                  (b) Compressed                  (c) Deflected but stable

Figure 6.1: Perfectly straight slender compression member (Trahair and Bradford 1998)

If the member is very stocky, a concentric force \( N \) can be applied and increased until the material yields. The general yield load \( N_y \) (squash load) is:

\[
N_y = A f_y
\]  

(6.1)

where \( f_y \) is the yield stress of the material.
For slender members, buckling may occur before reaching the squash load. Trahair and Bradford (1998) explain that if the member is slender, a concentric force, $N$, can be applied at the top of the member and the member will remain straight under axial compression, as shown in Figure 6.1(b). If a small lateral force is applied at mid-height, the member will deflect a small amount, as shown in Figure 6.1(c). When the lateral force is removed the member returns to its original straight position, it is stable. If the force $N$ is increased, a condition of instability is reached when a small lateral force will produce a deflection that does not disappear when the lateral force is removed. In this case, the force $N$ has reached the critical load (Euler load) or the elastic buckling load, $N_0$. The elastic buckling load is (Timoshenko and Gere 1961):

$$N_0 = \frac{\pi^2 EI}{L^2} \quad (6.2)$$

The elastic buckling load expressed in terms of the slenderness ratio $L/r$ is:

$$N_0 = \frac{\pi^2 EA}{(L/r)^2} \quad (6.3)$$

in which the radius of gyration $r = \sqrt{I/A}$.

**Initially Out-of-Straight Member**

In a compression member, the effect of an initial out-of-straightness is to introduce additional bending stresses which reduce the axial capacity of the member. This is commonly called the P-δ effect.

**AS 4100 – 1998**

Within specified tolerances, steel design standards implicitly take account of these initial imperfections, thereby negating the need for designers to consider their effects.
Chapter 6  Additional Imperfections

For compact sections, the nominal capacity of structural steel columns, according to AS 4100, is:

\[ N_c = \alpha_C N_S \leq N_S \]  

(6.4)

where

\[ N_S = \text{the nominal section capacity, given by:} \]

\[ N_S = A_n f_y \]  

(6.5)

in which \( A_n \) is the net area of the cross section.

\( \alpha_C = \text{the member slenderness reduction factor, given by:} \)

\[ \alpha_C = \xi \left\{ 1 - \left[ 1 - \left( \frac{90}{\xi \lambda} \right)^{1/2} \right] \right\} \]  

(6.6)

in which

\[ \xi = \left( \frac{\lambda}{90} \right)^2 + \frac{1 + \eta}{2(\lambda/90)^2} \]  

(6.7)

\[ \eta = 0.00326(\lambda - 13.5) \geq 0 \]  

(6.8)

\[ \lambda = \lambda_n + \alpha_a \alpha_b \]  

(6.9)

\[ \lambda_n = \left( \frac{L}{r} \right) \left( \frac{f_y}{250} \right) \]  

(6.10)

\[ \alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3 \lambda_n + 2050} \]  

(6.11)

The term \( \alpha_b \) is a member section constant that provides for different levels of residual stress for different section shapes and manufacturing processes. Its value
Chapter 6  Additional Imperfections

varies from -1.0 for low residual stress levels in stress relieved RHS and CHS sections to +1.0 for high residual stress levels in hot-rolled UB and UC sections and welded H and I sections with flange thickness over 40 mm.

Figure 6.2 plots the column strength curves for $N_Y$, $N_o$ and $N_c$ for $\alpha_b = -1.0$, 0 and 1.0. The curves are plotted in dimensionless terms for a wide range of modified slenderness $\lambda_n$ (Equation 6.10). These curves demonstrate the degrading effect of increasing residual stress with an initial out-of-straightness of $L/1000$, being the fabrication tolerance permitted in AS4100 Clause 14.4.4.1.

![Column Strength Curves from AS 4100](image)

**Perry Formula**

An alternative approach is to consider the load-moment interaction. For an elastic compression member subject to bending and compression, the member must satisfy the following load-moment interaction for yielding:
Chapter 6  Additional Imperfections

\[
\frac{N_m}{N_y} + \frac{M_m}{M_y} \leq 1
\]  \hspace{1cm} (6.12)

where \( N_m \) is the maximum (critical) axial force, \( N_y = Af_y \) is the squash load, \( M_m \) is the maximum bending moment and \( M_y = Zf_y \) is the elastic moment capacity at the yield limit.

In particular, a member with an initial out-of-straightness, as depicted in Figure 6.3, must satisfy

\[
\frac{N_m}{N_y} + \frac{N_m(\delta_o + \delta)}{M_y} \leq 1.0
\]  \hspace{1cm} (6.13)

where \( \delta_o \) is the initial out-of-straightness and \( \delta \) is the deflection occurring under load.

![Diagram of initially out-of-straight slender compression members.](image)

Figure 6.3  Initially out-of-straight slender compression members.

It can be shown that the second order effects in Equation 6.13 are approximated by (Timoshenko and Gere 1961):

\[
\frac{N_m}{N_y} + \frac{N_m\delta_o}{M_y \left( 1 - \frac{N_m}{N_o} \right)} \leq 1.0
\]  \hspace{1cm} (6.14)
where \( N_o \) is the elastic buckling load. Equation 6.14 can be solved for \( N_m \) and rearranged as the Perry formula

\[
N_m = \left[ \frac{N_y + (1 + \eta)N_o}{2} \right] - \left\{ \left[ \frac{N_y + (1 + \eta)N_o}{2} \right] - N_yN_o \right\}^{\frac{1}{2}} \tag{6.15}
\]

where \( \eta \) is a section parameter given by

\[
\eta = \frac{\delta_o A}{Z} \tag{6.16}
\]

Figure 6.4 plots, in dimensionless terms, the AS 4100 column curve for \( \alpha_o = -0.5 \) and the Perry Equation (Equation 6.15) for a 48.3CHS4.0 strut with an initial out-of-straightness \( \delta_o \) of \( L/1000 \).

The AS 4100 column curve for \( \alpha_o = -0.5 \) was chosen because test results of eccentrically loaded (\( L/1000 \)) SHS columns were best approximated by the \( \alpha_o = -0.5 \) curve (Key et al. 1998). In the Perry formula, \( \delta_o = L/1000 \) matches the out-of-straight fabrication tolerance for a compression member permitted in AS 4100.

Figure 6.4 demonstrates the close agreement of the Perry formula for \( \delta_o \) of \( L/1000 \) and the AS 4100 column curve, \( \alpha_o = -0.5 \).

