TRL REPORT 131

A LITERATURE AND DESIGN REVIEW OF CRIB WALL SYSTEMS

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Prepared for: Project Record: E480A/BG Design, Construction and Performance of Crib Wall Systems
Customer: Bridges Engineering Division, DOT (Dr D Bush)

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Civil Engineering Resource Centre
Transport Research Laboratory
Crowthorne, Berkshire, RG45 6AU
1995

ISSN 0968-4107
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EXECUTIVE SUMMARY

The long-term use of crib walls and the large number of successful applications, both in Western Europe and North America, suggest that there is a general acceptance of the suitability and economy of this form of construction. Crib walling has advantages associated with its relative ease of construction, economy, and aesthetics.

BS 8002 (1994), the Code of Practice for Earth Retaining Structures, states that crib walls 'are built of individual units assembled to create a series of box-like structures containing suitable granular free draining fill, to form a gravity retaining wall system. The units should be so spaced that the fill material is contained within the crib, is not affected by climatic changes and acts in conjunction with the cribwork to support the retained earth.'

This report reviews the use of crib walls and is arranged to follow the course of a project from the choice of using a crib wall, through design, to its construction and performance. The two main approaches to analysis, i.e. Monolith theory and Silo theory, are examined and analogies with infilled cofferdams are explored. The review of the methods used to assess the stability of crib walls and the loading on individual elements of the crib recommend the use of a limit value approach to design. A step-by-step approach to design is provided and modifications to take into account particular features of construction are considered. The fabrication of the crib elements and construction procedures are also covered.

BS 8002 (1994) proposes that crib walls should be designed as gravity mass walls, but this theory takes no account of the pressures generated within the crib or the forces sustained by the elements of the crib. The essential behaviour of such walls is therefore not captured and may cause designers to wrongly regard the design of the constituent elements and their interaction as being of little consequence. However the literature showed that there was a substantial amount of information available to enable an authoritative design guide to be written. This would be of value to clients, designers, contractors, and manufacturers and lead to cost-effective and safe structures - the key aspects affecting the public interest.
# CONTENTS

<table>
<thead>
<tr>
<th>ABSTRACT</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>2 FACTORS INFLUENCING THE CHOICE OF CRIB WALLING</td>
<td>2</td>
</tr>
<tr>
<td>2.1 Types of system</td>
<td>4</td>
</tr>
<tr>
<td>2.2 Project management</td>
<td>6</td>
</tr>
<tr>
<td>3 ECONOMICS</td>
<td>7</td>
</tr>
<tr>
<td>4 ANALYTICAL METHODS</td>
<td>8</td>
</tr>
<tr>
<td>4.1 Monolith theory</td>
<td>8</td>
</tr>
<tr>
<td>4.2 Silo theory</td>
<td>9</td>
</tr>
<tr>
<td>4.2.1 Janssen’s approach</td>
<td>9</td>
</tr>
<tr>
<td>4.2.2 Variations on Janssen’s approach</td>
<td>10</td>
</tr>
<tr>
<td>4.2.3 Airy’s approach</td>
<td>13</td>
</tr>
<tr>
<td>4.2.4 Coefficient of lateral earth pressure</td>
<td>14</td>
</tr>
<tr>
<td>4.2.5 Interaction of infill and crib skeleton</td>
<td>15</td>
</tr>
<tr>
<td>4.2.6 Relative movement between infill and crib skeleton</td>
<td>16</td>
</tr>
<tr>
<td>4.3 Analogy with infilled cofferdams</td>
<td>16</td>
</tr>
<tr>
<td>5 DESIGN AND ANALYSIS</td>
<td>17</td>
</tr>
<tr>
<td>5.1 Current codes and standards</td>
<td>17</td>
</tr>
<tr>
<td>5.2 Limit values</td>
<td>17</td>
</tr>
<tr>
<td>5.3 Interaction between the crib wall and retained backfill</td>
<td>18</td>
</tr>
<tr>
<td>5.3.1 Coefficient of lateral earth pressure</td>
<td>18</td>
</tr>
<tr>
<td>5.3.2 Point of application of earth pressure</td>
<td>19</td>
</tr>
<tr>
<td>5.3.3 Wall friction</td>
<td>19</td>
</tr>
<tr>
<td>5.4 Stability</td>
<td>21</td>
</tr>
<tr>
<td>5.4.1 General</td>
<td>21</td>
</tr>
<tr>
<td>5.4.2 Sliding</td>
<td>21</td>
</tr>
<tr>
<td>5.4.3 Overturning</td>
<td>23</td>
</tr>
<tr>
<td>5.4.4 Bearing</td>
<td>26</td>
</tr>
<tr>
<td>5.4.5 Overall stability</td>
<td>26</td>
</tr>
<tr>
<td>5.5 Design of elements</td>
<td>28</td>
</tr>
<tr>
<td>5.5.1 Lateral and axial loads</td>
<td>28</td>
</tr>
<tr>
<td>5.5.2 Forces at joints</td>
<td>32</td>
</tr>
<tr>
<td>5.5.3 Shape considerations</td>
<td>37</td>
</tr>
<tr>
<td>5.6 Special cases</td>
<td>37</td>
</tr>
<tr>
<td>5.6.1 Multi-cell walls</td>
<td>37</td>
</tr>
<tr>
<td>5.6.2 Terraced walls</td>
<td>40</td>
</tr>
<tr>
<td>5.6.3 Curved walls</td>
<td>40</td>
</tr>
<tr>
<td>5.6.4 Open-back walls</td>
<td>41</td>
</tr>
</tbody>
</table>
6 FABRICATION OF ELEMENTS

6.1 Materials
6.2 Specification for manufacture

7 CONSTRUCTION

7.1 Construction details
  7.1.1 Wall batter and cell shape
  7.1.2 Foundations
  7.1.3 Drainage
  7.1.4 Layout of elements
  7.1.5 Special features and additional requirements
7.2 Fill
7.3 Methods and specification

8 CONCLUSIONS

9 ACKNOWLEDGEMENTS

10 REFERENCES

11 BIBLIOGRAPHY

APPENDICES

Appendix 1 Derivation of Janssen's formula describing the distribution of vertical cell pressure
Appendix 2 Derivation of anchorage length of headers in open-back walls
ABSTRACT

This report reviews the design, construction, and performance of crib wall systems. The two main approaches to analysis, i.e. Monolith theory and Silo theory, are examined in detail and analogies with infilled cofferdams are also explored. The review of the methods used to assess the stability of crib walls and the loading on individual elements of the crib recommended the use of a limit value approach to design. A step-by-step approach to the design of crib walls is provided and modifications to take into account particular features of construction are considered. Methods and specifications for the manufacture of the crib elements and construction procedures are discussed, and general construction details are reviewed.

1 INTRODUCTION

BS 8002 (1994), the Code of Practice for Earth Retaining Structures, states that crib walls ‘are built of individual units assembled to create a series of box-like structures containing suitable granular free draining fill, to form a gravity retaining wall system. The units should be so spaced that the fill material is contained within the crib, is not affected by climatic changes and acts in conjunction with the cribwork to support the retained earth.’

Crib walling is an ancient method for retaining ground. For centuries primitive timber cribs have been constructed throughout Western Europe; Huder (1983) described fortifications built around 60 B.C. by the Gauls and Barrett (1981) reported the discovery of a Roman timber wharf in London that resembled a contemporary open-back crib wall. Much more recently precast concrete and metal elements have been used for their construction. Crib walls have been built in North America for over 100 years (Bovey, 1881); they were initially constructed from timber elements but reinforced concrete elements were introduced into railway works more than 75 years ago (Anon, 1918). The increase in the popularity of crib walls in the 1960’s was accompanied by research into aspects of analysis and performance with most of the work in Europe being undertaken in Austria and Germany.

This report reviews the use of crib walls and explores their economy, design, construction, and performance. The arrangement of this report follows the course of a project from the choice of using a crib wall, through design, to its construction. ‘Cellular walls’ - where a complete cell is formed from a single element - are not considered in this report.

Section 2 examines the various types of crib wall and their applications, advantages and disadvantages, as well as covering aspects of project management. The economy of crib walls is discussed in Section 3. Section 4 examines the theories used to analyze the behaviour of crib walls: the many variations in the form and application of crib systems make it inappropriate to follow a single approach. Section 5 reviews the more established methods of design: particular construction features and environmental factors are also considered. The fabrication of the crib elements and construction procedures are covered in Sections 6 and 7 respectively and the conclusions of the report are given in Section 8.
2 FACTORS INFLUENCING THE CHOICE OF CRIB WALLING

Crib structures were initially made with timber elements and so developed countries that had expanses of forestry saw their greatest use. Thus they have been employed for a wide range of applications in North America, New Zealand and Western Europe. Some of the advantages and disadvantages of crib walling are listed below:

Advantages

- High tolerance to differential settlements; this provides reasonably uniform foundation pressures thereby eliminating the need for specialist foundations, such as piles.
- Aesthetically pleasing with good provision for plant growth.
- Good sound absorbency.
- Robust and not easily disfigured by graffiti.
- Simple to erect with minimal requirement for skilled labour.
- High rates of construction.
- Elements are small and light thus avoiding the need for heavy cranage.
- Minimal disruption to construction due to adverse weather.
- Construction can be interrupted and restarted.
- No curing time required and so the full strength of a structure is available immediately.
- Structures can be dismantled or modified and the elements may be reusable.
- Good drainage properties.
- Structures can be built to a curved profile and accommodate changes in height.
- Relatively simple to incorporate anchors into the retained backfill.

Reinforced soil structures share some of the advantages listed above, but crib walls may require less excavation work for their construction and their elements may be less susceptible to corrosion.
Disadvantages

- The structures have a low tolerance to superimposed loads.
- The programming of site works must allow for the manufacture of the crib elements if stockpiles are insufficient.
- Interruption of the construction works may generate noticeable discontinuities along the line of a structure.
- Their performance is likely to be more susceptible to errors in analysis, design, and construction than other forms of construction.

Excessive differential movements may result in cracking of the crib elements; although this may not affect the short-term strength of the wall it may reduce durability. Their tolerance to differential movements is greatly reduced by the provision of multi-cells and by the addition of spurs on the retained side.

Applications

The combination of the above favour the following applications:

- Works with difficult access.
- Works with restricted working space.
- Where savings in construction time are important.
- Where future expansion or modification to a structure is likely.
- Emergency remedial works.
- Where other types of construction have a high risk of sustaining substantial damage, such as quasi-stable slopes.
- Noise barriers.
- Landscape structures.
- Landslide, rockfall and avalanche protection measures.
- Revetments and defences for erosion control.
- Cofferdams, wharves and agricultural cellars.

Crib walls have been built up to 15 metres high but they are usually better suited to heights of between 2 and 6 metres. According to Brandl (1982), wall heights greater than 15 metres may be achieved by incorporating prestressed ground anchorages or by terracing.
Crib walls can provide economies in construction costs, but it is also important to consider indirect savings resulting from: high rates of construction, minimal requirements for temporary works, smaller foundations, and their drainage capabilities. The appearance of crib walls can be improved by planting and so they may be chosen for aesthetic reasons.

2.1 Types of system

The most common type of crib comprises longitudinal beams (stretchers) and lateral ties (headers) made up into a series of regular shaped cells. The lower courses of high walls may be constructed from two or more rows of cells. Typical arrangements of single cell and double cell walls are shown in Figures 1 and 2 respectively. The dimensions of the cells can be changed by varying the lengths of the elements.

The second most common type consists of frames built up to form 'towers' with stretchers spanning between them to form 'fields'. A typical arrangement of a 'frame element' wall is shown in Figure 3. The frames can be filled with reinforced concrete and thereby become suitable for use with ground anchorages. With this type of wall the length of the stretchers is normally fixed and the size of the repeat unit varied by changing the dimensions of the frames. Frame element walls cannot be built with double cells nor constructed from timber elements without the fabrication of complex joints. Walls can be constructed from a series of towers, i.e. with no fields, but purpose-built frames are required for the ends of bays to ensure that the rows of elements are properly staggered.

Fig. 1 Typical arrangement of single cell crib wall
Fig. 2 Typical arrangement of double cell crib wall

Fig. 3 Typical arrangement of frame element crib wall
Other systems have been used with various forms of header, such as fish-tails or A-shapes, connected to facing units via dowels or lugs (Anon, 1953), but these have fallen out of usage. A novel system constructed from used vehicle tyres has been developed (Anon, 1977), but although the material costs were low the compaction of the fill around the tyres was labour intensive. A system constructed from 'single elements' has also been proposed; the elements in this system were connected by metal pins or dowels that slotted into channels and imparted some flexibility to the structure: details have been given by Crighton (1978) and by Ridout (1982).

