BubbleDeck Design Guide for compliance with BCA using AS3600 and EC2

Prepared by: kyng consulting pty ltd
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Background

BubbleDeck is a slab system that has become very popular in Europe and around the world in the past decade. The system was invented in Denmark after a government sponsored competition looked for new ways to constructing buildings, and in particular new ways to enhance the flexibility and efficiency of design using pre-fabricated techniques. BubbleDeck won the competition and was successfully introduced to large scale commercial construction in 1999 on the Wena tower in Rotterdam, Holland. For the first time, it was possible to prefabricate a two-way concrete slab economically. This report presents the results of international research into the section properties of BubbleDeck slabs, and proposes methods that allows it to be designed to satisfy the requirements of the Building Code of Australia (BCA), using first principles and international design guidance.

BubbleDeck is generally designed using conventional design methods for solid slabs in accordance with current local standards of design and good practice. The guidance included in this report is based on the best practice for the design of BubbleDeck derived through experimental data and practical expertise carried out in Denmark, Holland and Germany, and which has been adopted by the Standards Authorities in the respective country. The recommendations from the research has also been endorsed by the British Standard Institute in the UK, which recommends that BubbleDeck be treated as a solid slab, with due regards to the impact of the void formers.

The following adjustments must be made to allow for the effect of the void formers, or bubbles:

- The Dead Load, G, of the structure is significantly reduced;
- The stiffness of the slab is adjusted for both cracked and uncracked condition;
- The shear capacity is reduced in the voided areas of the slab; and
- The punching shear calculations are as for a flat slab, as the slab is left solid around the columns. The author of this paper recommends the designers follow the guidance of Eurocode 2 - BS EN 1992-1-1:2004 (EC2) for the design of punching shear as the Australian Standard AS 3600-2001 results in undesirable complex site detailing when using flat slabs, as noted by Rangan et al in "Concrete Structures", p573.

This paper relates the recommendations made by the European Standards Authorities for the design of BubbleDeck slabs for practical use with AS 3600.
BubbleDeck slab profiles and characteristics

BubbleDeck slabs can theoretically be manufactured to any profile, but experience around the world has shown that the following 5 cross sections cover most building applications.

<table>
<thead>
<tr>
<th>Version</th>
<th>Slab Thickness</th>
<th>Bubbles (mm)</th>
<th>Span (Multiple bays)</th>
<th>Cantilever Maximum Length</th>
<th>Span (Single bays)</th>
<th>Completed Slab Mass</th>
<th>Site Concrete Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm</td>
<td>mm</td>
<td>metres</td>
<td>metres</td>
<td>metres</td>
<td>Kg/m²</td>
<td>m³/m²</td>
</tr>
<tr>
<td>BD230</td>
<td>230</td>
<td>Ø 180</td>
<td>5 – 8.1</td>
<td>≤ 2.2</td>
<td>5 – 6.3</td>
<td>370</td>
<td>0.11</td>
</tr>
<tr>
<td>BD280</td>
<td>280</td>
<td>Ø 225</td>
<td>7 – 10.1</td>
<td>≤ 2.7</td>
<td>6 – 7.8</td>
<td>460</td>
<td>0.14</td>
</tr>
<tr>
<td>BD340</td>
<td>340</td>
<td>Ø 270</td>
<td>9 – 12.5</td>
<td>≤ 3.3</td>
<td>7 – 9.6</td>
<td>550</td>
<td>0.18</td>
</tr>
<tr>
<td>BD390</td>
<td>390</td>
<td>Ø 315</td>
<td>10 – 14.4</td>
<td>≤ 3.8</td>
<td>9 – 11.1</td>
<td>640</td>
<td>0.21</td>
</tr>
<tr>
<td>BD450</td>
<td>450</td>
<td>Ø 360</td>
<td>11 – 16.7</td>
<td>≤ 4.5</td>
<td>10 – 12.5</td>
<td>730</td>
<td>0.25</td>
</tr>
</tbody>
</table>

BubbleDeck has been successfully used as transfer slabs with depths of over 600mm. Mega-spans can be achieved when combined with post tensioning, as was the case for the Danish Radio complex in Copenhagen (DR-Byen).