**AS 3610 – 1995**

To demonstrate the degrading effect of a larger initial out-of-straightness, Figure 6.5 compares the AS 4100 column curve for \( \alpha_o = -0.5 \) and the column strength curve for a 48.3CHS4.0 with an initial out-of-straightness of \( L/300 \) using the Perry formula. The value of \( \delta_o = L/300 \) was chosen because it is the acceptable out-of-straightness deviation for formwork tubes, props, frames, modular scaffold and tilt-up braces, as specified in AS 3610 (SA 1995) and referenced in AS 3850.1 (SA 1990).
Figure 6.4: AS 4100 column strength curve for $\alpha_0 = -0.5$ and the Perry formula for $\delta_0 = L/1000$.

Figure 6.5: AS 4100 column strength curve for $\alpha_0 = -0.5$ and the Perry formula for $\delta_0 = L/300$ for a 48.3CHS4.0.
Partial Resistance Factor, $\gamma_o$

For practical design purposes, the ratio of the member capacities $N_c/N_m$ represented by $\gamma_o$ is introduced where

$$\gamma_o = \frac{N_c}{N_m} \geq 1.0$$  \hspace{1cm} (6.17)

and $N_c$ is the column strength from AS 4100.

The $\gamma_o$-curves for $L/1000$ and $L/300$ are plotted in Figure 6.6 against the modified slenderness $\lambda_o$. The good agreement of the Perry formula and the AS 4100 curve for $\alpha_o = -0.5$ is clearly demonstrated by how closely the $\gamma_o$ curve for $L/1000$ approximates 1.0.

![Figure 6.6: $\gamma_o$-curves for steel columns out-of-straightness $L/1000$ and $L/300$.](image)

For 48.3CHS4.0, the $\gamma_o$-curve for $L/300$ varies between 1.0 and 1.3, peaking for columns with a modified slenderness $\lambda_o = 90$. For slender columns fabricated from
steel with higher yield strengths, the degrading effect of additional out-of-straightness, and the value of $\gamma_o$, decreases.

It is a useful concept to consider $\gamma_o$ as a partial resistance factor that explicitly takes account of additional out-of-straightness, such that:

$$N^* \leq \frac{\phi N_c}{\gamma_o}$$  \hspace{1cm} (6.18)

Simply, the effect of an additional out-of-straightness imperfection can be taken into account by reducing the AS 4100 design capacity by an appropriate partial resistance factor $\gamma_o$.

Alternatively, a more accurate solution is achieved by satisfying the inequality

$$N^* \leq \phi N_m$$  \hspace{1cm} (6.19)

where $N_m$ is calculated using Equation 6.15.

### 6.3 Eccentricity

Due to the nature and conditions of working on construction sites, accidental or unintentional end eccentricities occur as in the following examples.

- Despite the best intentions, bearers may be placed eccentric to the shore centreline, see Figure 2.1(a).

- Eccentricities may be implicit in the arrangement or design of the formwork, see Figure 2.1(b).

- Irregular or stiffness variations in the bearing surfaces under formwork shores might cause eccentricities, see Figure 2.2.
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- Although, out-of-plumb has a negligible affect on the strength of (pin-ended) braced compression members and can be safely neglected (Clarke et al. 1992), eccentricities do arise when real columns are erected out-of-plumb, see Figure 2.4.

- Tolerances at joints in shores are also a source of eccentricities, see Figure 6.7.

![Figure 6.7: Eccentricities arise at spigot joints](image)

**Perfectly Straight Member**

The effect of end eccentricities is to introduce bending moments that degrade the axial capacity of the shore. Figure 6.8 models the bending of a straight pin ended strut with different end eccentricities. The effect of the eccentricities can be represented by an axially loaded strut with end moments \( M_1 = Ne_1 \) and \( M_2 = Ne_2 \). It is advantageous to further simplify the representation of the moments to \( M \) and \( \beta_m M \), where \( \beta_m \) represents the ratio of the smaller moment over the larger moment:

\[
\beta_m = \frac{M_{SMALLER}}{M_{LARGER}}
\]  

(6.20)
(a) Eccentricities on the same side.

(b) Eccentricities on the opposite side.

Figure 6.8: An eccentrically loaded pin-ended strut.
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If the moments are equal and in opposite directions (as would be the case for eccentricities on the same side), then \( \beta_m = -1 \). Conversely, if the moments are equal but in the same direction (as would be the case for eccentricities on opposite sides), then \( \beta_m = 1 \). More generally:

\[
-1 \leq \beta_m \leq 1
\]  \hspace{1cm} (6.21)

**Load-moment Interaction**

In a similar manner to out-of-straightness, load-moment interaction equations can be used to take into account the effects of end eccentricities. In this case, Equation 6.14 is replaced by:

\[
\frac{N_m}{N_y} + \frac{N_m c_m e}{M_y} \left( \frac{1}{1 - N_m / N_y} \right) \leq 1.0
\]  \hspace{1cm} (6.22)

where \( e \) is the largest end eccentricity and \( c_m \) is a factor to take account of unequal bending moments given in Equation 2.7.

**Initially Curved Member**

When the member is initially curved, calculating the maximum bending moment is more difficult. Figure 6.9 models the bending of an initially curved pin ended strut with different end eccentricities.

It can be shown (Trahair and Bradford 1998) that Equation 6.22 becomes:

\[
\frac{N_m}{N_y} + \left( 1 + \frac{M_m}{M} \frac{N_y e}{M_y} \frac{M}{M_y} \frac{1}{N_m \delta_o} \right) \leq 1
\]  \hspace{1cm} (6.23)
(a) Eccentricities on the same side.

(b) Eccentricities on the opposite side.

Figure 6.9: An eccentrically loaded pin-ended strut with an initial curvature.
where

\[
\frac{N_m \delta_o}{M} = \frac{1}{\sqrt{\frac{N_m}{N_o} \left(1 - \frac{N_m}{N_o}\right)}} \left\{ \beta_m \csc \pi \frac{N_m}{N_o} + \cot \pi \frac{N_m}{N_o} \cos \frac{c}{L} \pi \frac{N_m}{N_o} \right. \\
\left. + \sin \frac{c}{L} \pi \frac{N_m}{N_o} \right\} \\
\cos \frac{c}{L} \pi
\]

(6.24)

and

\[
\frac{M_m}{M} = \cos \frac{c}{L} \pi \frac{N_m}{N_o} \left( \beta_m \csc \pi \frac{N_m}{N_o} + \cot \pi \frac{N_m}{N_o} \sin \frac{c}{L} \pi \frac{N_m}{N_o} \right) \sin \frac{c}{L} \pi \frac{N_m}{N_o} \frac{N_m \delta_o}{M} \left(1 - \frac{N_m}{N_o}\right) \\
\frac{1}{\cos \frac{c}{L} \pi \frac{N_m}{N_o}}
\]

(6.25)

In these equations, \( c \) represents the position along the length \( L \) of the maximum moment \( M_{\text{max}} \).