2.2 Project management

Walls up to about 1.5 metres high and where the consequences of their collapse are not serious are usually designed and dimensioned according to the manufacturer's guidelines. For higher walls or where the consequences of failure are severe, then both collapse and serviceability limit states should be properly assessed. Crib wall manufacturers usually offer a complete design, supply and construction package.

The procedure for the procurement of crib walls for trunk road schemes in the UK is defined in the DOT Department Standard SD4 (MCHW 0.2.4). This recommends the engineer to issue an outline specification for the structure, containing all the information necessary for design but not to refer to a particular system - in this way tenderers are given the widest choice possible. An outline 'Approval in Principle' form should be included for structures higher than 1.5 metres. Tenderers then price, on a lump sum basis, to meet the specification. After award of the Contract, the contractor completes a detailed design and submits it for technical approval; following this the engineer adopts the structure and the Contract continues in the normal manner. Contractors are free to choose the type of structure that they judge to be the most suitable within the constraints of the outline specification and they must judge for themselves the extent of the design needed at the tender stage. The tender price is deemed to include for any measures necessary as a result of the engineer's check and the requirements of the Technical Approval Authority.

A conventional engineer-designed and contractor-supplied procurement process may be appropriate for non-proprietary systems.
3 ECONOMICS

The cost of a crib wall is dependent on a number of factors including the angle of the retained slope and the properties of the backfill. For comparative purposes, quotes were obtained from various manufacturers of crib walls to meet the following parameters:

- competent foundation soils,
- horizontal upper slope behind wall,
- fixed wall height,
- no surcharge load, and
- backfill having an angle of friction ($\phi'$) of 30° and zero effective cohesion.

The quotes were given per unit area of wall facade. The mean values for concrete and for timber crib cells for 'supply only' and 'supply and erection' are given in Table 1. The large difference in the costs of the 3.5 metre and the 7 metre high walls reflects the greater base width, i.e. the number of crib cells, that the higher wall requires for stability.

There was little or no difference in the unit rates for 500m² and 5000m² of facade area, suggesting that unit cost was largely independent of quantity. However when a small number of units are required, typically for less than 100m² of facing, the on-costs such as transportation and site mobilisation will form a higher proportion of the total cost.

Crib walls are commonly procured as self-contained packages with the price including design, manufacture, erection, and overheads. Comparison with an equivalent reinforced or mass concrete structure is often obscured by the different procurement process, whereby design and construction are distinct entities carried out by different parties so that some of the costs of the structure become hidden in the overall cost of the project. Nevertheless, estimated costs for equivalent reinforced concrete cantilever retaining walls are also given in Table 1: these estimates include allowances of 5 per cent for design and 15 per cent for construction preliminaries, overheads and profit.

<table>
<thead>
<tr>
<th>HEIGHT (metres)</th>
<th>CONDITION</th>
<th>CONCRETE CRIB WALL (mean values)</th>
<th>TIMBER CRIB WALL (mean values)</th>
<th>PLAIN FACED REINFORCED CONCRETE CANTILEVER WALL</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5</td>
<td>Supply only</td>
<td>58</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Supply and erection</td>
<td>126</td>
<td>115</td>
<td>100</td>
</tr>
<tr>
<td>7.0</td>
<td>Supply only</td>
<td>92</td>
<td>78</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Supply and erection</td>
<td>171</td>
<td>161</td>
<td>156</td>
</tr>
</tbody>
</table>

Table 1 Estimated costs (£/m²) for typical structures
(Figures exclude costs of excavation, backfilling, and temporary works)
The estimates in Table 1 suggest that, at least in the UK, crib walls may be more expensive than equivalent reinforced concrete cantilever walls. This conflicts with the commonly held view that crib walls are cheaper than more conventional retaining structures. It may be that crib walls are not cost effective for the case described above but are where other constraints apply. Crib walls are unlikely to be competitive on level green field sites, but their advantages over other structural forms make them attractive propositions in certain circumstances. In this respect, an important UK market for crib walls is for motorway widening works where lane space is at a premium and working space is often restricted. One of the more favourable aspects of crib walls is their appearance and this is likely to be the criterion which will have the greatest influence on their selection. The visual intrusion of crib walls can be softened by planting and so they are frequently favoured over other forms of construction, particularly in rural areas.

4 ANALYTICAL METHODS

The literature review identified two main approaches to the analysis of crib walls: Monolith theory and Silo theory. These are discussed in Sections 4.1 and 4.2 respectively: the analysis of infilled cofferdams is covered in Section 4.3.

4.1 Monolith theory

As its title suggests, in this theory the crib cell and its infill are assumed to act as a rigid homogeneous body. The wall is dimensioned as for a gravity structure of the same unit weight and having an effective wall width \((B)\): as defined in Figure 4, \(B\) is usually taken to be the distance between the outside faces of the stretchers. Monolith theory is simplistic but according to the 1985 German code, produced by the Transport and Road Research Association, it produces safe designs.

![Sectional view](header.png)

In poorly constructed walls, the formation of cavities within the fill may result in a low mean unit weight. Controlled infilling and compaction will prevent this and so the unit weight derived from the results of compaction tests is usually assumed.

At the onset of forward tilting or overturning, the infill may run out of the crib thereby reducing the weight that is effective in stabilizing the structure. If this is the case, then the
assessment of stability should be based on the downward force of the infill acting on the inside of the crib cell rather than the full weight of the infill. The magnitude of the wall friction developed between the infill and the skeleton of the cell is dependent on conditions within the crib.

4.2 Silo theory

The interaction between the infill and the crib skeleton is analogous to that between grain and their tall storage bins, or silos, and theories used to determine the pressures within grain silos have been adapted for the design of crib walls. Frictional forces generated between the infill and crib skeleton reduce the vertical earth pressures at an horizon to less than the equivalent geostatic pressures. This ‘arching’ of the infill between the walls of the cell is called the ‘silo effect’. It transfers part of the weight of the infill to the skeleton of the crib, thereby improving stability, and the pressures in the crib cell determine the forces on the individual elements.

4.2.1 Janssen’s approach

Most of the approaches for determining the cell pressures are based on that developed by Janssen, cited by Brandl (1980). This assumes a uniform distribution of vertical pressure across the cell. For infills with little cohesion, the vertical pressure \( p_{vc} \) within a silo is:

\[
p_{vc} = (\gamma - c'U/A)z_0(1 - e^{-\frac{z}{z_0}})
\]

where,

\[
z_0 = \frac{A}{U} \frac{1}{K \tan \delta_i}
\]

\( \gamma \) = unit weight of infill,
\( c' \) = effective cohesion,
\( U \) = internal perimeter of cell,
\( A \) = cross-sectional area of cell,
\( z \) = depth of cross-section below top of wall,
\( K \) = coefficient of lateral earth pressure for infill, and
\( \delta_i \) = angle of interface friction between infill and skeleton of the cell.

The product \( K \tan \delta_i \) is referred to as the silo parameter.

It can be seen from Figure 5 that with increasing depth the vertical pressure tends to a limiting value. Cohesion can have a significant effect on the cell pressure but it is normally ignored in design and for cohesionless infills the maximum vertical pressure is:

\[
p_{vc \ max} = \gamma \cdot z_0
\]
thus, \[ p_{vz\ max} = \frac{A}{U} \cdot \frac{\gamma}{K \cdot \tan\delta} \] (4)

A surcharge placed on top of the silo will not increase \( p_{vz\ max} \) but it will increase the vertical pressure near the top of the structure, thus the vertical cell pressure with surcharge (q) is, according to Schuster et al (1975):

\[
p_{vz} = \gamma z_0 (1 - e^{-\frac{z}{z_0}}) + q e^{-\frac{z}{z_0}}
\] (5)

Derivations of the above equations are given in Appendix 1.

4.2.2 Variations on Janssen's approach

The design manuals for Permacrib (undated) and Timbacrib (Pinex Timber Products Ltd, 1989), quote Reinbert's formula for estimating the vertical pressure within a crib cell:

\[
p_{vz} = \frac{\gamma z}{(z/A_d) + 1}
\] (6)
where;  
\[
A_1 = \frac{d_{\text{min}}(2d_{\text{max}} - d_{\text{min}})}{d_{\text{max}} \cdot \pi \cdot \tan \delta \cdot \tan^2(\frac{\pi - \phi'}{4})}
\]  

(7)

\[\phi'\] = angle of friction of the infill,
\[d_{\text{min}}\] = smaller internal dimension of cell, and
\[d_{\text{max}}\] = larger internal dimension of cell.

Janssen assumed a uniform distribution of vertical pressure across the cell but Brandl (1980) found that the distribution was asymmetric as shown in Figure 6. Openings in the face of the wall reduce the cell pressures in their vicinity, but surcharges and the pressures due to the retained backfill push the position of the peak value towards the front of the wall.

Fig. 6 Typical distribution of vertical cell pressure, after Brandl (1980)

Brandl (1980) found that as the height of the wall increased, the vertical cell pressures became increasingly higher than predicted by Janssen’s theory. This disparity was due to variations over the height of the wall of the degree of compaction of the infill, the lateral earth pressure coefficient (K), and angle of mobilised internal wall friction (\(\delta_i\)). The infill in the lower courses of a wall is often over-compactred thereby reducing the ‘silo effect’. The increased cell pressures can be accommodated by varying the ‘silo parameter’ with depth; thus ‘K.tan \(\delta_i\)’ in the Janssen formulae is replaced by ‘\(x.K.tan \delta_i\)’ where \(x\) is a reduction factor derived from model tests. The variation of \(x\) with height is shown in Figure 7. The expression for the vertical cell pressure for a cohesionless fill then becomes:

\[
P_{vz} = \gamma \cdot z_0 \cdot (1 - e^{-z_0})
\]  

(8)
Brandl (1984a) suggested that compaction operations may lead to increased lateral pressures in the top two metres or so of a structure, and the lateral earth pressure coefficient (K) for calculating horizontal pressures in this zone could therefore lie between $K = 1.5K_0$ and $K = 2K_0$, where $K_0$ is the coefficient of at-rest earth pressures. This was adopted in the 1985 German specification for the design and construction of crib walls and embankments. Mean cell pressures in excess of those predicted do not cause concern when designing the elements of a crib, because pressures decrease near the front face due to openings in the wall and the loading on the rear stretchers is more onerous with lower cell pressures. On the other hand, overall stability is affected by the magnitude of the silo effect within the cell and an increase in cell pressure implies that less of the weight of the infill is transferred to the crib skeleton.

According to Kovacshazy (1965), it is essential for economic design that the proportions of the cell induce a silo effect: pressures within the cell may be more dependent on the proportions of the cell than the properties of the infill. It can be seen from Figure 5 that the wall height must be greater than $z_0$ for any appreciable silo effect to occur. A low value of $z_0$ can be obtained by using an infill with a high angle of friction and an appropriate cell shape factor (m) as defined in Figure 8. Silo conditions can be generated for values of m of up to 3, but it is usually best to ensure that m ≤ 1.

\[
z_0 = \frac{A}{UxK \tan \delta_i}
\]  

\[Fig. 7 \text{ Reduction factor for modified silo theory, after Brandl (1980)}\]

\[Fig. 8 \text{ Definition of shape factor (m)}\]

where;  

\[z_0 = \frac{A}{UxK \tan \delta_i}\]
The situation inside the crib cells is more complex than assumed by Janssen's theory. The cell pressures are more accurately determined by the above modified version of Janssen's theory but, for simplicity, the original formulae are more useful for the purposes of design as the inaccuracies err on the conservative side.

4.2.3 Airy's approach

An alternative theory for grain silos was presented by Airy (1898), based on the stability of sliding wedges. As grain silos are usually large the original theory took no account of 3-D effects. However, because crib structures are affected by 3-D effects, Airy's formula has been modified to incorporate a wedge shaped structure as shown in Figure 9.

![Diagram of crib wall and surcharge dimensions](image)

Fig. 9 Dimensions of wedge-shaped surcharge for use with Airy's (1898) formula

The comparison, undertaken by Schuster et al (1975), given in Figure 10 shows that the approaches of Airy and Janssen give similar but different results. For this comparison;

\[
\gamma = 20 \text{ kN/m}^3, \quad \phi' = 45^\circ, \quad c' = 0,
\]

Surcharge slope = 1 (vertical) : 1.5 (horizontal),
Crib dimensions = 2.2 x 1.4 metres,
\(K = 0.33\), and
\(p_{wz} = p_{hw}/K\).
4.2.4 Coefficient of lateral earth pressure

The lateral earth pressure coefficient (K) acting within a crib cell is generally taken as the at-rest value \( K_0 \), where:

\[
K_0 = 1 - \sin \phi'
\] (10)

Other values of K have been proposed: Kovacshazy (1965) suggested that active conditions are present within the crib cell; Schuster et al (1975) in their comparison of Airy’s and Janssen’s formulae, and Jumikis (1971) in the design of crib cofferdams, used Krynine’s coefficient that was developed from consideration of vertical shear in cellular cofferdams; Yan (1983) suggested the use of Janssen’s theory with a value of K of 0.5. According to Schlosser (1990) conditions may vary from active conditions near the face of the wall, due to the openings making compaction difficult, to passive conditions near the back of the wall due to pressures exerted by the backfill. Nonetheless the results of model tests reported by Brandl (1980) and the investigations by Gerabek, referred to by Yan (1983), confirm the generally accepted view that at rest conditions prevail within a crib cell.