DR-Byen project showing prestressing tendons integrated with BubbleDeck slabs

BubbleDeck can be implemented as one of three alternatives:

1. **In-situ application** - BubbleDeck slabs can be constructed as in-situ slabs. The “bubbles” are placed in modules of top and bottom steel and are effectively held in place. The modules are then placed on conventional formwork, and the slab is poured in two parts; a first shallow pour is carried out to provide enough weight to resist the uplift force on the bubbles. A second pour, usually one day after the first pour, completes the slab. The in-situ alternative can be very attractive for projects where the finished soffit of the slab is domed or curved, or where access is restricted.
2. **Precast elements** - BubbleDeck slabs can be delivered to site as fully precast elements. This option would still allow the substantial materials savings provided by BubbleDeck, but it would limit the two-way benefit aspects of the slab, if the spans under consideration are larger than the precast element. The two-way action can be maintained by designing appropriate connections between the elements.

3. **Semi-precast elements** - The most effective delivery method for BubbleDeck slabs is through the use of semi-precast elements. These include the bubbles, and most of the main reinforcement for the slab. The elements are then stitched on site through a concrete pour. The semi-precast system offers great cost benefits, and it provides a unique solution for achieving precast two-way spanning slabs.

### Design for durability - Section 4 of AS 3600

All the recommendations in Section 4 of AS 3600 are unaffected by the use of void formers in the slab. The guidance therein applies to BubbleDeck slab design, and minimum cover for exposure classification shall be determined accordingly. Note that when semi-precast minimum cover at BubbleDeck splices is achieved through adequate detailing of precast element.

![Typical splice detail](image)

*Typical splice detail ensuring minimum cover for both fire and durability requirements*

### Design for fire resistance - Section 5 of AS 3600

Recommendations from the CSIRO using conservative analysis tools (2D model) to model fire loads in accordance with AS 1530.4 has shown that for BD280 slabs and above, a 120 minutes rating is achieved using the same provisions defined in Tables 5.5.1 and 5.5.3(A) of AS 3600. The same provisions can also be used for BD 230 slabs up to 90 minutes fire periods. (refer to CSIRO conclusions report)
The CSIRO assessment was generally governed by the insulation performance of the slab. Noting that the modelling was performed on a 2D basis, which assumed the void to be continuous along the section of the slab, a large degree of conservatism was introduced in the results. The voids formed in BubbleDeck slabs are discreet, and hence they would not provide the thermal bridging that a prismatic void would, as assumed by the CSIRO analysis.

More detailed analysis carried out on the BD 280 slab (using 3D modelling capabilities) has shown that BD280 slab exceeds the requirements of AS 1530.4 for 180 minutes period and hence performs in accordance with section 5 of AS 3600 for up to 180 minutes.

Table extracted from CSIRO report FCO 2645 on BubbleDeck Fire Performance dated 24 June 2008

<table>
<thead>
<tr>
<th>System</th>
<th>Simply Supported Slab</th>
<th>Continuous Slab</th>
<th>FRL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cover to bottom reinforcement (mm)</td>
<td>Cover to bottom reinforcement (mm)</td>
<td></td>
</tr>
<tr>
<td>BD230</td>
<td>20 15</td>
<td>60/60/60</td>
<td>90/90/90</td>
</tr>
<tr>
<td></td>
<td>25 15</td>
<td>120/120/120</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30 15</td>
<td>120/120/120</td>
<td></td>
</tr>
<tr>
<td>BD280</td>
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<td>60/60/60</td>
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<td>25 15</td>
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<td></td>
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<td>120/120/120</td>
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<td>90/90/90</td>
</tr>
<tr>
<td></td>
<td>25 15</td>
<td>120/120/120</td>
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<tr>
<td></td>
<td>30 15</td>
<td>120/120/120</td>
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<td></td>
<td>45 25</td>
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<td></td>
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<td>45 25</td>
<td>180/180/180</td>
<td></td>
</tr>
<tr>
<td></td>
<td>55 35</td>
<td>240/240/240</td>
<td></td>
</tr>
</tbody>
</table>

3D model of BD 280 slab by Arup with temperature distribution after 180 minutes
The CSIRO table shown above can be used conservatively in the preliminary phases of the design. 3D analysis of the slab demonstrates that the provisions of AS 3600 Section 5 are also applicable to most BubbleDeck slabs. The effective thickness of the BubbleDeck slab for determining insulation in Table 5.5.1 can be approximated as the total volume of concrete in one square meter of slab divided by one square meter of slab.