To check Equation 6.23, requires a process of reiteration by first solving Equation 6.24 using an initial guess of \( c/L \), and then solving Equation 6.25.

**Practical Considerations**

The most adverse effects occur when the end eccentricities are on the same side, as depicted in Figure 6.9(a). In the absence of any data, it would seem reasonable to assume that the same nominal unintentional eccentricity occurs at each end of the shore. In this case, if the eccentricities are equal, then Equation 6.23 simplifies to:

\[
\frac{N_m}{N_f} + \frac{N_m (\delta_o + e)}{M_f} \left(\frac{1}{1 - N_m/N_o}\right) \leq 1.0
\]

(6.26)

Equation 6.26 can be resolved using Equation 6.15 where \( \eta \) is a section parameter given by
\[
\eta = \frac{(\delta_o + \varepsilon)A}{Z}
\] (6.27)

As the moment increases relative to the axial load, considering only elastic behaviour would be conservative. Tests of tubular falsework units demonstrate that prior to failure the stresses in the concave side are in the non-linear (plastic) range (Peng et al. 2001). Figure 6.10 models the behaviour of tubular falsework members as plastic on the concave side while remaining elastic on the convex side.

![Diagram of yield boundary and stress distribution](image)

Figure 6.10: Elastic-Plastic behaviour of tubular falsework units prior to failure (Peng et al. 2001).

To take advantage of any plastic behaviour, the plastic moment capacity \( M_p \) can be substituted for \( M_r \) and Equation 6.26 becomes:

\[
\frac{N_m}{N_r} + \frac{N_a (\delta_o + \varepsilon)}{M_p} \left( 1 - \frac{1}{1 - N_m / N_a} \right) \leq 1.0
\]

(6.28)

As for Equation 6.14, Equations 6.15 and 6.16 can be used to solve this inequality, in this case \( \eta \) is given by

\[
\eta = \frac{(\delta_o + \varepsilon)A}{S}
\]

(6.29)

where \( S \) is the plastic section modulus.
Partial Resistance Factor, $\gamma_e$

Previously, the utility of a partial resistance factor $\gamma_e$ was beneficial in taking account of the effects of any additional initial out-of-straightness. In a similar way, $\gamma_e$ can be considered as a partial resistance factor to take account of unintentional eccentricities, such that:

$$\gamma_e = \frac{N_e}{N_m} \geq 1.0$$  \hfill (6.30)

where $N_e$ is the column strength from AS 4100 and $N_m$ is the maximum axial load determined from Equations 6.15 and 6.29.

![Graph](image)

Figure 6.11: $\gamma_e$-curves eccentrically loaded shores in "new" condition.
Chapter 6  Additional Imperfections

Figure 6.11 depicts the $\gamma_e$-curves for a 48.3CHS4.0 steel column out-of-straight $L/1000$ and loaded eccentrically 5 mm, 15 mm and 25 mm. In a similar manner to Figure 6.6, Figure 6.11 plots $\gamma_e$ curves against the modified slenderness $\lambda_m$.

Figure 6.11 demonstrates that the influence of eccentricity decreases with increasing slenderness. For formwork shores, typical values of modified slenderness will fall between 75 to 200. The slenderness of formwork frames and modular scaffolding is at the lower end of this range, the most slender shores being single post shores, such as adjustable props. For shores with a slenderness of either 75 or 200 and end eccentricities of 5, 15 and 25 mm: the maximum and minimum values of $\gamma_e$ are (1.35, 2.0 and 2.6) and (1.06, 1.2, 1.35) respectively.

**Partial Resistance Factor, $\gamma_{oe}$**

For formwork shores complying to AS 3610, it might be useful to consider $\gamma_{oe}$ as a partial resistance factor to take account of the combined effects of additional out-of-straightness and unintentional eccentricities, such that:

$$\gamma_{oe} = \frac{N_c}{N_m} \geq 1.0 \quad (6.31)$$

and $N_c$ is the column strength from AS 4100 and $N_m$ is the maximum axial load determined from Equations 6.15 and 6.29.

In a similar manner to Figure 6.11, Figure 6.12 compares the $\gamma_{oe}$-curves for the same 48.3CHS4.0: concentrically loaded sections with an out-of-straightness of $L/1000$ (as per AS 4100); and the same section with an out-of-straightness $L/300$ and loaded eccentrically by 5 mm, 15 mm and 25 mm.

Over the range of modified slenderness of 75 to 200, the maximum values of $\gamma_{oe}$ for 5, 15 and 25 mm eccentricities vary from (1.55, 2.17 and 2.76) and (1.18, 1.32, 1.47), respectively. This demonstrates the decreasing influence of additional out-of-straightness with increasing slenderness.
Chapter 6  Additional Imperfections

Over the same range of modified slenderness and eccentricities, the values of the ratio $\gamma_{oe}/\gamma_e$ are (1.15, 1.08 and 1.06) and (1.11, 1.10, 1.09), respectively. This demonstrates a reduction in load capacity for eccentrically loaded shores out-of-straightness by $L/300$ of approximately 10%.

![Graph showing $N_0/N_m$ against modified slenderness, $\lambda_a$.

Figure 6.12: $\gamma_{oe}$-curves shores, out-of-straight $L/300$ and eccentrically loaded.

Significantly, the effects of the nominal values of out-of-straightness and eccentricity are section specific, in that the value of both $\gamma_e$ and $\gamma_{oe}$ decrease as section dimensions increase. Thus, not only will a formwork shore fabricated from a larger section, say a 60.3CHS3.6, have a greater strength than the 48.3CHS4.0 modelled in Figures 6.10, 6.11 and 6.12, it will be less affected by imperfections. Similarly, for sections with a modified slenderness $\lambda_a > 90$ increasing the strength of steel will amplify the benefit.
6.4 Example

To demonstrate these techniques for taking account of additional out-of-straightness and end eccentricities, reconsider the example calculation in Chapter 2.

Determine a suitable section for a shore, given a design axial force $N^* = 41.6$ kN, equal and opposite end eccentricities of 19 mm and a permitted out-of-straightness of $L/300$ for an effective length of 2000 mm.

Try a commonly used formwork shore section, namely: 60.3x 3.6 CHS G250.