Excessive compaction will increase the value of K and the unit weight of the fill (\( \gamma \)). The increase in K will reduce the vertical cell pressure (\( p_{\text{v}} \)) but an increase in \( \gamma \) will increase it: of the two variables K has more effect on the pressures within a cell.
4.2.5 Interaction of infill and crib skeleton

The frictional forces mobilised between the infill and the elements of the cell are dependent upon the relative movements between them and the particle size of the infill. Göbel (1969) found that the angle of wall friction ranged between 0.7\(^\circ\) for sand infills (<2mm particle diameter) and 0.5\(^\circ\) for particularly coarse infills (>60mm particle diameter). Brandl (1980) recommended a value of 0.4\(^\circ\) and this was incorporated in the 1985 German code for crib walls and embankments.

The internal frictional forces can significantly affect the pressures in the crib. Even a low value of cohesion can substantially increase the silo effect due to the increased shear forces that can be developed between the infill and the crib skeleton.

4.2.6 Relative movement between infill and crib skeleton

The conditions inside the crib depend greatly on the movement between the skeleton and infill. Where a crib wall is founded on a continuous concrete base then the infill will settle with respect to the skeleton, and the internal wall friction will act downwards on the framework; this is known as the 'active' state. The weight of the infill, being partly transferred to the skeleton, results in a convex pressure distribution across the cell as shown in Figure 11(i). If, however, the base slab is omitted and the soils beneath the wall are compressible then the crib may settle relative to the infill. In this case the frictional forces act upwards on the skeleton thereby partly supporting its weight. The shape of the pressure distribution will now be concave across the cell with the peak vertical cell pressures being in excess of the value calculated using Janssen's equations and the mean vertical cell pressure being higher than the equivalent geostatic pressures; this is known as the 'passive' state and is illustrated in Figure 11(ii).

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Fig. 11 Comparison of vertical cell pressure distributions for 'active' and 'passive' states, after Brandl (1980)
Passive conditions reduce overall stability and generate higher loadings on the crib elements. Foundation requirements depend on soil conditions and it is impossible to provide any general rules. It has been suggested that a continuous concrete foundation becomes a necessity once a wall exceeds a certain height - suggestions range from 2 to 6 metres. Regardless of whether the 'passive' state occurs or not, settlement of the crib will reduce the downward movement of the infill relative to the skeleton - this in turn reduces the mobilized wall friction and the silo effect leading to an increase in the pressures within the cell.

Following the failure in 1979 of a crib wall at Wintertur-Wulflingen, the cell pressures and deflections of the reconstructed wall were monitored and reported by Kuhn (1983). The measurements indicated higher cell pressures and lower nodal forces than predicted using Janssen's theory. Less silo effect implies a lower retaining capability and indeed outward movements of up to 27mm were recorded in the first 10 months following the end of construction. A 'passive' condition was generated with frictional forces acting upwards on the walls of the cell and with the resulting cell pressures being greater than the equivalent geostatic pressures. This condition resulted from the combination of a relatively flexible wall system and heavy compaction of the infill.

4.3 Analogy with infilled cofferdams

Infilled cofferdams are cellular structures that rely to some extent upon the interaction between the infill and the walls of the cell for stability. The theories developed for cofferdams may therefore have some application to the design of crib walls.

Three mechanisms describing the failure of cofferdams have been proposed: (i) Terzaghi's (1944) 'vertical shear', (ii) Cummings' 'horizontal shear', and (iii) Hansen's method. As shown in Figure 12 they all involve some form of internal shearing. Methods (i) and (ii) have a proven use in the field but method (iii) was considered to be the soundest by Babtie Geotechnical (1988).

The shear strength of the crib elements introduces a large resistance to failure and so it would seem at first sight that the mechanisms are not appropriate to the design of crib walls. However the failure of a crib wall at Canniesburn Road, Glasgow indicated that these internal shear mechanisms could be used to model ultimate limit states. According to the Babtie, Shaw and Morton report (1991a) the mechanism of failure was similar to the vertical shear mechanism proposed by Terzaghi in that a concentrated force under the front stretchers caused a progressive shearing of the headers, however the failure plane occurred just behind the front joints rather than down the centreline as in Terzaghi's mechanism.
An ‘at rest’ earth pressure coefficient based on Terzaghi’s ‘vertical shear’ mechanism was devised by Krynine (1944) for use in the design of crib walls:

\[ K = \frac{\cos^2 \phi'}{2 - \cos^2 \phi'} \]  

(11)

Jumikis (1971) also considered its use for crib cofferdams. Whatever the at rest earth pressure coefficient used with the Terzaghi’s mechanism, as the angle of internal friction increases the factor of safety for the cofferdam increases to a peak and then decreases. As pointed out by Esrig (1970), this gives the inconsistency that as soil strength increases the strength of the structure decreases.

5 DESIGN AND ANALYSIS

5.1 Current codes and standards

In the UK, BS 8002 (1994) deals with the design and construction of all types of retaining walls. It states that crib walls should be designed as gravity mass walls using a partial factor of safety approach; however it gives no guidance on the distribution of forces within the crib cell.

In 1985, the Transport and Road Research Association in Germany published ‘Instruction for the design and construction of crib walls and embankments’: this code refers to a number of reports and papers by Brandl.

The American Railway Engineers Association published the ‘Manual for Railway Engineering’ in 1987. This included a section on the design and construction of crib walls constructed from timber, concrete, or metal elements, but as with BS 8002 (1994) the approach was rather simplistic and took no account of the distribution of forces within the structure.

5.2 Limit values

The pressure distribution within a crib cell is complex and difficult to predict with accuracy, particularly as systems and installation conditions vary. Thamm (1986 and 1987) reported that the degree of compaction, differential movement, and the effects of frost can all influence the conditions within a cell. Thus to ensure safety it is necessary to consider the conditions that give the maximum loads on the crib elements and the minimum levels of overall stability. These conditions are determined by the limiting values for the cell pressure and for the silo effect.

The forces acting on the crib elements and the sliding resistance through a wall are both dependent on the degree of silo effect. The maximum forces between the elements could be determined by taking the upper limit for the silo effect which corresponds to the situation where the full weight of the infill is transferred to the skeleton of the cell. According to the 1985 German code, the lower limit can be taken to be when 20 per cent of the weight of the infill is transferred to the crib skeleton.
The application of limit values in design is described further in Sections 5.4 and 5.5.

5.3 Interaction between the crib wall and retained backfill

5.3.1 Coefficient of lateral earth pressure

The assessment of the interaction between the retained backfill and the crib wall is central to the design process. Lateral earth pressures generated by the backfill are usually assumed to correspond to active conditions with calculations based on Coulomb's theory being considered adequate. However, there are different definitions of the active earth pressure coefficient ($K_a$). In most sources $K_a$ is the ratio of the horizontal and the vertical earth pressures, but some, such as Smith (1982), give $K_a$ as the ratio of the total earth pressure inclined to the normal to the back face of the wall (by the angle of wall friction) and the vertical pressure. Wall friction is usually considered in the design of crib walls and so it is important to ascertain which definition of $K_a$ is being used.

When backfilling is undertaken during construction then according to Brandl (1980), and as shown in Figure 13, the width of the active wedge behind a crib wall is narrower than predicted by theory.

![Diagram of section through crib wall showing active wedge, theoretical sliding planes, transition zone, and equilibrium condition](image)

Fig. 13 Comparison of actual and theoretical active zones, after Brandl (1980)

If the movement of the wall is restricted in any way, such as by the installation of ground anchorages, or relatively rigid wall systems are used, then the backfill may not achieve active conditions and it will be necessary to assume at rest pressures. According to Brandl (1982), the varying stiffness along the length of frame element walls can generate backfill pressures and base pressures at the towers up to 2.5 times those at the fields.

It is necessary to differentiate between dimensioning the wall and designing the crib elements. It would seem sensible to check the stability of the wall using active earth pressure coefficients. However, if the wall did not move (tilt or slide) with the placement of the backfill then pressures corresponding to at rest conditions may be sustained and the crib elements should be designed to resist such pressures.

Compaction operations can lead to pressures in excess of at rest pressures, particularly at the top of a wall, but such pressures will tend to be dissipated through the gaps in the framework.
5.3.2 Point of application of earth pressure

In accordance with classical theory, the resultant thrust from the backfill could be taken to act at a point one third of the wall height up from the base of the wall.

A mechanism of failure may have little if any influence on the magnitude of the thrust from the backfill, but the line of action of the resultant thrust may vary. During sliding failure or rotation about the top of the wall the point of application will move upwards. This will also tend to happen when the width of the supported backfill is narrow; when the supported slopes are steep (i.e. greater than 35°); where the foundation or supported soils are weak; and also where ground anchorages are installed. For all these situations Brandl (1980) and Thamm (1986) recommended that the point of application of the resultant earth pressure be taken to act at the mid-height of the wall.

The failure of a crib wall constructed at Canniesburn Road, Glasgow exemplified such problems; details of the failure were reported by Babtie, Shaw and Morton (1991a). A lack of control over the extent of the excavation (and the consequent narrow backfilled zone), together with an overly steep surcharge slope, an excessive wall height, and an unyielding base, increased the likelihood of at rest pressures being developed at this site. This combination of factors led to the progressive failure of the plain concrete headers. If at rest pressures are assumed, then the factors of safety for sliding and overturning are unacceptable with the resultant thrust at the base falling outside the middle quarter.

Moving the point of application of the resultant thrust has more effect on stability than variations in the frictional force on the back of the wall or the mean density of the infill, the latter being insignificant. The inclination of the wall and cohesion of the infill can also have a substantial effect on the stability of a wall but, for simplicity walls are often designed as vertical structures, with a consequential in-built safety margin, and as cohesion may be only a temporary phenomenon it is good practice to ignore it.

5.3.3 Wall friction

The interface friction mobilized between the crib wall and the retained backfill is dependent on the ratio of the areas of soil and concrete on the back face. The larger the area of concrete then the lower the mobilized angle of interface friction (\(\delta_u\)). BS 8002 (1994) and DIN 1055 (1963) recommend a \(\delta_u\) value of \(\%\phi'\) - this is about the value usually allowed for reinforced soil walls. Brandl (1980) found that the value of \(\delta_u\) varied from 0.75\(\phi'\) to \(\phi'\) in the manner shown in Figure 14; this relation was incorporated into the 1985 German code. Thamm (1986 and 1987) found from a series of model tests that the relatively small displacements following construction meant that active conditions were not attained in the backfill and so the value of \(\delta_u\) could be as low as 0.43\(\phi'\). However in one test he showed that differential settlement increased the mobilized value of \(\delta_u\) from 0.6\(\phi'\) to 0.8\(\phi'\).

It is essential to determine the direction in which the frictional force acts. BS 8002 (1994) recommends that the value of \(\delta_u\) be taken as zero if there is a tendency for the wall to move downwards with the backfill. In extreme cases, settlement of the structure may result in a downward movement relative to the retained soil and so the angle of wall friction would be reversed - earth pressures would then act upwards thereby reducing greatly the stability of the
Tschebotarioff (1965) analyzed the failure of a crib wall where movements resulted in the frictional force acting upwards on the back of the wall. The calculated factor of safety against sliding with the frictional force acting downwards was 2.82, but when the direction of wall friction was reversed the calculated factor of safety was 0.95. The effect of this reversal on stability is illustrated in Figure 15.
The provision of adequate foundations to prevent settlement of the crib relative to the retained ground is of key importance.

5.4 Stability

5.4.1 General

According to Schuster et al (1975) the dimensions of a crib wall are usually limited by stability considerations rather than the strength of the crib elements. The main loads on a crib wall derive from the self weight of elements, the soil pressures generated by both the backfill and infill, and any surcharge loads.

Most design documents recommend that crib walls be dimensioned as a gravity retaining structure using Monolith theory. Recognising that the inherent flexibility of crib walls increases greatly their stability against overturning, sliding and bearing, Brandl (1985) proposed that lower factors of safety than adopted for rigid gravity walls be allowed when checking stability using Monolith theory.