It is recommended that for the thinner BubbleDeck slabs, BD 230 and BD 280, an assessment is undertaken for the actual slab section profile by a reputable Fire Engineer in order to confirm the fire performance, when this is required to be above 90 minutes or 180 minutes respectively.

**Structural analysis - Section 7 of AS 3600**

BubbleDeck slabs behave isotropically, and hence they can be analysed using the same methods used for solid two-way slabs. The recommendations of Section 7 of AS 3600 can be used with the following provisions for the section properties of the slab.

**Bubbledeck section properties**

The typical BubbleDeck module is defined by the parameters $a$ and $D$, where $a$ is a measure representing the matrix of the void formers in the slab and $D$ is the overall depth of the slab.

### Basic Bubbledeck Geometry

![Basic Bubbledeck Geometry](image)

**Uncracked section properties**

The second moment of area for the concrete part of the slab $I_{BD,conc}$ can be expressed through the simple formula (Darmstadt University of Technology, Investigation of BubbleDeck slabs, Professor Dr. Ing. Martina Schnellenbach-Held):

$$I_{BD,conc} = D^4/12 - 0.124 a^3$$

Thus the uncracked transformed section's second moment of area can then be obtained assuming that the centre of gravity for the concrete cross section lies at $D/2$. 

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Design Guide to BCA
If the second moment of area $I_{eq,steel}$ of the transformed steel area $A_{eq,steel}$, made up from the top $A_{sc}$ (compression) and bottom $A_{st}$ (tension) reinforcement areas which centre of gravity is located at a depth $d_{eq,steel}$, the total uncracked second moment of area of the BubbleDeck section becomes:

$$I_{BD,un-cracked} = I_{BD,conc} + I_{eq,steel} + A_{eq,steel}(D/2 - d_{eq,steel})^2;$$

assuming that the impact of the transformed steel on the centre of gravity of the total transformed section does not shift the centre of gravity of the concrete section significantly.

The equation above results in an approximate stiffness of the BubbleDeck slab of around 90% of that of a solid slab with the same depth.

Thus, the uncracked second moment of area can be determined for both short and long term conditions.

**Cracked section properties**

The effect of the voids on the cracked section properties is less than that manifested on the uncracked section. Hence, the cracked second moment of area for BubbleDeck slabs can be taken simply as:

$$I_{BD,cracked} = 0.9 I_{solid,cracked}$$

Thus, the cracked second moment of area can be determined for both short and long term conditions.

**Structural Analysis**

A structural analysis of the slab can now be carried out using the properties derived above, with due consideration to the reduced weight of the slab.

The effective second moment of area can be derived using the expression in 8.5.3.1 of AS 3600 for both short term and long term effects, using a reduced value of the cracked moment $M_{BD,cr} = 0.8 M_{cr}$ where the cracked moment $M_{cr}$ is calculated using the uncracked BubbleDeck section properties. This translates to the following expression;

$$M_{BD,cr} = 0.8 \left[ I_{BD,un-cracked} / \gamma_t \right] f'c$$

where $\gamma_t$ is the distance to the extreme tensile fibre, measured from the centroidal axis of the section (ignoring reinforcement).

Thus;

$$I_{BD,ef} = I_{BD,cracked} + (I_{BD,un-cracked} - I_{BD,un-cracked}) (M_{BD,cr} / M_c)^3 \leq I_{c,max}$$

where $I_{c,max}$ is as defined in 8.5.3.1 of AS 3600

The analysis can be carried out using commercial software packages through finite element analysis, or through 3D frame analysis packages modelling the slab as a grillage. It is recommended that any grillage be modelled on a 1m wide beam basis to facilitate the extraction and interpretation of results.

It is recommended that the deflection checks for the design of the slab are carried out in conjunction with the analysis of the slab. Generally, the requirements for deflection and crack width calculations become governing in the final design of the slab.
**Design of BubbleDeck slabs**

Once the analysis of the slab is complete, strength calculations can be carried out on the preliminary section derived from the analysis.