To satisfy Equation 6.19, $N^* \leq \phi N_m$

Substituting into Equation 6.29, $\delta_o = 2000/300 = 6.7$ mm, $e = 19$ mm, $A = 641$ mm$^2$ and $S = 11,600$ mm$^3$ (AISC 1999), gives:

$$\eta = \frac{(6.7+19)641}{11600} = 1.42$$

(6.32)

Substituting into Equation 6.15, $\eta = 1.42$, $N_Y = 250(641) = 160.25$ kN and $N_{omb} = 128$ kN (AISC 1992), gives:

$$N_m = \left[ \frac{160 + (1+1.42)(128)}{2} \right] - \left[ \frac{160 + (1+1.42)(128)}{2} \right]^2 = 48.6 \text{ kN}$$

(6.33)

therefore

$$N_m = 48.6 \text{ kN}$$

(6.34)

and

$$\phi N_m = 0.9(48.6) = 43.7 \text{ kN}$$

(6.35)

$$41.6 \leq 43.7, \text{ OK}$$

(6.36)
Chapter 6  Additional Imperfections

A 60.3 x 3.6 CHS G250 section is satisfactory and more cost-effective than the 60.3 x 5.4 CHS G250 section necessary to comply with AS 4100 Clause 8.4.2.2.

As a matter of interest, in this example $\gamma_{oe} = \frac{N_c}{N_m} = \frac{87.1}{48.6} = 1.79$.

6.5 Conclusions

This chapter highlights that in some applications such as formwork, scaffold, tilt-up panels and transportable bridging, it is acceptable to reuse compression members with an out-of-straightness over and above that resulting from initial fabrication. In addition, those same members are often eccentrically loaded.

New column strength curves were derived using the Perry formula that enables each of these imperfections to be taken account of explicitly. For similar values of out-of-straightness, the new column curves closely approximate the appropriate AS 4100 column curves.

Techniques for explicitly taking account of the second-order effects of formwork shores with additional out-of-straightness and unintentional eccentricities are presented. For practical design purposes, these techniques have several attractions:

- they are easily recognised and understood by designers familiar with permissible stress methods;

- they provide an exact second-order solution without the trial and error that characterises load-moment interaction equations; and

- they are easily modified to take account of plastic behaviour.

The concept of a partial resistance factor $\gamma$ was introduced. When plotted against slenderness, $\gamma$ curves proved useful for realising the influence of imperfections and provide a simplified design procedure.
Chapter 6  Additional Imperfections

For typical formwork shore sections, neglecting even small additional imperfections leads to significant overestimation of the axial capacity of a shore. In particular, shore capacity is sensitive to end eccentricities. Relaxation of permitted out-of-straightness tolerances are less significant, e.g. $L/1000$ to $L/300$ reduces the capacity of shores fabricated from 48CHS4.0 G250 sections by approximately 10%.
Chapter 7  The Reliability of Used Shores

7.1 Introduction

Chapters 4 and 5 investigated the reliability of current practices based on data for “new” steel columns used in permanent construction. In contrast, during their working life, the condition of some formwork shores deteriorates because they are reused many times or sustain damage. As a result, these members have additional imperfections over and above any initial imperfections that were present when they were new. As it would be uneconomical to discard or continually repair members with minor damage, it is accepted practice to reuse formwork shores with imperfections greater than permitted in normal steel construction.

In addition, due to the nature and conditions of working on construction sites, as well as erection procedures, accidental or unintentional end eccentricities are likely to occur. Furthermore, formwork shores are erected out-of-plumb to a degree greater than would be expected in normal steel construction.

Additional imperfections and unintentional end eccentricities degrade the strength and reliability of the formwork shores. Chapter 6 provided an insight into the influence of additional out-of-straightness and unintentional end eccentricities as well as establishing techniques for explicitly taking account of their effects.

This chapter investigates whether past and current national formwork standards reliably address the effects of additional imperfections and unintentional end eccentricities. Some standards deal with these issues explicitly, while others simply recommend larger factors of safety. Understanding how the different approaches compare is important.

This is achieved by using the same first-order probability methods used to determine the reliability of “new” formwork shores. In some cases, the resistance data for “new” steel columns is replaced by practical estimates of the strengths of formwork
as specified in the relevant standard. Otherwise, the data for “new” columns is adjusted by specified resistance factors.

Initially, the relative reliability of the modified AS 3610 and new design loads (from Chapter 5) are compared to past and current permissible stress and limit states practice, specifically past Australian practice (SAA 1974) and current American (ACI 2001) and British (BS 1996) permissible stress standards as well as current Israeli (SII 1998) and draft European (ECS 2000) limit states standards. The relative reliability of these authoritative references is used as a basis to calibrate a capacity reduction factor compatible with the proposed design loads.

### 7.2 Probability Analysis

In Chapter 5, the following generic expression for the reliability index $\beta$ was derived:

$$\beta = \ln \left( \frac{\lambda_R \left( \gamma_f G_f + \gamma_c G_c + \gamma_q Q \right)}{\phi \lambda_S \left( G_f + G_c \right)} \right)$$

where

$G_f$ and $G_c$ are the weight of the formwork and concrete (including reinforcement), and $Q$ is an imposed action;

$\gamma_f$, $\gamma_c$ and $\gamma_q$ are load factors for formwork, concrete and imposed actions;

$\lambda_R$ is the ratio of mean to the nominal resistance ($R_m/R_n$);

$\lambda_S$ is the ratio of mean to the nominal action effect ($S_m/S_n$);

$\phi$ is the characteristic value of the load action effects; and

$V_R$ and $V_S$ are the coefficients of variation of resistance and action effects; and
\( \phi \) is the ultimate limit state capacity reduction factor for steel columns specified in AS 4100 \((\phi = 0.90)\) that defaults to unity when calculating \( \beta \) for permissible stress methods.

As before, this analysis adopts some of the design load combinations in the relevant standards as presented in Tables 5.1 and 5.2 as well as new Equations 5.8 and 5.9. The latter equations were calibrated so that “new” formwork shores achieved a target reliability index compatible with past practice, indices recommended in the literature and adopted in companion standards.

For “used” shores, the value of some terms in Equation 5.7 are adjusted; specifically:

- where a formwork standard specifies that the capacity of shores shall be based on testing, e.g. the ultimate capacity (determined by testing) divided by a specified safety factor \( \Omega \), it is assumed, \( \lambda_R = \Omega \) and \( V_R = 0.15 \). This value of \( V_R \) is recommended in AS 3610 Table A2 as appropriate in the absence of data from more than 30 tests; or

- where a formwork standard specifies additional resistance factors, the value of \( \lambda_R \) and \( V_R \) from Chapter 5 remain unaltered. However, the value of the capacity reduction factor \( \phi \) is adjusted accordingly.