BS 8002 (1994) states that crib walls can support surcharged slopes, but foundations for buildings or other structures should not impose a loading on the wall or its foundation. To allow for future excavations, it is prudent not to rely on passive resistance being generated by the soil in front of the lower courses of the wall.

According to the American Railway Engineers Association's design manual produced in 1987, the wall should be designed in lengths not exceeding 33 metres and, even though drainage is to be provided, hydrostatic pressures should be allowed for. One of the main advantages of crib walls is their essentially 'free-draining' nature and it is usually considered unnecessary to allow for hydrostatic pressures. However 'free-draining' infill and a drainage system must be specified and the possible silting-up of drainage channels with the subsequent generation of water pressures should be considered.

Some product manuals give design charts and nomographs for dimensioning walls but it is important to check when using these that the stated conditions are applicable to the site and structure under consideration. To reduce trial-and-error calculations it has been suggested (Anon, 1970), that the overall dimensions are first determined by checking stability against overturning.

5.4.2 Sliding

The results presented by Brandl (1985), from both model tests and field investigations, show that sliding is most likely to arise at the base of a wall or at changes at a wall section. Nonetheless stability against sliding must be ensured at all levels through a wall.

If a continuous concrete base is provided then the crib wall can be treated as a gravity structure comprising the weight of the wall and the infill between front and back faces. Manufacturers' literature, such as those provided by Pinex Timber Products Ltd (1989) and ICB (undated), as well as Brandl (1980), recommend a factor of safety of 1.5 against sliding. Resistance to sliding can be increased by providing a backward sloping base or by providing...
a shear key. If a continuous concrete slab is not provided but the wall is of the type where supports, for example sleepers, are spaced at intervals beneath the stretchers then some of the weight of the infill will be transferred directly to the subsoil. The calculation for sliding should take account of this transfer of load and also the different coefficients of friction between the base and the subsoil, and the infill and the subsoil.

The resistance to sliding at some higher horizon through the wall is dependent on the extent of the silo effect and so a limit value approach should be adopted. The use of mean values of $\phi'$ and $\delta_i$ are likely to over-estimate the shear resistance through a section because of the concentration of load through the crib skeleton. The sliding resistance of the infill and the crib elements should therefore be considered separately. In determining the resistance to sliding, the frictional resistance generated between elements should be considered but no account should be taken of any interlocking of the elements.

The factor of safety against sliding should be determined using the limit value producing the least sliding resistance and it is good practice to ignore the stabilizing effects of cohesion. The upper limit corresponds to the maximum possible silo effect where all the weight of the infill is transferred to the crib skeleton. According to the 1985 German code, the lower limit is equivalent to 20 per cent of the weight of the infill being transferred to the skeleton. Accordingly the sliding resistance $(S)$ at any section is, with reference to Figure 16, the lesser of the following:

$$S = (W_1 + W_2 + P_v) \cdot \mu_c$$

$$S = (W_1 + 0.2W_2' + P_v) \cdot \mu_c + 0.8W_2'\mu$$

where;

$W_1$ = weight of crib skeleton,
$W_2$ = weight of infill,
$P_v$ = vertical component of resultant thrust from backfill,
(The above forces are per metre run of the wall)
$\mu_c$ = coefficient of interface friction between the crib elements, and
$\mu$ = coefficient of friction of infill ($= \tan \phi'$).

For stability $S \geq P_h/F$, where $P_h$ is the horizontal component of the resultant thrust from the backfill, per metre run, and $F$ is the required factor of safety.

The selection of the value for the coefficient of friction between crib elements ($\mu_c$) should take account of any bedding materials placed between them. Such materials are used to eliminate points of high stress and to maintain the level of the crib. Strips of roofing felt or damp proof course can be used, but these have relatively low frictional coefficients whereas layers of mortar will have at least the same coefficient of friction as a concrete to concrete interface. Lacher (1915) reported that the coefficient of friction for concrete to concrete interfaces on block walls ranged between 0.53 and 0.75.
Kuhn (1983) reported a sliding failure of a crib wall at Wintertur-Wulfsingen: in this wall the headers and stretchers had been extensively cracked and spacer blocks had been broken. The wall was designed with an assumed value of 0.71 for $\mu$, but the actual value for the roofing felt spacers lay between 0.1 and 0.2. Furthermore, variations in the size of the elements introduced some uncertainty into the distribution of stress within the crib, and the shear strength of the infill had been reduced by contamination with topsoil and by inadequate compaction.

The New Zealand Concrete Construction document on crib walling (Anon, 1970), provided guidelines for designing walls subject to earthquake loading: for sliding failure it suggested increasing the coefficient of lateral earth pressure by 40 per cent and reducing the factor of safety to 1.3.

5.4.3 Overturning

Stability against overturning should be checked for the structure as a whole and at any horizontal section through the wall. The simplest way of calculating stability against overturning is to assume the wall behaves as a monolith and to consider overturning about the front of the wall. Using the assumption that the full weight of the infill can be used to determine stability against overturning, most design manuals suggest that a factor of safety of 1.5 is adequate. The results of model and field experiments show that crib walls are considerably more resistant to overturning than Monolith theory would predict. The inherent flexibility of crib walls can induce a transfer of force away from the front to the rear joints and in light of this Brandl (1980) suggested that a factor of safety close to unity, i.e. 1.1, may be appropriate. If the wall system was relatively inflexible or if concrete was used as the infill then the factors of safety applicable to conventional gravity structures should be adopted.

If a trapezoidal pressure distribution is assumed at a section, then, to ensure that joints between elements do not open, the full cross-section should be in compression and the resultant force should pass within the 'middle third' of the wall. This is equivalent to limiting the eccentricity of the thrust to one sixth of the width of the wall, i.e. $e \leq B/6$. Brandl (1980) proposed that only 75 per cent of the cross-section need be involved in the transfer of force,
i.e. \( e \leq \frac{B}{4} \), but this does imply opening of joints between the elements of the crib. A restriction on the position of the resultant force is equivalent to enforcing a factor of safety against overturning, the magnitude being dependent on the allowable eccentricity and the frictional force developed on the back of the wall. For example, if the frictional force is ignored then an eccentricity of \( \frac{B}{6} \) corresponds to a factor of safety of 3.

Brandl (1980) proposed a method to check stability against overturning based on the opening up of joints between the crib elements, and this is perhaps a more realistic model of the mechanism of failure. The full weight of the infill should perhaps not be used to determine the resistance to overturning, but rather that which is transferred to the crib skeleton through frictional forces. The transferred weight can be taken as the difference between total weight of the infill and the product of the mean vertical cell pressure and the cross-sectional area of the cell. Using Janssen's approach described in Section 4.2.1, the weight of infill \( (W) \) transferred to the skeleton is thus:

\[
W = A \cdot \gamma \cdot [z - z_0 \cdot (1 - e^{\frac{-z}{b'}})]
\]

Alternatively, because of the uncertainty of the silo effect, a lower limit of 20 per cent of the weight of the infill can be taken to resist overturning.

The forces (per metre run of wall) to be considered are shown in Figure 17 and the factor of safety \( (F) \) can be calculated as follows:

\[
N'_B = \frac{W_i}{2} + \frac{W}{2} + W_B + P_v
\]

\[
N''_B = \frac{P_v \cdot h_v}{b'}
\]

\[
F = \frac{N'_B}{N''_B}
\]

where;

- \( N'_B \) = force at the rear joint due to vertical loads,
- \( N''_B \) = force at the rear joint due to horizontal loads,
- \( W_i \) = weight of units,
- \( W \) = wall frictional force due to silo pressure,
- \( W_B \) = weight of column of infill between the rear stretchers,
- \( P_v \) = vertical component of resultant thrust from backfill,
- \( P_h \) = horizontal component of resultant thrust from backfill, and
- \( h_v \) = point of action of resultant earth pressure on the back face of the wall for the level considered.
Continuous slabs could be provided across the cells to ensure that the full weight of the infill is utilised in resisting overturning. This arrangement, shown in Figure 18, is an expensive solution and is also likely to produce higher cell pressures and higher forces at the joints in the skeleton.
A backward inclination of the wall reduces the tendency to overturn, but normal practice is to design an inclined wall assuming it to be vertical thus introducing an additional factor of safety.

The New Zealand Concrete Construction document (Anon, 1970) recommends that when designing crib walls to withstand seismic loading, the coefficient of earth pressure should be increased by 40 per cent but the factor of safety against overturning could be reduced.

5.4.4 Bearing

The usual method of determining foundation pressures is to treat the wall as a gravity structure and assume a trapezoidal distribution of pressure beneath the base. Current practice requires that the peak pressure under the base should be less than a third of the ultimate bearing capacity of the subsoil, i.e. a factor of safety of 3, and the resultant thrust should act within the middle third of the base - this is in accordance with the design of conventional retaining walls. The inherent flexibility of crib walls gives a reasonably uniform distribution of pressure at their base and so a lower factor of safety and possibly smaller foundations would seem appropriate: Brandl (1980) recommended a factor of safety of 1.5. The distribution of pressure is however dependent on the flexibility of the wall, if a relatively inflexible system was used, one for example that had concrete infill, then a factor higher than 1.5 would be appropriate. In frame element walls the base pressures under the stiffer frames can be up to 2.5 times greater than under the more flexible fields.

A trapezoidal distribution provides an estimate of the peak pressure but the actual distribution may be quite different. For example, the base pressure at the heel of an inclined wall may be greater than that at the toe, and so providing foundations to the toe and not the heel may lead to differential settlement that will damage the wall, see for example Tschebotarioff (1965).

In considering bearing failure during seismic loading, the New Zealand Concrete Construction document (Anon, 1970) recommends an increase in the coefficient of earth pressures of 40 per cent; it also allows the maximum base pressures to be up to 75 per cent of the ultimate bearing capacity of the soil and the mean pressures to be up to 50 per cent of the ultimate bearing capacity. An alternative approach was given in the ICB literature (undated); this recommended that the resultant force, taking account of seismic loading, should remain within the middle half of the structure.

5.4.5 Overall stability

The stability along slip surfaces passing beneath the structure, or partly through the structure, should be checked. For the latter case the sliding resistance of the wall should be determined using a limit value approach as discussed previously. Where applicable hydraulic pressures should be taken into account when checking stability.

The critical slip surface through a crib wall on the A12 at Capel St. Mary was not identified at the design stage. According to the report by Babtie Geotechnical (1991b) into the failure, the slip surface seemed to pass along the interface between the backfill and the in situ glacial clay at the back of the structure and then through the wall some distance up from the base.
The postulated slip could be approximated by a multi-linear wedge as shown in Figure 19. The frictional resistance that could be developed along the interface between the backfill and the glacial clay was low because:

- the interface was steep,
- there was no benching along the interface,
- the glacial clay was softened during construction.

The disturbing force was also increased by saturation of the backfill. The headers in the crib became overloaded and although yielding allowed a redistribution of the forces, and some temporary gain in stability, creep movements led to collapse. Details of the investigation of the collapsed wall and its subsequent reconstruction were provided by M'Kenzie (1993).

---

According to BS 8002 (1994) crib walls are unsuitable for supporting earthworks that are liable to slip, but Brandl (1984b) gave the opposite view. Crib walls are more susceptible to superficial damage than conventional gravity walls but are much less prone to collapse. Moreover it is relatively simple to repair a damaged but serviceable crib wall compared to, for example, a toppled gravity wall. Due to their flexibility, crib walls can withstand very large deformations without collapsing. With severe deformation, the crib wall behaves like a truss, with tensile forces being transmitted by the headers and compressional forces transmitted diagonally through the infill between the front and rear stretchers. Continuous vertical joints in a crib wall allow movement and help to prevent excessive cracking. Crib
walls constructed on quasi-stable slopes can often be strengthened by anchors, particularly frame element walls.

5.5 Design of elements

5.5.1 Lateral and axial loads

The pressures from the infill and the backfill impose axial tension, bending, shear, and torsional forces on the elements. Because the loads in the elements are largely dependent on the magnitude of the silo effect within the crib cell, it would seem appropriate to determine limit values for these pressures.

Brandl (1980) proposed that the elements be designed according to the modified Silo theory, as described in Section 4.2.2, with the loads on the elements as defined in Figure 20. The headers are deemed to be loaded symmetrically and so are not designed to resist lateral loads. This limits the difference in height of infill that can be tolerated between adjacent cells. If the weight of fill lying directly above the front stretchers is ignored and the maximum possible arching effect between stretchers is assumed, then the difference in the pressures due to the infill and the backfill is deemed to be borne entirely by the rear stretchers. If the wall does not move or deform then the earth pressures applied by the backfill will correspond to at rest conditions: in the short-term, arching between the elements may prevent movement of the backfill. Accordingly, the rear stretchers should be designed to withstand at rest earth pressures from the backfill.