**Strength in bending**

BubbleDeck omits a significant volume of concrete (compared to a solid slab) in the central core of the slab, where stress levels are relatively insignificant when the section is in bending. When designing for flexural resistance, the depth of the stressed concrete in compression (often called the 'stress block') is concentrated within the solid concrete between the outermost extent of the bubble and the slab surface, whether the designer considers the stress block to be rectangular, recto-parabolic or other shape in accordance with accepted design methodology.

![Stress distribution through BubbleDeck under normal loading](image)

*Stress distribution through BubbleDeck under normal loading*

Sometimes, in heavily stressed slabs, the stress block will encroach slightly within the bubble zone. Studies and tests have demonstrated this has an insignificant effect on the resistance of a BubbleDeck slab in normal design situations. The following recommendation has been adopted by the German Standard DIN 1045, and gives a simple check which limits the extent with which the elastic neutral axis is allowed to encroach within the bubble zone.

![Compression zone can encroach on bubble zone for heavily reinforced slabs](image)

*Compression zone can encroach on bubble zone for heavily reinforced slabs*
In the expression below, $\mu_{ms}$ is a parameter defining the ratio of the moment resisted by the bubble zone to the total moment resisted by the cross section, $M_{ball}$ / $M_u$. BubbleDeck slabs can be designed using conventional design principles if this ratio is limited to 0.20, i.e. the stresses are allowed to redistribute locally, when this ratio is less than 20%.

$$\mu_{ms} = \frac{M_u}{1.96D} \leq 0.20 \; ; \text{where}$$

- $D$ is the ball diameter
- $h$ is the depth of the slab
- $M_u$ is the design Moment

The maximum depth of the neutral axis can also be derived through the following expression, using the limiting ratio above;

$$\frac{M_{ball}}{M_u} = \mu_{ms} = \left[ \frac{(d_n - c_{ball}) z_{ball}}{d_n z} \right] = 0.20 \; ; \text{where}$$

- $M_{ball}$ is the contribution of moment resistance by the section within the ball zone
- $c_{ball}$ is the top cover of the ball
- $d_n$ is the depth of the neutral axis
- $z_{ball}$ is the lever arm contributing to $M_{ball}$
- $z$ is the lever arm contributing to $M_u$

Thus, the strength in bending of a BubbleDeck slab can be calculated in accordance with 9.1.1 of AS 3600, provided that the proportion of the moment resistance by the concrete within the ball zone is limited to 20%. This effectively allows for a redistribution of the moment of resistance within the slab.

**Strength in shear**

BubbleDeck provides the designer with great flexibility, as the discreet bubbles can be omitted locally to either allow for the location of openings or cast in elements that would otherwise interfere with the overall design of the building, such as drainage pipes and electrical conduits. It also allows the designer to provide local increased resistance of the section for areas of high shear stress as it is the case around the columns.

AS 3600 addresses the effects of shear and unbalanced moments in solid slabs through the provision of torsional beams within the slab. The detailing of such reinforcement can become complex for thinner slabs, and becomes impractical when using a semi precast system such as the BubbleDeck semi precast elements, where the torsional reinforcement can lose its continuity through the splices. For this reason, it is recommended that codes of practice that use punching shear perimeter design be used, such as BS 8110 or the more up to date Eurocode 2 - BS EN 1992-1-1:2004 (noted EC2 hereafter), available from SAI Global. The use of section 6.4 of EC2 allows for simple detailing around the columns, and effectively increases the effective column head by providing shear reinforcement along the critical shear perimeters.
Punching shear calculations
The procedure for designing for shear resistance consists of determining four main conditions, as follows:

- Checking for column shear failure at the perimeter of the column;
- Determining the extent of the solid area around the columns, i.e. the area around the columns where the bubbles are omitted;
- Determining the minimum extent of the tension steel in the column area, i.e. defining the minimum length of the steel reinforcement that is included in the shear resistance calculation of the hollow section; and
- determining the shear reinforcement and its layout, as required.

Note that as the use of EC 2 is recommended, the analysis should include one load case for shear check using the combination factors of Eurocode, namely:

\[ E_{d,\text{dst}} = 1.35 G + 1.5 Q \]

Design shear magnification factor for out-of-balance forces
For structures where lateral stability does not depend on frame action between the slab and the column, and where adjacent spans do not differ in length by more than 25% the maximum design shear should be multiplied by the following factors, depending on the location of the column in the slab.

\[ V_{\text{max}} = \beta V_{Ed} \]

\[ \beta = \begin{cases} 1.15 & \text{for internal columns} \\ 1.4 & \text{for edge columns} \\ 1.5 & \text{for corner columns} \end{cases} \]

Where the spans differ more than 25%, refer to EC2 to calculate the appropriate magnification factor.