**AS 1509**

When formwork materials were to be reused, AS 1509 Clause 3.5 limited the maximum permissible stress to 85% of that given in the applicable material standard.

For proprietary formwork components (including shores), the SAA Formwork Committee recommended that the strength be determined by the ultimate strength method. This meant, according to AS 1509 Clause 3.6, that the load on a component could not exceed its ultimate strength determined from tests divided by a
load factor. In the case of shores, load factors of 2.5 and 3.0 were specified for formwork frames and single steel props, respectively.

Significantly, under AS 1509, steel formwork shores were required to be straight and true within the tolerances of the relevant material Standard, e.g. a maximum out-of-straightness of $L/1000$ (SAA 1981). This implied that the safety factors did not take account of any additional out-of-straightness.

Thus, to determine the reliability index $\beta$ of the AS 1509 permissible stress method, the value of $\lambda_R$ and $V_R$ remain unaltered and the value for $\phi$ is set to 0.85, not unity as before. For the ultimate strength method, nominal values of $\lambda_R = 2.75$ and $V_R = 0.15$ were chosen.

**ACI 347**

In a similar manner to AS 1509, ACI 347 recommends that the load carrying capacity of formwork shores be determined by applying a safety factor of 2.5 to the ultimate strength, established by testing. In addition, Committee 347 recommends the use of reduced values for allowable loads for shores that experience substantial reuse.

Accordingly, the reliability index $\beta$ for reused shores to ACI 347 is based on nominal values of $\phi = 0.9$, $\lambda_R = 2.5$ and $V_R = 0.15$.

**BS 5975**

For reusable components whose strength is established by testing, a minimum safety factor of 2.0 is recommended (CS 1995). For example, props needed to be tested with out-of-plumb imperfections and eccentric loading.

Thus, the reliability index $\beta$ for reused shores to BS 5975 is determined assuming a nominal values of $\lambda_R = 2.0$ and $V_R = 0.15$. 

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SI 904

SI 904 provides no guidance on eccentric loading or the acceptability of imperfections in formwork shores. It does require that the capacity of steel formwork components to be designed using a partial resistance factor of 1.15 rather than 1.08 as specified in SI 1225 for normal construction. This is equivalent to a 6% reduction in capacity.

Thus, the reliability index $\beta$ for reused shores to SI 904 is determined assuming nominal values of $\phi = 0.94(0.9) = 0.85$.

PrEN 12812

This standard is unique in that unless the formwork design and construction is of equal quality to permanent construction, the capacity of formwork calculated in accordance with the relevant standard must be reduced by the application of a partial resistance factor of 1.15. In addition, prEN 12812 requires designers to explicitly account for imperfections such as: eccentricities of loads; angular changes and eccentricities caused by joint looseness; and bow and sway imperfections.

Thus, the reliability index $\beta$ for "new" shores to prEN 12812 is determined assuming a nominal value of $\phi = 0.9/1.15 = 0.78$

7.3 Results

Figure 7.1 shows the relative reliability of designing shores in "new" condition using the modified AS 3610 and new design load combinations (Equations 5.8 and 5.9) compared to the same shores tested using the ultimate strength methods in AS 1509, ACI 347 and BS 5975.
Chapter 7  The Reliability of Used Shores

Figure 7.1: A comparison of the reliability of new and modified AS 3610 limit states design loads for shores in “new” condition ($\phi = 0.9$) versus past Australian and international ultimate strength practice for testing shores.

Figure 7.2: A comparison of the reliability of new and modified AS 3610 limit states design loads of shores in “new” condition ($\phi = 0.9$) versus current international limit states practice designing shores.
Chapter 7  The Reliability of Used Shores

Figure 7.3: A comparison of the reliability of new and modified AS 3610 limit states design load shores with $\phi = 0.7$ versus past Australian and ultimate strength methods for shores tested in "new" condition.

Figure 7.4: A comparison of the reliability of new, existing and modified AS 3610 limit states design loads ($\phi = 0.8$) versus current international limit states practice for designing shores.
Figure 7.2 shows the same comparison (Equations 5.8 and 5.9), but for shores designed using the limit states methods in SI 904 and prEN 12812.

The comparisons in Figures 7.1 and 7.2 are replicated in Figures 7.3 and 7.4 with one difference; for Equations 5.8 and 5.9, a nominal capacity reduction factor $\phi = 0.7$ and 0.8 is used instead of value $\phi = 0.9$ from AS 4100, respectively.

### 7.4 Discussion

In Australia, prior to the publication of AS 3610, formwork shores could only be reused if they were in “new” condition, which negated the need for designers to consider the effects of additional out-of-straightness. At the time, in the absence of specific guidance in AS 1509, designers largely neglected eccentricities. However, it is not clear if this was justified, i.e. if the value of the load factors for shores (2.5 and 3.0) were intended to take account of unintentional end eccentricities.

The approach taken in AS 1509 was similar to ACI 347 which required similar factors of safety (2.5) on ultimate strength of a concentrically loaded shores. One difference is that ACI 347 requires a further reduction in allowable load for shores experiencing substantial reuse, presumably, to take account of any minor degradation of shore capacity with reuse.

Figure 7.3 demonstrates to calibrate Equations 5.8 and 5.9 to past Australian practice in AS 1509 it is necessary to use a capacity reduction factor $\phi = 0.7$ instead of $\phi = 0.9$, as specified in AS 4100. To meet target reliability indices this would imply that the ratio of $0.9/0.7 = 1.3$ was sufficient to take account of unintentional eccentricities.

It is interesting to compare these similar approaches with that adopted in BS 5975: shores are permitted to be out-of-straight up to $L/500$; tests must take account of the detrimental effects imperfections such as out-of-straightness, column inclination and end eccentricities; and the factor of safety on testing is lower (2.0). It would seem that the British consider that a lower factor of safety is appropriate if any detrimental effects are explicitly taken into account.
Possibly, the two approaches achieve a similar result: adopt a high safety factor and safely neglect imperfections versus using a lower safety factor and explicitly taking account of imperfections.

If this were true, the ratio of safety factors (median AS1509/BS 5975) 2.75/2.0 = 1.38 might reflect a reasonable value for a partial resistance factor necessary to take account of imperfections and eccentric loading. There is some support for this proposition in Chapter 6, but it is limited to slender shores (i.e. with a modified slenderness $\lambda_n$ approaching 200) in “new” condition, leaving some doubt in the case of less slender shores.