Brandl (1980) proposed that the elements be designed according to the modified Silo theory, as described in Section 4.2.2, with the loads on the elements as defined in Figure 20. The headers are deemed to be loaded symmetrically and so are not designed to resist lateral loads. This limits the difference in height of infill that can be tolerated between adjacent cells. If the weight of fill lying directly above the front stretchers is ignored and the maximum possible arching effect between stretchers is assumed, then the difference in the pressures due to the infill and the backfill is deemed to be borne entirely by the rear stretchers. If the wall does not move or deform then the earth pressures applied by the backfill will correspond to at rest conditions: in the short-term, arching between the elements may prevent movement of the backfill. Accordingly, the rear stretchers should be designed to withstand at rest earth pressures from the backfill.

Fig. 20 Lateral loading on elements, after Brandl (1980)
For economy, safety, and for ease of construction, the front and rear stretchers are usually interchangeable, so their design should be based on the more onerous load case - usually that for the front stretcher. If the silo effect was low or the infill was over-compacted, then the mean cell pressure would be greater than anticipated and it is possible that the elements would be overloaded. However, because of the openings between the stretchers, the cell pressures close to the front of the crib wall will be lower than predicted by Silo theory - they may correspond to active earth conditions. If the lateral loading on the rear stretchers was assumed to be the difference between the pressures exerted by the backfill and the pressures in the cell, then an increase in the cell pressure would reduce this loading. Thus it seems reasonably safe to calculate the cell pressures using the modified Silo theory proposed by Brandl (1980), or possibly Janssen's formulae.

The cell pressures induce bending and shear forces in the headers. The headers will also be subject to an axial tension due to the thrust from the cell pressure acting on the stretchers being transferred to the headers by friction or interlocking or both of these. The magnitude of the axial tension (T) can be derived, as shown in Figure 21, from the relation:

\[ T = \sigma_{th}a(d_1 + d_2) \]  

(18)

where; \( \sigma_{th} \) = horizontal component of the cell pressure, 
\( a \) = horizontal distance between headers, 
\( d_1 \) = depth of stretchers, and 
\( d_2 \) = vertical distance between stretchers.

Based on the results of experiments, Göbel (1969) proposed the following:

\[ T = 0.75.\sigma_{th}.a.(d_1 + d_2) \]  

(19)

Support for this expression derives from the fact that the pressures from the backfill will induce a compression in the headers thereby reducing the maximum possible tension. This effect is illustrated in Figure 22.
In determining the loads on the elements of a crib, Brandl (1980) assumed that arching within the fill occurs so that the pressures from the fill lying between the stretchers act on the stretchers. The relation between the size and spacing of the stretchers and the arching effect, as described in the 1985 German code, is shown in Figure 23. The maximum possible arching effect occurs when:

$$\frac{d_1}{(d_1 + d_2)} \geq 0.4$$  \hspace{1cm} (20)

Typical values of the ratio $d_1 / (d_1 + d_2)$ vary from less than 0.3 up to about 0.6, but for safe design the maximum possible arching effect should be assumed.

Given the difficulty of assessing the stresses within a crib wall, the elements should be insensitive to loading conditions that are only marginally different from those assumed in design. Such robustness is particularly important when there is limited experience of the particular crib wall system. As the forces in the elements cannot be determined accurately, then it is appropriate to base design on limit values derived from consideration of the likely worst case loading. The limit values proposed in the 1985 German code and also by Thamm (1986) are listed in Tables 2, 3 and 4 with the loading arrangements being shown in Figures 24, 25 and 26 respectively.
Minimum Value | Maximum Value
--- | ---
$\sigma_{ih} = p_{hz}$ (where $p_{hz} = K_0 p_v$) | $\sigma_{ih} = p_{hz}$ (where $p_{hz} = 2K_0 p_v$ for top 2 metres of wall but elsewhere, $p_{hz} = K_0 p_v$)
| or |
| $\sigma_{ih} = p_{hz} (d_1 + d_2)/d_1$ (where $p_{hz} = K_0 p_v$) |
| or |
| $\sigma_{ih} = 0.5p_{vz}(d_1 + d_2)/d_1$ |

$\sigma_{iv} = \sigma_{ih} \tan \delta_i$ (where $\sigma_{ih}$ is as above) | $\sigma_{iv} = \sigma_{ih\text{ max}} \tan \delta_i$ (where $\sigma_{ih\text{ max}}$ is the maximum value of $\sigma_{ih}$ given above)

$\sigma_0 = \gamma d_2$ | $\sigma_0 = \gamma d_2$

Table 2 Limit values for determining loading on rear stretchers

Minimum Value | Maximum Value
--- | ---
$\sigma_{ih} = 0$ | $\sigma_{ih} = \sigma_{ih\text{ max}}/2$ (where $\sigma_{ih\text{ max}}$ is the maximum value of $\sigma_{ih}$ for the rear stretchers defined above)

$\sigma_{iv} = 0$ | $\sigma_{iv} = \sigma_{ih} \tan \delta_{ih}$ (where $\sigma_{ih}$ is as above)

$\sigma_0 = \gamma d_2$ | $\sigma_0 = \gamma d_2$

Table 3 Limit values for determining loading on front stretchers

Minimum and Maximum Values

| $\sigma_{ih}, \sigma_{iv}, \sigma_0$ | As for rear stretchers, taking into account different dimensions of the elements |

Table 4 Limit values for determining loading on headers
All combinations of the limit values should be evaluated to determine the most onerous loading condition on the elements.

The stresses in the elements resulting from stacking or transportation may be greater than those experienced in the constructed wall and so where necessary the design should ensure that such stresses are acceptable.

5.5.2 Forces at joints

Brandl (1980) found by experiment that the bearing forces at joints ranged between values determined from a vertical pressure of $p_{ve}$ derived from Janssen's approach, and a vertical pressure of $0.5(p_{ve} + \gamma z)$, where $\gamma z$ is the geostatic pressure. A comparison of the variation of these pressures with depth is shown in Figure 27. (The measured values usually fell below the line midway between the relations for Janssen's theory and an equivalent geostatic pressure).
Upper values for the forces at the joints can be determined from Monolith theory. According to Brandl (1980), the theory provides safe designs even with optimistic assumptions such as; the angle of friction between the back of the wall and the backfill is equal to that of the backfill; the earth pressure resultant from the backfill acts in the lower third of the wall; and tensile stresses are sustained in the base. As shown in Figure 28, the theory predicts that joints at the front of the wall are more highly stressed than those at the rear, so the design for
A symmetrical crib should be based on the loads acting on the front joints. The loads on the front and rear joints are given by:

\[ N_A, N_B = N \left(0.5 \pm \frac{e}{B}\right) \]  

(21)

where;  
\( N \) = normal force at section,  
\( N_A \) = normal force at front joints,  
\( N_B \) = normal force at rear joints,  
(These forces are per metre run of wall)  
e = eccentricity of \( N \), and  
B = statically effective wall width.

In practice, the backwards inclination of the wall and frictional forces developed on the back of the wall may cause the joints at the rear to become more highly stressed than those at the front. A backwards inclination will, on the whole, reduce the forces at the joints and so the assumption that the wall is vertical will introduce some conservatism into design. However the differences between theory and practice must be considered at the design stage, particularly if the front and rear joints are not identical.

The maximum forces occur when the full weight of the infill is transferred to the crib. This assumption is evident in the form of the Monolith theory proposed by Thamm (1986) where, with reference to Figure 29:

\[ N_A = \frac{W}{2} + \frac{P_h \cdot z}{3b'} \]

(22)

\[ N_B = \frac{W}{2} - \frac{P_h \cdot z}{3b'} + P_v \]  

(23)

where;  
\( W \) = weight of crib elements and infill,  
\( P_h \) = horizontal component of earth pressure thrust,  
\( P_v \) = vertical component of earth pressure thrust,  
(These forces and thrust are per metre run of wall)  
z = depth of section, and  
b' = distance between centrelines of front and rear stretchers.
The results of tests undertaken by Thamm (1987) confirm that the theory is a safe method for determining the forces at joints.

The bearing stresses at joints can be assumed to be uniformly distributed over the bearing area and the element designed accordingly. A pad or mortar bedding between elements will help distribute the stresses.

In addition to checks on bearing failure, the designer must also consider other forms of failure at or near the joints between the elements. The frictional resistance of the joints to sliding will lead to an eccentricity in the forces above and below the headers, as illustrated in Figure 30. The headers should be designed to withstand the additional moment \((R_e)\) due to the eccentricity of the forces.

The transfer of the shear stresses, generated by the forces in the joints, to the headers and stretchers should also be considered. These stresses could be calculated from the pressures of the infill acting on the elements, but much higher stresses can be generated in structures subjected to extreme conditions. The crib wall at Canniesburn Road, Glasgow failed due to the overstressing of the unreinforced headers (Babtie, Shaw and Morton, 1991a). The resultant thrust at the bottom of this overloaded wall acted directly beneath the line of the
front stretchers. The high forces in the joints were transferred away from the front of the wall and the additional bending moment led to the overstressing and progressive failure of the headers. The form of failure is shown in Figure 31.

Sliding between the crib elements will lead to some deformation of the wall. Excessive movements may result in the stretchers bearing against the lugs on the headers and ultimately to shear failure. However good design and construction practice should prevent shear failure of the lugs - this pattern of failure is shown in Figure 32.

The lack of fit between elements of the crib, particularly timber ones that often have lower tolerances and are affected by warping, may induce additional stresses in the crib. Elements must be capable of resisting these secondary effects.
5.5.3 Shape considerations

The elements are designed to be symmetrical and interchangeable so as to prevent construction errors. However if an element is asymmetric then it should be obvious which way round it is to be placed in the crib.

The shape of the elements varies considerably between the various crib systems. The main function of the front elements is to retain the infill within the crib. There is usually no need for the rear stretchers and headers to retain the infill. Systems having closed front faces prevent the spilling out of infill, but at some cost to their drainage and appearance. The infill will be retained if the vertical spacing between the stretchers is small enough to allow arching of the soil between them.

Tschebotarioff (1952) suggested a ratio of 2:1 for the depth of the stretchers and the vertical spacing between them, but systems having ratios as low as 0.75:1 have proved satisfactory. From consideration of economy, simplicity, and aesthetics, Brandl (1982) suggested that the vertical spacing between stretchers should lie between 150 and 450mm: although this may be suitable for concrete elements, timber elements are typically spaced at about 75mm.

Forces are transmitted between the elements through bearing and friction. Interlocking devices such as lugs or dowels should perhaps only be used to aid construction, but they may provide some resistance to movement once the interface frictional forces are overcome. Features that provide interlock between elements should be able to resist the forces resulting from movements of the wall, but a lack of flexibility at the joints restricts the tolerance of the structure to differential movements.

5.6 Special cases

5.6.1 Multi-cell walls

The safe height of single-cell cribs is generally less than about three metres; higher walls require two or more rows of crib cells to achieve stability. It is unnecessary and uneconomic to continue the multi-cell section to the top of the wall. The wall may be stepped on either the front or rear faces as shown in Figure 33. The ledges to front-stepped walls provide a good facility for planting shrubs and bushes, but their overall appearance may be less pleasing than a plane facing. The cross-section of the cells can also be changed by using headers of different lengths.

![Fig. 33 Multi-cell walls with stepped profiles](image-url)
The following points should be carefully considered in the design of stepped walls:

(i) The distribution of soil pressures and hence the results of stability calculations depend on the assumed location of the 'virtual' back of the wall. Three options for the location of the virtual back of a stepped wall are shown in Figure 34. The simplest to apply is option (c), but according to Brandl (1982) it provides an unrealistically high factor of safety to design because of the location of the resultant force at the base of the wall. Brandl (1982) recommended the use of option (b) over option (a) as the former was simpler to apply and gave a more onerous loading condition.

![Figure 34](image)

**Fig. 34** Distributions of earth pressures behind multi-cell walls, after Brandl (1982)

(ii) For walls with a step on the back face, both the externally applied earth pressures and the internal cell pressures will be higher in the partial-height section than in the full height section. As shown in Figure 35, the vertical cell pressure at the top of the partial-height crib cell will be equivalent to the geostatic pressure at that level. This pressure will be higher than the pressure at the same level in the full height crib and will often be higher than the maximum cell pressure calculated from Janssen's theory.

![Figure 35](image)

**Fig. 35** Cell pressure in partial-height crib, after Brandl (1982)
(iii) In design, the critical sections will usually be at the changes of cross-section and just above the base of the structure.

(iv) If a change in cross-section is achieved by changing the length of the headers then high bending stresses may be developed in the longer headers, as shown in Figure 36. It may therefore be necessary to provide either strengthened headers or spacer blocks to help dissipate these forces. The spacers must not be too large as this may reduce the sliding resistance between the elements and induce forces due to lack of fit. A slight undersizing may prevent these problems occurring but this will also reduce the usefulness of the blocks.