Column shear failure
The shear stress \( V_{Ed} \) along the column perimeter should be limited to the maximum allowable shear stress in the concrete section \( V_{Rd,\text{max}} \).
\( V_{Ed} < V_{Rd,max} \); where

\[
V_{Ed} = \beta \frac{V_{Ed}}{[u_0 d]} = \frac{V_{max}}{u_{col} d_{om}};
\]

\( u_0 = u_{col} = \text{perimeter of the column}; \)

\( d = d_{om} = \text{mean effective depth of the slab}; \)

\[
V_{Ed,max} = 0.5 \nu f_{cd}; \quad \text{EC2 6.4.5 (3) Note}
\]

\[
\nu = 0.6 / [1 - (f_{ck}/250)]; \quad \text{EC2 6.6N}
\]

\( f_{cd} = \text{characteristic compressive cylinder strength at 28 days (} = f'_c\) \)

\( f_{ck} = \text{Design value of concrete compressive strength} \quad \text{EC2 (3.15) and 2.4.2.4} \)

**Extent of solid area around columns**

The extent of the solid area around the column is determined by locating the perimeter at which the shear stress in the slab falls below the shear resistance of the hollow slab, including any tension column reinforcement.

The introduction of the voids in the slab reduces its shear strength. Studies carried out at Denmark’s Technical University, Darmstadt University in Germany and Eindhoven University in Holland have shown that the shear strength of a BubbleDeck slab can conservatively be taken as the shear strength of a solid slab of the same depth multiplied by a reduction factor of 0.6.

The maximum allowable shear stress in a solid slab is determined by:

\[
V_{Rd,c} = C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} \geq V_{min}; \quad \text{where EC2 (6.47)}
\]

\[
C_{Rd,c} = 0.18 / 1.5 = 0.12; \quad \text{EC2 6.4.4 (1) Note}
\]

\[
k = 1 + \left( \frac{200}{d} \right)^{1/2} \leq 2.0, \text{ where}
\]

\( d = d_{om} = \text{mean effective depth of the slab (}d_{ox}+d_{oy}\)/2\)

\( \rho_l = (\rho_{lx} \rho_{ly})^{1/2} \leq 0.02, \text{ where} \)

\( \rho_{lx} \) and \( \rho_{ly} \) are the reinforcement ratios relating to the tension steel in the x and y direction respectively, i.e. \( A_{st,x} / (bd) \) and \( A_{st,y} / (bd) \) respectively. The values of \( \rho_{lx} \) and \( \rho_{ly} \) should be calculated as mean values taking into account a slab width equal to the column width plus 3d each side. Only column steel is taken into account, disregarding the typical BubbleDeck top reinforcement.

\( f_{ck} = \text{characteristic compressive cylinder strength at 28 days (} = f'_c\) \)

\[
V_{min} = 0.035 k^{3/2} f_{ck}^{1/2} \quad \text{EC2 (6.3N)}
\]

Accordingly, the maximum allowable shear stress in the hollow areas of a BubbleDeck slab is:

\[
V_{BD, Rd,c} = 0.6 V_{Rd,c}
\]

Thus, the control perimeter that defines the extent of the solid zone around the column can be derived from the following expression:

\[
U_{solid} = V_{max} / (V_{BD, Rd,c} d)
\]
Note that the perimeters calculated in accordance with EC2 are the shortest perimeters at a given distance from the face of the column, e.g. for a rectangular section, the control perimeter should be obtained as follows:

![Diagram](image)

**Extent of column tension reinforcement**

In order to determine the minimum extent of main tension reinforcement around the columns for which the requirements outlined in the section above are met, a similar perimeter check needs to be carried out, taking into account only the typical top steel of the BubbleDeck elements, without the column steel.