The relationship between the reliability of Equations 5.8 and 5.9 and BS5975, in Figure 7.1 suggests that provided that any detrimental effects of reuse are explicitly accounted for, Equations 5.8 and 5.9 are satisfactory. Countering this is Figure 7.2, where Equations 5.8 and 5.9 appear less reliable than prEN 12812 (which requires any detrimental effects to be taken into account).

The reliability index for BS 5975 is in the range 3.3 to 2.3 and less than prEN 12812 at 4.2 to 2.9. The former falls short of the target reliability index range of 3.5 to 2.5 while the latter, particularly for thin concrete, exceeds the range.

To achieve parity with prEN 12812, it is necessary to calibrate Equations 5.8 and 5.9 by reducing the capacity reduction factor $\phi$ from 0.9 (as specified in AS 4100) to 0.8. Figure 7.4 demonstrates the closeness of the result.

In the absence of any test data, no attempt has been made to determine the relative reliability of Equations 5.8 and 5.9 when the capacity of a shore is established by testing in accordance with Appendix A of AS 3610. Some insight might come from comparing the recommended values for sampling factor $k_s$ in AS 3610 Table A1 and the ratio $\lambda_R/\phi$ as it applies for calculations to AS 4100. Such a comparison suggests that given the same coefficient of variation ($V_R = 0.15$), testing might be equally or more reliable than limit states calculations when $k_s \geq \lambda_R/\phi$ (i.e. for Equations 5.8 and 5.9, $k_s \geq 1.21/0.9 = 1.34$ and for the existing AS 3610 design load equations, $k_s \geq 1.3(1.21/0.9) = 1.75$).
Chapter 7  The Reliability of Used Shores

7.5 Conclusions

Simple first order probability techniques are used to plot the relative reliability of the methods past Australian and current international practice use to take account of detrimental effects present in formwork shores.

In general, national formwork Standards follow one of two approaches.

- Specify high safety factors ($\Omega > 2.5$) and neglect the effects of imperfections (AS 1509 and ACI 347);

- Specify lower safety factors and explicitly account for the effects of imperfections (BS 5975 and prEN 12812).

For slender shores, there is some evidence that the different approaches are comparable. Otherwise, the former approach appears less reliable.

If the effects of imperfections are dealt with explicitly, the relative reliability of steel shores designed to AS 4100 using design load Equations 5.8 and 5.9 falls between BS 5975 and prEN 12812. It would be necessary to reduce the capacity reduction factor ($\phi$) for steel columns given in AS 4100 to achieve a similar level of reliability to prEN 12812. To achieve a similar level of reliability to past Australian practice would require a further reduction in the capacity reduction factor ($\phi$).
Chapter 8 Discussion

8.1 Introduction

The objective of this research was to develop more reliable methods for the design of the steel shores that commonly support horizontal slab and beam formwork. To achieve this:

- current Australian practice was reviewed and compared to past Australian and current international Standards/Codes of Practice, as well as the results of recent research;
- first-order probability techniques were developed to compare the relative reliability of current Australian Practice;
- new design load combinations were calibrated;
- the influence of additional imperfections not present in normal steel columns was investigated;
- procedures were derived that enable each additional imperfection to be taken into account explicitly; and
- first-order probability techniques were used to compare the relative reliability of the different methods that past and current national formwork Standards use to address the effects of additional imperfections.

In this chapter, the results of the research are reviewed.

8.2 Analysis of Results

Current design loads

In Figures 3.3 and 3.4, the formwork shore design loads specified in past Australian, as well as current international formwork Standards and Codes of Practice were compared over a range of concrete thickness. Similarly, Figure 5.1 compared the relative reliability of the design methods in the same references. The close match suggests that formwork shore design loads from past Australian and current international formwork Standards/Codes of Practice are consistent.
Chapter 8  Discussion

In a similar way, Figures 3.4, 5.2 and 5.3 compared the design loads and relative reliability of limit states methods from AS 3610. These comparisons suggest that, especially as concrete thicknesses increase, current Australian limit states methods appear less reliable than international practice. In fact, except for formwork supporting concrete less than 0.3 m thick, current Australian limit states methods appear not to achieve target reliability indices chosen in Chapter 4, i.e. $2.5 \leq \beta \leq 3.5$.

When compared to predictions of the maximum load in formwork shores, Figure 3.3 supports suggestions in the literature that as concrete thickness increases, current permissible stress methods increasingly underestimate the maximum load in shores. Figure 3.4 demonstrates that the same cannot be said for limit states methods.

Interestingly, the shape of the $\beta$ curves in Figures 5.1 and 5.2 do not suggest such a distinction. In fact, the reliability of both permissible stress and limit states methods decreases with increasing concrete thickness. However, the shape of the curves might be misleading because the probability analysis assumes a constant COV for the action effects. In contrast, the distribution of data in Figures 3.1 and 3.2 suggests that the COV is a function of concrete thickness and reduces with increasing concrete thickness. Similarly, it could be argued that the apparent higher levels of reliability for shores supporting thinner concrete are also misleading.

**Proposed New Design Load Combinations**

To address the shortcomings in the existing limit states design load combinations, new combinations were calibrated to match the reliability of past Australian practice and target reliability indices (see Figure 5.4). The design load combinations proposed are:

$$S^* = 1.3(1.25G_r + 1.25G_c + 1.5Q_{uv} + 1.5M), \text{ kN/m}^2$$  \hspace{1cm} (5.8)

$$S^* = 1.6(G_r + G_c) + 2.5, \text{ kN/m}^2$$  \hspace{1cm} (5.9)

Equation 5.8 amplifies the existing AS 3610 design load combination (Equation 2.2) by a factor of 1.3 and is useful for amending current methods. Equation 5.9 reflects
the fact that the actual load in shores can be accurately estimated considering only combined weight of the formwork and concrete plus a constant.

The mean and COV of the relative shore loads used to calibrate Equations 5.8 and 5.9 were largely comparable irrespective of whether the continuity of bearers was taken into account. Thus, for simplicity, the continuity of bearers may safely be neglected when using Equations 5.8 and 5.9.

Once the concrete has reached full height, the action effects arising from concrete delivery methods, concrete mounding, workmen and equipment and stacked materials need to be taken into account. Recent research suggests that the value of 2.4 kN/m² is appropriate for the weight of workmen, equipment and stacked materials. In addition, there is evidence in the literature that this value exceeds the dynamic effects of concrete placement.