![Fig. 36 Overloading of headers at changes of section](image)

(v) Multi-cell walls are inherently less flexible than single cell walls. They are therefore less tolerant of differential movements, particularly perpendicular to the line of the structure.

The above points may also apply to walls with recesses.

The failure of a multi-cell wall was reported by Tschebotarioff (1965). The direction of the frictional force on the back of this wall was reversed by settlement of the back of the upper part of the wall resulting from differential movement between the toe and the centre of the foundations. In design, it was assumed that the wall behaved as a monolith and this led to the installation of a concrete footing beneath the toe of the wall, as shown in Figure 37, with stretchers being used as foundations to the central and rear joints. The inclination of the wall and the distribution of internal stresses led to a higher reaction force on the central foundation than on the larger front foundation: subsequently the foundations at the centre settled by up to 0.6 metres more than at the toe.

![Fig. 37 Foundation of failed crib wall, reported by Tschebotarioff (1965)](image)
5.6.2 Terraced walls

The safe height of terraced structures is usually greater than that of a single structure, but where two or more walls are built on a slope they cannot be regarded as independent structures. The stability of individual structures and combinations of structures must be confirmed.

As a rule of thumb, the interaction between the walls is negligible when they are separated by a terrace having a width not less than the overall height of the lower wall, this does not of course eliminate the need for checking the overall stability of the combined structures. Terracing also provides excellent planting facilities and in some situations may improve the overall appearance: this aspect was discussed by Fuessinger (1983).

5.6.3 Curved walls

Most crib wall systems can be constructed to a curved alignment but such structures require particular consideration at the design stage.

Convex walls - i.e. walls that on plan bow away from the retained soil - are more prone to damage from outward movements than concave walls. Brandl (1982) found that the forces at the front joints of convex walls were greater than for straight walls. These higher forces, and also any increase in the bearing pressures at the toe of the wall generated by the curvature, must be accommodated in design. Any outward deflection of a convex wall results in gaps appearing between the ends of stretchers on the front face; this mechanism is shown in Figure 38. This movement induces a tension in the stretchers and can lead to spalling and separation of the elements.

![Plan view of joints](image)

**Fig. 38** Effect of outward movement on convex walls

Curvature of an inclined wall can be achieved in two ways. The wall can be built so that its face has the form of a tilted cylinder, and so with a convex wall the courses rise up at the centre and fall off at the wings. Alternatively the courses can be kept horizontal, but in this case either the length of the stretchers will vary over the height of the wall or gaps in the face will occur. The latter approach may be preferred because it is more aesthetically pleasing and easier to set out, but both techniques require careful construction.
5.6.4 Open-back walls

For economic reasons some crib walls have been constructed where some or all of the rear stretchers have been omitted. This has usually been done in the construction of low walls, where the consequences of failure are not serious, and for the upper section of high walls. Such open-back walls do not usually perform well. Their load bearing capacity is reduced and, according to Brandl (1982), the silo effect does not occur to the same extent as for walls with closed cells: thus the earth pressures and the joint forces are greater in such open cell cribs. The omission of alternate rear stretchers is common, but Brandl (1984b) suggested that such walls behave much the same as fully open-backed walls rather than walls with closed cells and also stated that many failures have occurred because the effect of omitting rear stretchers was not properly assessed.

As a first approximation to analysing the cell pressures within an open-back wall, Janssen’s theory could be applied with the cell width (b) tending to infinity. This gives the following expression for vertical cell pressure \( p_{ve} \):

\[
p_{ve} = \frac{\gamma \cdot a}{2K \cdot \tan \delta_i} \left[ 1 - e^{-2K \cdot \tan \delta_i \cdot \frac{z_i}{a}} \right] (\text{with } b \to \infty \text{ and } c' = 0)
\]

The results from both site and model studies presented by Brandl (1984a) show that higher pressures than calculated from the above are mobilized. The pressures developed near the front face of the wall are generally less than calculated (but they are still greater than those developed in closed cells) and the pressures at the back of the wall can attain equivalent geostatic pressures. As with closed cell walls, pressures are dependent on the inclination of the wall and the relative movements between the infill and the crib skeleton.

In general the bearing stresses at the base of an open-backed wall will be higher than for a similar closed crib wall. According to Brandl (1984a), these stresses can be conservatively calculated using Monolith theory and assuming a trapezoidal pressure distribution. However as these walls are usually of low height they are unlikely to be provided with slab foundations, in which case concentrations of load will be generated beneath the headers.

A quick method of assessing overall stability is to ignore the weight of any infill behind a vertical line from the topmost rear stretcher, as shown in Figure 39, but to take account of the effect of friction on the headers.

![Fig. 39 Quick method for assessing stability off walls with omitted rear stretchers](image-url)
Two approaches were given in the Permacrib literature (undated) for determining the overall stability of low height walls, i.e. up to 3 metres, and where the consequences of failure are not serious.

I. **Monolith theory:** This can be applied to open-back walls in a similar way to closed-cell walls except that the design must take account of the reduction in restoring moment due to the infill not being contained by the cell walls. This is modelled by using a reduced statically effective width ($B_{red}$), as shown in Figure 40. The reduced width is determined by calculating the length of header required to transfer the tension in the headers to the fill, i.e. the anchorage length ($l_a$). The method for calculating the anchorage length is given in Appendix 2. No account is taken of lugs on the headers and frictional forces are assumed to be generated only on the sides of the header and so the method has some in-built reserve of safety. The reduced statically effective width is dependent on the location of the maximum frictional force and the limiting cases are shown in Figure 41.

![Diagram of reduced statically effective wall width](image)

**Fig. 40** Reduced statically effective wall width for open-back walls

![Diagram of sectional and plan view](image)

**Fig. 41** Options for the determination of reduced statically effective wall width, $B_{red}$, after Brandl (1984)
If the design value of the angle of interface friction between the back of the wall and the backfill ($\delta_w$) is less than $\frac{3}{4} \phi$, then the required anchorage length may be greater than the length of the header units. The reserve of safety in the method should allow values of $\delta_w$ of between $\frac{1}{2} \phi$ and $\phi$ to be used - this is supported by the satisfactory performance for walls up to about three metres high.

II. Analogy to reinforced soil: The concept of an anchorage length is implicit in the design of reinforced soil structures and so the stability of open-back crib walls may be assessed by analogy to such structures. When considering overall stability, the analogy may give unreliable results for crib walls having surcharge slopes steeper than about 25° to the horizontal.

The possibility of individual headers being pushed or pulled out is greatest at the base of a wall, but this is still unlikely and internal stability checks as undertaken for reinforced soil structures have limited relevance. Brandl (1980) suggested that checks on the overall stability and on the stability of wedges cutting through the wall are sufficient to give a safe design. The stability of such wedges can be assessed by comparing the total pullout resistance of the embedded length of the elements behind an assumed slip plane with the sliding force of the wedge; the arrangement is shown in Figure 42. The walls are more stable than the above analysis predicts, possibly due to the under-estimation of the pullout resistance of the headers, and Brandl (1980) suggested that a factor of safety of unity would therefore be appropriate with this method. The pullout resistance of headers can be determined from the frictional forces developed on their sides.

![Fig. 42 Wedge stability analysis of an open-back wall](image)

It is likely that pressures due to at rest conditions will be generated at the front face of low walls, but active conditions may prevail for higher, and therefore more flexible, walls. It is commonly assumed that the lateral earth pressures acting on the sides of the headers correspond to at rest conditions. The dimensioning of the members of the crib should perhaps be based on geostatic cell pressures with either active or at rest conditions assumed to be acting at the wall face. As discussed in Section 4.2.6, the effect of unfavourable settlements must also be taken into account when designing the elements.
5.6.5 Environmental barriers

Some crib walling systems have been used in residential areas to construct noise and visual screens to heavily-trafficked roads. Environmental barriers are usually free-standing and although there are no externally applied earth pressures the effects of wind loading must be considered. If protection is not provided then impact loads on the face of structures abutting highways must be considered.

Little or no silo effect may be developed within the cells of a structure having a trapezoidal cross-section. Those with vertical sides may develop an appreciable silo effect but the cell pressure will be affected by the gaps between stretchers on both faces of the structure. Nevertheless Silo theory could be used to calculate the forces generated within the cribs. The minimum cell pressures could also be determined by considering failure wedges through the structure, as shown for example in Figure 43.

![Wedge analysis for determining stability of an environmental barrier having a trapezoidal cross-section](image)

5.6.6 Walls with spurs

As described by Tsagareli (1969), the performance of a reinforced concrete cantilever retaining wall can be improved by providing a shelf on the back of the wall. Such an arrangement is shown schematically in Figure 44. The bending moment at the base of the wall is reduced by the moment generated by the weight of the fill acting on the shelf and also by the reduction in soil pressures on that part of the wall below the shelf. These therefore allow a reduction in the section of the stem. The vertical pressure immediately below the shelf is likely to be low, particularly if the soil has settled and the distribution of lateral pressure is shown in Figure 45. The accompanying increase in the flexibility of the structure may also increase the likelihood of active conditions being generated at the back of the wall.

A similar concept can be used to increase the stability of crib walls, with the spur either being constructed from standard wall elements or cast as a reinforced concrete slab.
To provide adequate rigidity for screening of the soil pressures, Brandl (1982) recommended that spurs constructed from standard elements should be at least five courses deep. The screening effect may not reduce the lateral pressures to zero as the vertical pressures acting on the spur will be transferred to the fill directly beneath it. The crib elements must be designed to withstand the moments and the joint forces generated at the connection between the spur and the wall, and also, as shown in Figure 46, the relatively high cell pressures within the spur. Nonetheless this type of feature may prove to be economic.
A greater screening effect may be achieved with a reinforced concrete cantilever spur. The resultant additional loading on the crib elements is likely to be very high, particularly on the rear joints, and substantial strengthening will be required at the connection between the wall and spur.

The optimum height of the cantilever to achieve a maximum reduction in soil pressures, and thereby a maximum increase in stability, is given by:

\[ \frac{h}{H} = 0.7 \]  

where; \( h \) = depth to cantilever, and \( H \) = overall height of wall.

This optimum may result in heavy loading of the slab, generating high joint forces that cannot be easily accommodated, and severe practical difficulties during construction. Brandl (1982) therefore recommended that:

\[ 0.4 \leq \frac{h}{H} \leq 0.6 \]

To minimize the forces at the joint with a spur, a reinforced concrete slab could be continuous through the section of wall. This would effectively divide the crib wall into two parts and thereby substantially affect the conditions within the cell: the pressures in the cells above the slab may be increased, as may the joint forces below the slab. Such changes must be taken into account during design and the designer must also be aware that the introduction of any kind of spur will reduce the tolerance of the system to differential movements.

5.6.7 Anchored walls

Prestressed ground anchorages can be installed to provide stability to crib walls built on quasi-stable slopes. The anchorages can be installed during the construction of the crib wall, or at a later date in conjunction with injection grouting. It may be sensible in the design of such walls to provide spare sockets for additional anchors should future strengthening be required.

Frame element walls lend themselves to such works because the frames can easily be strengthened with concrete infill to resist the anchor forces without much reduction in the flexibility of the structure. However most crib systems are susceptible to concentrated loads and excessive local deformation may be generated by high anchor forces. It is therefore usually better to introduce a large number of low capacity anchors than a few highly stressed ones. Local strengthening and stiffening of the wall with concrete infill may also be necessary.

Stiffening of the wall will probably increase the lateral earth pressures acting on the back of the wall, possibly up to at rest pressures, and also raise the line of action of the resultant thrust to above \( H/3 \) from the base.
Deadmen anchors, as for example used in the TRL Anchored Earth system described by Murray and Irwin (1981), could be used but it is unlikely that this form of construction would prove economic.

5.7 Environmental considerations

The two main characteristics of crib walls that can offer environmental benefits are appearance and sound absorption. Appearance is often as important as technical or economic factors in the selection of the choice of wall. A few guidelines have been offered by Schuler (1983):

(i) Attempts to imitate nature and its complexities may produce unsatisfactory results, but in the right setting large, obviously man-made, simple structures may look attractive.

(ii) Walls with many discontinuities and lacking any regularity of form may appear as a stockpile of components. The use of repeated patterns of units and overt terracing will help to avoid this.

(iii) Consideration should be given to the ends of the walls at an early stage in design. It should not be presumed that these will somehow blend into the background.

(iv) A regular pattern of planting will look unnatural and should be avoided.