**Additional Imperfections**

Concerns that current formwork shore design loads tend to underestimation might be somewhat academic if the factor of safety used to determine the safe working load of formwork shores was sufficiently high. Indeed, it is argued in the literature that a factor of safety of 2.5 is sufficient to compensate for the level of underestimation measured in shore loads. However, this argument neglects to consider the detrimental effects of additional imperfections and unavoidable end eccentricities common to formwork shores but not present in normal steel columns.

In Chapter 6, techniques for explicitly taking account of the second-order effects of formwork shores with additional out-of-straightness and unintentional eccentricities are presented, namely:

\[ N' \leq \phi N_m \]  

(6.19)

where \( N_m \) is given by
\[ N_m = \left[ \frac{N_y + (1 + \eta)N_o}{2} \right] - \left[ \frac{N_y + (1 + \eta)N_o}{2} \right]^2 - N_y N_o \right]^{1/2} \quad (6.15) \]

and where \( \eta \) is a section parameter such that:

\[ \eta = \frac{(\delta_o + e)A}{S} \quad (6.29) \]

For practical design purposes, these equations have several attractions:

- they are easily recognised and understood by designers familiar with permissible stress methods;

- they provide an exact second-order solution without the trial and error that characterises load-moment interaction equations; and

- they take account of plastic behaviour.

In addition, Chapter 6 introduced the concept of a partial resistance factor \( \gamma \) such that:

\[ N^* \leq \frac{\phi N_{\gamma}}{\gamma} \]

where

\[ \gamma = \frac{N_{\gamma}}{N_m} \geq 1.0 \]

Plotting curves for \( \gamma \) against slenderness proved useful in realising the influence of each of these additional imperfections. For typical formwork shore sections, Figures 6.6, 6.11 and 6.12 demonstrated that neglecting even small additional imperfections leads to significant overestimation. In particular, the sensitivity of shore capacity to end eccentricity becomes apparent from comparing Figures 6.6 and 6.11. For example, the relaxation of the out-of-straightness tolerance from \( L/1000 \) (AS 4100)
to $L/300$ permitted in AS 3610 reduces the capacity of shores fabricated from 48CHS4.0 G250 sections by approximately 10%. For the same section, there is an overall reduction in capacity of between 35% to 70% due to the combined effects of an out-of-straightness of $L/300$ as well as equal and opposite end eccentricities of 25 mm.

The concept of a partial resistance factor $\gamma$ is also useful for simplifying design, especially if manufacturers of proprietary shores were to publish $\gamma$-curves. Such an approach would go some way to addressing criticism that addressing imperfections explicitly is overly complex and onerous.

**Factors of Safety**

In Chapter 7, first-order probabilistic techniques were used to compare the techniques different Standards and Codes of Practice use to take account of additional imperfections and unintentional end eccentricities. In general, one of two approaches are adopted.

- Specify high safety factors ($\Omega > 2.5$) and effectively neglect the effects of imperfections (AS 1509 and ACI 347).

- Specify lower safety factors and explicitly account for the effects of imperfections (BS 5975 and prEN 12812).

Although both AS 1509 and ACI 347 recommend that end eccentricities be avoided and shores be in “new” condition, this rarely occurs. In practice, imperfections and end eccentricities such as that depicted in Figures 2.1, 2.2, 2.3, 2.4 and 6.7 are common. The results from Chapter 6 suggest that, possibly except for slender shores, this approach is less reliable and casts doubt on the validity of arguments that a factor of safety of 2.5 adequately compensates for load underestimation.

Importantly, in developing Equations 5.8 and 5.9 any detrimental effects arising from reusing steel shores and differences in the quality of formwork practices compared to normal permanent construction were neglected. However, Figures 7.1 and 7.2 demonstrate that providing that the effects of imperfections are dealt with
explicitly, the relative reliability of steel shores designed to AS 4100 using the proposed new design load combinations falls between current British and draft European practice. To achieve a similar level of reliability to prEN 12812, Figure 7.4 demonstrates that it would be necessary to reduce the capacity reduction factor for steel columns from $\phi = 0.9$ given in AS 4100 to $\phi = 0.8$.

As a matter of interest Figure 7.3 demonstrates that to achieve a similar level of "reliability" to past Australian practice, the capacity reduction factor would need to be further reduced to $\phi = 0.7$.

### 8.3 Further Research

The cause of the disparity between predicted and actual loads in formwork shores is unclear. It is unlikely to be the result of concrete placement patterns, initial preloading or neglecting the effects continuous bearers. A probable cause is the presence of small initial gaps between shores and the bearers they support, combined with differences in shore and foundation stiffness. If this were the case, the action effects in the continuous bearers the shores support may also be underestimated.

On the other hand, nearly all the limited data on formwork shore loads has been measured at serviceability limit state and possibly at ultimate limit states a partial equalisation of the loads occurs. In addition, in determining the influence of additional imperfections the semi-rigid behaviour of connections in some proprietary formwork shores systems has been neglected.

In the absence of further research to address these issues, a conservative approach has been followed.
Chapter 9  Conclusions

Formwork shore design loads from past Australian and current international formwork Standards/Codes of Practice were compared and found to be consistent. A similar comparison of limit states design loads suggests that, especially as concrete thicknesses increase, current Australian limit states methods appear less conservative than international practice.

The cause of this disparity is unclear but might not be of concern if shores have a sufficiently high factor of safety against collapse. Unfortunately, factors of safety vary from Standard to Standard. In addition, some Standards permit (while others neglect) unintentional eccentric loading and shores with levels of imperfections, such as out-of-straightness, much greater than allowed in normal steel construction. This makes comparisons difficult, particularly when trying to compare permissible stress and limit states methods.

First order probability techniques were developed to determine and compare the reliability of current practice. A target reliability index in the range of 2.5 to 3.5 was chosen. Using these techniques, the reliability of formwork shores designed using current Australian limit states design loads was compared with past Australian and current international practice. This analysis was based on action effect data from measurements of the actual load in shores and the resistance of “new” steel columns. The resulting comparisons show that, except for shores supporting thin concrete slabs, current Australian limit states design loads appear less reliable and do not achieve target indices. Accordingly, new limit states design load combinations are calibrated to achieve a level of reliability compatible with past practice.

To take account of additional imperfections and construction techniques permitted in formwork but not present in normal steel construction, their influence was investigated. New column strength curves were derived, using the Perry formula, that enables each of these imperfections to be taken into account. For typical formwork shore sections, neglecting even small additional imperfections leads to
significant overestimation of the axial capacity of a shore. To take account of the reduction in capacity, the concept of a partial resistance factor \( \gamma \) was introduced.