The opportunity for planting is a significant advantage that crib walls have over other types of walls. The vegetation may dampen down the dust generated on carriageways and eliminate problems of graffiti. With effective planting the crib wall may become overgrown to form a ‘greenwall’ but the specification for planting requires careful consideration - planting should not be so dense as to prevent proper and regular inspection of the supporting crib. Plants should be suitable for the conditions of the site and altitude, precipitation, exposure, atmosphere, and the effect of de-icing salts must be considered. Maintenance should also be minimized. Although, initially, planted native species may look sparse they will need little maintenance once they begin to flourish. The use of native plants is probably best suited to rural areas. The growth of weeds, especially in rural areas, will help generate the ‘greenwall’ effect, but for agricultural reasons ragwort and thistles should be removed. In urban areas it may be better to use flowering plants and shrubs but these may then require regular maintenance.

The requirements for plant growth may conflict with those demanded for stability. For stability the infill should be specified as a well-draining granular soil but this discourages plant growth. As shown in Figure 47, a layer of topsoil could be provided on the air side but this will contaminate the infill. Alternatively, ‘grow-bags’ could be used but the plants will be affected by desiccation and successful growth cannot be guaranteed. Walls having a steep front face may collect little rainwater and a batter of 1 (Horizontal) in 5 (Vertical) is required to avoid artificial watering. A shallower incline will be necessary for sheltered faces with batters of 1 (H) in 2 (V) and 1 (H) in 3 (V) being more appropriate. The watering of environmental barriers is particularly difficult, but mulching may help retain moisture in the topsoil.
The ability of a structure to absorb sound is mainly dependent on the proportion of concrete in the face area: as shown in Figure 48, the larger the proportion the greater the reflection. Consequently the requirements for establishing plants and sound absorption can conflict but the growth of vegetation will reduce the amount of reflected sound.
6 FABRICATION OF ELEMENTS

6.1 Materials

Cribs have been constructed from steel, plain and reinforced concrete, and timber elements. Problems of durability and cost rule out the widespread use of steel but both concrete and timber elements are commonly used.

Walls with plain concrete elements have been built satisfactorily. However given the difficulty of accurately assessing stresses within the cribs, there is a high risk that plain concrete elements will be overloaded. Reinforced concrete elements have a greater capacity to withstand forces that deviate from those predicted and they also increase the resistance to differential movements. Crib walls are often constructed adjacent to carriageways and may therefore be subject to spray laden with de-icing salts. Setting the wall back from the carriageway will reduce the severity of the conditions and exposure may then not be as severe as for road pavements and bridge parapets. According to BS 8002 (1994) it is preferable that the carriageway should be set back from the crib wall a distance of 4.5m or the height of the wall, whichever is greatest. It is usual to specify air entrained concrete for the elements and it should certainly be specified for more extreme exposures to increase resistance to frost attack. Characteristic strengths specified for reinforced concrete units typically range between 30 and 50N/mm². Air-entrainment for the higher strengths may be inappropriate.

Timber is often assumed to have a short design life and has therefore only been recommended for use in temporary structures. Modern timber crib walls are however almost all built using Radiata Pine and this permits effective penetration of copper, chrome, and arsenic (CCA) preservatives and allows manufacturers of timber cribs to guarantee a service life of 50 years. The manufacturers in the UK include Anda-Crib Ltd., Permacrib (Euro) Ltd., and Pinex Timber Products Ltd. This combination of timber and preservatives has not had a long history of use but tests have shown that the performance of CCA preservatives is superior to that of creosote - a well proven preservative - and that service lives in excess of 100 years might be expected. Timber has a natural and pleasing appearance and when logged from managed pine forests can be regarded as an environmentally friendly building material.

Some wall systems include metal dowels or pins to provide interlock between elements, but as these are prone to corrosion they are not recommended for long-life structures.

6.2 Specification for manufacture

Freeman Fox & Partners (undated) reported difficulties with the control of the size of precast reinforced concrete elements; these were due to the moulds becoming worn as the job progressed and also on the reliance of 'floating' the top surface of the concrete. The cumulative effect of these errors led to variations in the level of the top of the constructed wall. Steel moulds are therefore recommended, and a tolerance of ± 1mm may be appropriate.
Another recurrent problem is the lack of cover provided to the reinforcement due to inaccurate bar bending. The required cover is dependent on the exposure level but it should comply with the relevant standards and specifications of the procuring authority. Cover is typically between 30mm and 40mm but greater cover may be required for crib walls constructed adjacent to carriageways.

The criteria for reinforcement given in the 1985 German code for crib walls are as follows:

- Minimum bar diameter = 6mm,
- Maximum bar spacing = 100mm,
- Minimum reinforcement at ends and corners = 1 No. T10.

These criteria are applicable to most situations but more stringent requirements may be required for some structures. A symmetric layout of reinforcement is recommended and should be mandatory where there is a possibility that the elements could be placed upside down. The detailing of the reinforcement to the ends of the headers is also important - Brandl (1984b) proposed the arrangement shown in Figure 49.

![Fig. 49 Proposed reinforcement detail for the ends of headers](image)

In New Zealand the elements have to undergo pre-qualification tests both on site and in the laboratory. Details of the test are given in the 1988 New Zealand Specification CD209. The following are examined:

- Minimum area of reinforcement,
- Anchorage of reinforcement in header,
- Condition of reinforcement,
- Placing of reinforcement,
- Cover to reinforcement,
- Strength,
- Bearing area between elements,
- Dimensions and tolerances,
7 CONSTRUCTION

7.1 Construction details

7.1.1 Wall batter and cell shape

Inclining the face of a wall will reduce the internal cell pressures, joint forces, and base pressures, and will increase the margins of safety against overturning and sliding. It is usual to treat battered walls as vertical structures thus introducing a margin of safety into their design. Walls of comparable height and width could be constructed with a vertical face, but otherwise a batter should be provided. Taking account of economic aspects, erection difficulties, and soil pressures, Brandl (1982) proposed an optimum batter of 1(H):5(V); but he recommended batters ranging from 1(H):12(V) to 1(H):4(V) for general structures and batters of between 1(H):2(V) and 1(H):3(V) for planting purposes and for erosion control. Nonetheless batters ranging from 1(H):4(V) to 1(H):6(V) are most commonly specified. However BS 8002 (1994) stipulates that the batter should not be steeper than 1(H):4(V) for walls with a height greater than their thickness.

The influence of the shape of the cell on the silo effect means that square cells are the most economic form. Deep cells with closely spaced headers may be necessary on quasi-stable slopes, otherwise longer cells will be less costly and quicker to erect. A compromise solution is possible with frame element walls in that short cells can be used in the towers and long cells in the fields.

7.1.2 Foundations

Serviceability failures may be due to the excessive cracking of elements, the opening of joints, and loss of infill from the cribs. To prevent such failures it is necessary to determine the maximum deformations that can be attained and to provide adequate foundations. The distribution of the pressures at the base of crib walls allows low walls to be constructed without concrete foundations; however such walls still require reasonably uniform and firm subsoils. Walls higher than about three metres, and those constructed on poor subsoils, will usually require reinforced concrete slab foundations. If walls are constructed on compressible subsoils, then without the provision of a base slab the crib cell may settle relative to the infill thereby greatly increasing the cell pressures. With a suitable foundation, settlement of the infill should generate a downwards frictional force on the crib thereby increasing stability. It is usually uneconomic to provide individual footings to the headers and the provision of a continuous base slab may be more convenient.
The two most common arrangements for constructing in situ reinforced concrete foundations are shown in Figure 50. In (i) the cribs are constructed upon the base slab whereas in (ii) a number of courses are built up on the subsoil and then concrete is poured to engulf the bottom one or two courses. It may prove difficult with option (i) to produce a sloping surface with an adequate finish. With a slope of 1(V):4(H) it is difficult to impart sufficient vibration to properly compact the concrete and so a stepped surface may be beneficial. Surface irregularities may make it difficult to properly bed-down the elements and uneven support may result in the failure of some members. A thin layer of mortar or sand can be placed to provide an even bedding; alternatively, elements can be laid into wet concrete but this may present problems of setting-out. Three or four courses of units can be placed to help prevent movements of the bottom layer. Option (ii) requires that elements be set out accurately on the subsoil and secured prior to the placing of the concrete, and requires a greater volume of in situ concrete than option (i).

![Fig. 50 Options for constructing in situ concrete foundations](image)

Walls that do not require concrete bases will usually be founded on their lowest stretchers, with sand or mortar being used to provide a level base. Stretchers can be installed to act as strip foundations but these and any other additional components must be strong enough to withstand the foundation loads. The analysis by Tschebotarioff (1965) of a failed crib wall indicated that its foundation was inadequate; site excavations revealed cracks in the headers and in the blocks provided for spreading the load. The mechanism of failure is shown schematically in Figure 51.

![Fig. 51 Failure of stretcher footing due to inadequacy of header and spacer block to distribute load, after Tschebotarioff (1965)](image)
Whatever type of foundation is used at least one course of units should be below ground level and concrete foundations should be below the level affected by frost.

7.1.3 Drainage

A drain should be provided at the back of the wall at the level of the foundation. Water draining from the surrounding retained ground should be prevented from eroding the fill and whenever necessary land drains should be provided. If a structure is used to retain a cutting and the permeability of the natural soil is low then adequate drainage must be provided. A geosynthetic filter placed between the retained soil and the backfill may prevent fines contaminating the backfill, but the creation of a potential failure surface must be considered as must the possibility of the filter becoming clogged leading to a build-up of pore water pressures. The long term permeability of the filter must be assured.

If the backfill is relatively free-draining then large influxes of water, due to a thaw or heavy rainfall, may wash out part of the infill if the horizontal gaps between stretchers exceed about 30mm, Brandl (1984b). The presence of vegetation may resist this to some extent - but it cannot be relied upon to prevent it.

7.1.4 Layout of elements

The tolerance to differential movements can be improved by providing longitudinal discontinuities along the line of the structure. All structures built on poor ground should have such discontinuities every 4 to 15 metres. In walls where the stretchers span more than a single cell the joints between stretchers should be staggered both vertically and between the front and back faces. With this arrangement, stretchers or ‘closers’, as detailed in Figure 52, are required for wall terminations. Structures with staggered joints are stiffer than ones in which the joints are aligned vertically.
Headers are most commonly positioned in a vertical plane but, as shown in Figure 53, they may also be staggered. If headers are staggered then spacer blocks or false headers may be necessary to prevent excessive bending moments developing in the stretchers. Thus with a constant spacing between the headers a larger number of components is required, but the spacing between the headers can be increased.

![Figure 53 Layout of headers]

High walls, particularly those constructed from timber, will often have spacer or ‘pillow’ blocks installed between the headers in their lower courses. Spacer blocks are used to provide support to the headers thereby reducing bearing and bending stresses, but experience of their use with concrete elements shows that they tend to restrain movements and lead to flexural and torsional cracking. The use of such blocks should therefore be limited and they should possibly be slightly undersized so that dimensional inaccuracies do not lead to any restraint in movement. The use of header support blocks is more common in timber walls - due to the greater flexibility of the timber components the effect of additional restraints is less significant.

The members of a crib should be able to act independently and so restraints and indeterminate wall systems should be avoided, as these may generate problems due to differential temperatures and settlements within the crib. Pads or mortar beds placed between elements can increase the flexibility of a wall and these also produce a more uniform distribution of bearing pressure at the joints. The pads should not be too thick or deformable so as to introduce stresses into cells or reduce the frictional resistance available at joints. Mortar beds are usually adequate and the coefficient of friction of a cracked bed is equivalent to that of a concrete to concrete interface.

7.1.5 Special features and additional requirements

Features, such as corners, that need to be accurately positioned should be set-out and constructed first with the wall being built away from them. The batter of the wall generates problems with corners and with curved structures. This necessitates the cutting of stretchers to size so that there are no large gaps between them. An advantage of timber systems is that cutting the elements to size is a simple procedure, but preservatives must be applied to freshly cut ends. Similarly, the reinforcements within concrete elements are exposed when cut - an epoxy mortar coating could be applied to prevent corrosion at these ends. Special header units, as shown in Figure 54, are also required at corners. Curved crib walls can be built to a radius as tight as 3.5 metres, but although corners may therefore be avoided this does not necessarily eliminate the need to trim elements to length.
Details at the top and the ends of walls need to be considered. The topmost header in timber walls will usually need to be fixed down, and this can be simply done with nails. Timber elements can also be nailed together at the ends of walls as an aid to construction, but nails reduce the durability of timber.

Loads from traffic or railway sleepers acting directly on the top of the wall should be avoided as these may lead to the failure of the crib elements. Walls built adjacent to carriageways should be provided with protection against vehicle impact. If, for example, a parapet or safety fence is anchored to the foundations of a crib wall then the ramifications for the stability of the structure must be checked.