First order probability techniques were also used to compare the level of safety of formwork shores taking into account the approaches and factors of safety specified in different formwork Standards. The results show that the reliability of steel formwork shores designed in accordance with AS 4100 is similar to current international practice, providing the effects of additional imperfections are dealt with explicitly.

In summary, formwork shores are reliable if designed in accordance with the following.

- The load in formwork shores is determined using design load combinations that take account of the effects of load redistribution. In this case, the continuity of bearers may safely be neglected.
- Once the concrete has reached full height, the action effects arising from concrete delivery methods, concrete mounding, workmen and equipment and stacked materials are taken into account.
- The capacity of steel formwork shores is determined by calculation or testing that explicitly accounts for any permitted or unavoidable imperfections.
Appendix A  Relative Shore Loads

Table A1: Relative shore loads, (maximum loads) beam method, (Ikäheimonen 1997)

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<thead>
<tr>
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1) only the concrete load  2) loads measured after concreting

mp1, mp2, etc identify shores measured

Table A2: Relative shore loads, (maximum loads) simplified method, (Ikäheimonen 1997)

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1) only the concrete load  2) loads measured after concreting

mp1, mp2, etc identify shores measured
Appendix B  An Example of Manufacturer’s Information

Formwork Shore Frame Capacities

![Graph showing Formwork Shore Frame Capacities with lines for Primary Beams Perpendicular and Parallel to Frames.](image)

**Figure B1:** Maximum allowable leg load for various jack extensions

The capacities shown in Figure B1 are working loads determined by destructive tests of used equipment in accordance with AS 3610 – 1995 Appendix A using a limit state conversion factor of 1.31. The capacities are appropriate for braced frames 1829 mm high and 1219 mm wide, assembled as top restrained towers up to three frames high with top and bottom jacks loaded eccentrically up to 19 mm.
### Appendix C  Data for Figures

#### Table C1: Figure 3.3 – Design Action Effect (kN/m²)

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#### Table C2: Figure 3.4 – Design Action Effect (kN/m²)

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### Appendix C  Data for Figures

#### Table C3: Figure 3.5 – Design Action Effect (kN/m²)

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- 115 -
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**Table C5.** Figure 5.2 – Reliability Index ($\beta$)

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**Table C6.** Figure 5.3 – Reliability Index ($\beta$)
### Table C7: Figure 5.4 – Reliability Index ($\beta$)

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### Table C8: Figure 6.2 – Dimensionless Actions ($N/N_0$)

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### Table C9: Figure 6.4 – Dimensionless Actions \( \left( \frac{N}{N_y} \right) \)

| Column Curve | \( 6.4 \) | \( 19.1 \) | \( 31.8 \) | \( 44.5 \) | \( 57.2 \) | \( 69.9 \) | \( 82.7 \) | \( 95.4 \) | \( 108.1 \) | \( 120.8 \) | \( 133.5 \) | \( 146.3 \) | \( 159.0 \) | \( 171.7 \) | \( 184.4 \) | \( 197.1 \) | \( 209.8 \) | \( 222.6 \) |
|--------------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|
| \( N_t \), Yielding | 1.00      | 1.00      | 1.00      | 1.00      | 1.00      | 1.00      |           |           |           |           |           |           |           |           |           |           |           |           |           |           |
| \( N_t \), Elastic Buckling |           |           |           |           |           |           | 0.87      | 0.68      | 0.54      | 0.44      | 0.37      | 0.31      | 0.27      | 0.23      | 0.20      | 0.18      | 0.16      |
| \( N_c, \alpha_b = -0.5 \) | 1.00      | 0.99      | 0.96      | 0.93      | 0.87      | 0.81      | 0.73      | 0.63      | 0.54      | 0.46      | 0.39      | 0.33      | 0.28      | 0.25      | 0.22      | 0.19      | 0.17      | 0.15      |
| \( N_m, L/1000 \) | 0.99      | 0.97      | 0.95      | 0.92      | 0.88      | 0.82      | 0.74      | 0.64      | 0.54      | 0.46      | 0.39      | 0.33      | 0.28      | 0.24      | 0.21      | 0.19      | 0.17      | 0.15      |

### Table C10: Figure 6.5 – Dimensionless Actions \( \left( \frac{N}{N_y} \right) \)

| Column Curve | \( 6.4 \) | \( 19.1 \) | \( 31.8 \) | \( 44.5 \) | \( 57.2 \) | \( 69.9 \) | \( 82.7 \) | \( 95.4 \) | \( 108.1 \) | \( 120.8 \) | \( 133.5 \) | \( 146.3 \) | \( 159.0 \) | \( 171.7 \) | \( 184.4 \) | \( 197.1 \) | \( 209.8 \) | \( 222.6 \) |
|--------------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|
| \( N_t \), Yielding | 1.00      | 1.00      | 1.00      | 1.00      | 1.00      | 1.00      |           |           |           |           |           |           |           |           |           |           |           |           |           |           |
| \( N_t \), Elastic Buckling |           |           |           |           |           |           | 0.87      | 0.68      | 0.54      | 0.44      | 0.37      | 0.31      | 0.27      | 0.23      | 0.20      | 0.018     | 0.016     |
| \( N_c, \alpha_b = -0.5 \) | 1.00      | 0.99      | 0.96      | 0.93      | 0.87      | 0.81      | 0.73      | 0.63      | 0.54      | 0.46      | 0.39      | 0.33      | 0.28      | 0.25      | 0.22      | 0.19      | 0.17      | 0.15      |
| \( N_m, L/300 \) | 0.97      | 0.91      | 0.85      | 0.78      | 0.71      | 0.63      | 0.55      | 0.48      | 0.41      | 0.36      | 0.31      | 0.27      | 0.23      | 0.21      | 0.18      | 0.16      | 0.15      | 0.13      |
### Table C11: Figure 6.6 – Partial Resistance Factor ($\gamma_6=N_c/N_m$)

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Table C13: Figure 6.12 – Partial Resistance Factor ($\gamma_m = N_p / N_m$)
### Table C14: Figure 7.1 – Reliability Index ($\beta$)

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### Appendix C  Data for Figures

#### Table C16: Figure 7.3 – Reliability Index ($\beta$)

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ACI (2001). **ACI 347R-2001 Guide to Formwork for Concrete.** Detroit, American Concrete Institute.


ASCE (1996). **Design loads on structures during construction.** New York, N.Y., American Society of Civil Engineers.


References


References


References


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References


References