Crib walls should not be subjected to appreciable vibrations without adequate interlocking being provided between the elements. A 'locking bar' between the elements, as shown in Figure 55, may be beneficial when walls are constructed in earthquake zones (Göbel, 1969).
7.2 Fill

Both the backfill and infill should be clean, well graded, coarse granular soils - they should be free-draining and have a reasonably high angle of internal friction.

The maximum particle size of the soils should not be so large that effective compaction is hindered or the crib elements are damaged during compaction. It is therefore usual to specify a maximum particle size of between 75 and 100mm. Brandl (1985) proposed that the maximum particle size should be less than 1/6th the smallest internal dimension of the cell and also less than half the depth of a layer of fill.

The maximum fines content of the fills should also be limited. The higher the proportion the lower the permeability and the greater the tendency for the fill to spill through the gaps between the elements of the crib. No more than 40 per cent by weight of the fill should pass a 63μm sieve, but a figure of 15 per cent is usually specified so that investigations into the shear strength and permeability of the soil are unnecessary.

The infill to a crib structure used as a revetment should be at least as permeable as the most permeable layer of the retained ground. This should prevent the build-up of hydrostatic pressures behind the structure as may occur, for example, in the arrangement shown in Figure 56.

![Cross-section through crib revetment](image)

**Fig. 56 Build up of hydraulic pressure behind crib revetment**

Topsoil and any growing medium, added to promote and sustain the growth of plants, must not adversely affect the durability of the components of the crib. The degree of contamination of the fills by the topsoil must not be such as to lead to excessive deformation of the structure. It may prove advantageous to install a geosynthetic layer to separate the topsoil from the fill; this was discussed in Section 7.1.3.
7.3 Methods and specification

As shown in Figure 57, the movements of a crib wall are dependent on the method of construction. If the backfill is placed as the crib cells are built up then movements are concentrated in the lower half of the structure. The maximum displacement is greater if the crib wall is constructed before placement of the backfill and it occurs at the top of the wall.

The horizontal movements within the retained backfill decrease with increasing distance from the wall. The extent of the zone of movement will be smaller when the backfill is placed with the infilling of the cribs than when the backfill is placed after the crib wall has been built: thus it is usually advantageous to place the infill and the backfill simultaneously. Regardless of the sequence of construction, movements generated by surcharge loads will be concentrated at the top of a wall.

Headers are not usually designed to withstand horizontal lateral pressures; it is therefore important that the difference in the level of infill in adjacent cribs is controlled. Restricting the thickness of the layers of the infill will generally improve the performance of a structure.

The following construction procedures are recommended:

- the construction of the crib cells should not precede infilling by more than 3 or 4 courses.
- the fill should be placed and compacted in layers no more than about 500mm thick.
- care must be taken during filling so that voids are not formed within the infill and around the elements, and the elements are not disturbed or damaged.
- heavy compacting equipment should not be used close to the wall. Small plate vibrators, or simply just a tool handle, should be used for compacting within and close to the crib structure.
- to ensure that the structure behaves more or less as a monolith the level of compaction within the cells should be between 95 and 98 per cent of the Proctor Density.
to ensure adequate anchorage of the headers in open-back walls the level of compaction should be between 97 and 100 per cent of the Proctor Density.

Accurate setting-out and construction of the first few courses will ease construction of the upper part of the wall. This initial work should therefore be undertaken by qualified personnel but the construction of the remainder of the wall does not usually require skilled labour. It may not be efficient, or economically desirable, to maintain the same rate for backfilling the wall as for constructing and infilling the cribs. Nonetheless it is important that this be done, as far as possible, because interruptions in construction may produce discontinuities in the horizontal alignment of the elements.

The rate of construction is dependent on the size and weight of the elements. Concrete elements will probably require more than one labourer to carry and place, with cranage being necessary for large elements. Timber elements are relatively light and are therefore more likely to be disturbed during the placing of the infill, but this problem can be alleviated by nailing together the elements on the first course and at the ends: warped timber elements are difficult to place.

8 CONCLUSIONS

The long-term use of crib retaining walls and the large number of successful applications throughout the world suggest that there is a general acceptance of the suitability and economy of this form of construction. Crib walling has advantages associated with its relative ease of construction, economy, and aesthetics.

BS 8002 (1994) proposes that crib walls should be designed as gravity mass walls, but this theory takes no account of the pressures generated within the crib or the forces sustained by the elements of the crib. The essential behaviour of such walls is therefore not captured.

The results from an extensive investigation into the performance of crib walls have been presented by Brandl (in references between 1980 and 1985) and useful data from model tests and field studies have also been given by Thamm (1986, 1987). These provide a good basis for the development of a method of design, and a recommended step-by-step approach to design is summarized in Table 5. (This approach is similar to that given in the 1985 German Transport and Road Research Association guide for the construction of crib walls and embankments). The details of a design will vary with the form of construction: for example, environmental barriers are normally free-standing, so there will be no externally applied earth pressures but wind loading will have to be assessed.

The literature review showed that there was a substantial amount of information available to enable an authoritative design guide to be written. Such a design guide would be of value to clients, designers, contractors, and manufacturers and lead to cost-effective and safe structures - the key aspects affecting the public interest.
1. **DETERMINE OPTIMUM WALL DIMENSIONS BY CONSIDERING STABILITY AGAINST OVERTURNING**
   - Assume 'active' conditions prevail behind wall
   - Determine an appropriate angle of interface friction for the back of wall
   - Use Monolith theory

2. **CHECK STABILITY AGAINST SLIDING**
   - Consider failure beneath the foundation, and between the base of wall and the foundation, using Monolith theory
   - Calculate the sliding resistance through sections of the wall using Silo theory with a limit value approach

3. **CHECK BEARING CAPACITY**
   - Use Monolith theory and assume a trapezoidal distribution of pressure
   - Provide adequate foundations to reduce settlements to acceptable levels and eliminate bearing failure of the subsoils

4. **CHECK OVERALL STABILITY**
   - Include a check of failure planes passing through wall

5. **CHECK CAPACITY OF CRIB ELEMENTS TO WITHSTAND FORCES**
   - Assume at-rest conditions prevail behind the wall
   - Calculate cell pressures using Silo theory with a limit value approach

6. **CHECK CAPACITY OF CRIB ELEMENTS TO WITHSTAND FORCES AT JOINTS**
   - Use Monolith theory to calculate joint forces
   - Consider bearing pressures at contact points and additional bending moments and shear forces due to joint forces
   - Consider shear of interlocking devices

7. **DESIGN PERIPHERY AND FEATURES**
   - Consider drainage, planting, terminations, foundations, corners, etc

**TABLE 5** Step-by-step approach to design
This report is based on the work presented by Babtie Shaw & Morton under contract to TRL. The TRL Project Manager for the contract was Guy Watts.

The authors would like to thank Robin Whitworth of Babtie Shaw & Morton for his assistance in the preparation of the report. Thanks are also due to Heinz Brandl and the University of Toronto Press for their permissions to reproduce some of the figures contained within this report.

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61


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Appendix 1 DERIVATION OF JANSSEN'S FORMULA DESCRIBING THE DISTRIBUTION OF VERTICAL CELL PRESSURE

Janssen assumed that the vertical cell pressure \( p_v \) was uniformly distributed across a cell filled with cohesionless backfill. Considering an incremental slice at depth \( z \) of thickness \( \delta z \), as shown in Figure A1.1 the weight of the slice \( \delta w \) is given by:

\[
\delta w = A \cdot \gamma \cdot \delta z
\]  

(A1.1)

where; \( A \) = cross-sectional area of the cell, and  
\( \gamma \) = unit weight of the infill.

Fig. A1.1 Forces acting on incremental slice through crib cell

The upwards thrust due to wall friction \( (F) \) is given by:

\[
F = K \cdot \tan \delta_i \cdot p_v \cdot U \cdot \delta z
\]  

(A1.2)

where; \( K \) = coefficient of lateral earth pressure of infill,  
\( \delta_i \) = angle of interface friction between the infill and the skeleton of the cell,  
\( p_v \) = vertical cell pressure at depth \( z \), and  
\( U \) = internal perimeter of cell.

Vertical equilibrium demands that:

\[
A \cdot (p_v + \delta p_v) - A \cdot p_v + K \cdot \tan \delta_i \cdot p_v \cdot U \cdot \delta z - A \cdot \gamma \cdot \delta z = 0
\]  

(A1.3)
thus,

\[ \frac{dp_{vz}}{dz} + \frac{K.U}{A}.\tan \delta_i . p_{vz} = \gamma \]  

(A1.4)

At the top of the cell the vertical pressure, with no surcharge load, is zero, thus;

\[ p_{vz} = \frac{A}{U} \frac{\gamma}{K.\tan \delta_i} \left[ 1 - e^{-\frac{Z}{z_0}} \right] \]  

(A1.5)

This can be written:

\[ p_{vz} = \gamma . z_0 \cdot (1 - e^{-\frac{Z}{z_0}}) \]  

(A1.6)

where,

\[ z_0 = \frac{A}{U} \frac{1}{K.\tan \delta_i} \]  

(A1.7)

For soils exhibiting cohesion \((c')\) the formula is:

\[ p_{vz} = (\gamma - c'.\frac{U}{A}) . z_0 \cdot (1 - e^{-\frac{Z}{z_0}}) \]  

(A1.8)

A uniform surcharge applied at the top of the cell \((q)\) will not increase the limiting value of \(p_{vz}\), but the equation for vertical cell pressure with cohesionless soils then becomes:

\[ p_{vz} = \gamma . z_0 \cdot (1 - e^{-\frac{Z}{z_0}}) + q . e^{-\frac{Z}{z_0}} \]  

(with \(c' = 0\))

(A1.9)
Appendix 2 DERIVATION OF ANCHORAGE LENGTH OF HEADERS IN OPEN-BACK WALLS

Consider an open-back wall with cell dimensions as defined in Figure A2.1. A linear distribution of horizontal cell pressure between the front and back faces, with a mean value of $p_h$, can be assumed for the purposes of this derivation: the maximum variation from the mean (both above and below) is represented as a fraction of the mean by $\Delta p_h$. The data from site experiments indicate that this is a reasonable assumption. The distribution of horizontal cell pressure is shown in Figure A2.2. A method for deriving anchorage lengths for more complex distributions is possible but this requires the integration of the cell pressure over the anchorage length.

The horizontal cell pressure ($p$) at $x$ is given by:

$$p(x) = p_h(1 + \Delta) - \frac{2x}{b}[p_h(1 + \Delta) - p_h] \quad (A2.1)$$
where; \[ p_h = K \cdot \bar{p}_{vz} \] (A2.2)

(K being the coefficient of lateral earth pressure of the infill)

Thus;

\[ p(x) = p_h \left( 1 + \Delta - \frac{2x \cdot \Delta}{b} \right) \] (A2.3)

Thus the frictional force \( p_w \) acting between a header and the infill is:

\[ p_w(x) = \tan \delta_i \cdot p(x) \] (A2.4)

\[ p_w(x) = p_h \cdot \tan \delta_i \left( 1 + \Delta - \frac{2x \cdot \Delta}{b} \right) \] (A2.5)

where \( \delta_i \) = angle of interface friction between the infill and the headers.

Thus for \( x = 0 \);

\[ p_{w(x=0)} = p_h \cdot \tan \delta_i \cdot (1 + \Delta) \] (A2.6)

and for \( x = b \);

\[ p_{w(x=b)} = p_h \cdot \tan \delta_i \cdot (1 - \Delta) \] (A2.7)

The force dissipated from headers to the infill via friction is given by;

\[ Z_R = 2 \cdot p_w \cdot l_a \cdot d \] (A2.8)
where; \( d \) = course height, and
\( l_a \) = anchorage length

The mean frictional resistance on the anchorage length is generated at \( x = l_a/2 \);

\[
\bar{p}_w = p_h \cdot \tan \delta_i \cdot \left( 1 + \Delta - \frac{a \cdot \Delta}{b} \right)
\]

(A2.9)

and so;

\[
Z_R = 2 \cdot p_h \cdot \tan \delta_i \cdot \left( 1 + \Delta - \frac{a \cdot \Delta}{b} \right) \cdot l_a \cdot d
\]

(A2.10)

The tensile force in the headers (\( Z_s \)) resulting from the horizontal pressure on the stretchers is:

\[
Z_s = a \cdot (1 - \Delta) \cdot p_h \cdot d
\]

(A2.11)

This must be dissipated through the headers by interface friction into the infill, thus;

\[
Z_s = Z_R
\]

(A2.12)

Therefore, the anchorage length can be calculated from the following equation:

\[
l_a \cdot (1 + \Delta) - \frac{l_a^2 \cdot \Delta}{b} = a \cdot (1 - \Delta) \frac{2 \cdot \tan \delta_i}{A}
\]

(A2.13)