The shear capacity of the hollow slab with only top steel can be calculated using $\rho_{lx}$ and $\rho_{ly}$ for the steel included in the top mat only and a new perimeter calculated.

$$v_{BDtyp, Rd,c} = 0.6 \times v_{typ, Rd,c} \quad \text{where}$$

$$v_{typ, Rd,c} = v_{Rd,c} \text{ using } \rho_{lx} \text{ and } \rho_{ly} \text{ for the steel included in the top mat only.}$$

Thus, a perimeter $u_{steel}$ can be determined:

$$u_{steel} = \frac{V_{max}}{v_{BDtyp, Rd,c} d}$$

The perimeter $u_{steel}$ defines a minimum distance from the face of the column to which the column steel must extend. The total length of the reinforcement bars over the columns can then be determined using figure 9.1.3.4 of AS 3600.
Column shear reinforcement

Column shear reinforcement is calculated in accordance with the guidelines of section 6.4 of EC2, as the provisions of AS 3600 result in complex reinforcement detailing for shallow slabs. The guidelines are applied without alterations as the slab is solid near the columns, and the shear strength is not reduced.

The process for designing any reinforcement that may be required around the columns is iterative. Once the shear check at the perimeter of the column has been satisfied, the shear stress is checked at the basic control perimeter.

The basic control perimeter is taken at a distance 2d from the face of the column, using d as the mean effective depth of the slab $d = (d_x + d_y)/2$, where $d_x$ and $d_y$ are the effective depths in the x and y direction respectively.
Typical basic control perimeters as per EC2

The shear stress $v_{Ed1}$ at the basic control perimeter $u_1$ should be checked against the shear strength $v_{Rd,c}$, where:

$$v_{Ed1} = \beta \frac{v_{Ed}}{u_1 d},$$

$$v_{Rd,c} = C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} \geq v_{min},$$

using the column tension reinforcement only to determine $p_l$.

Punching shear reinforcement is not necessary if:

$$v_{Ed1} < v_{Rd,c}$$

Where $v_{Ed1}$ exceeds $v_{Rd,c}$ a further perimeter check should be carried out to determine the outer perimeter $u_{out}$ beyond which shear reinforcement is no longer required.

$$u_{out} = \frac{V_{max}}{v_{Rd,c} d}$$

Where shear reinforcement is required, it is should be calculated using the following expression:

$$v_{Rd,cs} = 0.75 \frac{v_{Rd,c}}{u_1 d} + 1.5 \frac{(d/s_r) A_{sw} f_{ywd,ef}}{1/u_1 d} \sin \alpha,$$

where $EC2 (6.52)$

- $A_{sw}$ is the minimum area of one perimeter of shear reinforcement around the column
- $s_r$ is the radial spacing of perimeters of shear reinforcement and should be limited to 0.75 $d$
- $f_{ywd,ef}$ is the effective design strength of the punching shear reinforcement, according to $f_{ywd,ef} = 250 + 0.25d \leq f_{ywd}$
- $f_{ywd}$ is the design yield strength of the shear reinforcement = $f_{sy} / 1.15$
- $f_{sy}$ is the yield strength of the shear reinforcing steel as per AS 3600
- $d$ is the mean effective depth of the slab
- $\alpha$ is the angle between the shear reinforcement and the plane of the slab
- $u_1$ is the basic shear perimeter
- $v_{Rd,c}$ is the shear strength of the slab without shear reinforcement as calculated in $EC2 (6.47)$
Once $A_{sw}$ is determined, the reinforcement is placed around the column at successive perimeters spaced at a maximum of 0.75d, up to a distance of 1.5d from the outer perimeter $u_{out}$. The first perimeter should not be located at a distance greater than 0.3d from the face of the column. The following rules for the shear reinforcement should be observed:

![Diagram of shear reinforcement]

*Shear reinforcement detailing requirements*

Note that $A_{sw}$ is a minimum area, and additional reinforcement should be added as required in the subsequent perimeters to satisfy the requirement above. Furthermore, at least two perimeters should be provided.

**Splice crack width check**

The effect of the increased cover to the reinforcement at the splice, when the splice is in a high stress area should be checked for maximum crack width.

An initial analysis of the strains induced by the service moment at the splice can be carried out to determine the minimum area of steel required at the splice. Effectively, the designer can simply check that the maximum strain caused by the service moment at the splice is not greater than the strain caused by the maximum service moment of the element taken at the depth of the splice steel.

![Diagram of splice crack width check]
\[ \varepsilon_{s,\text{splice}} < \varepsilon_{s,\text{max}} \cdot \frac{(d_s - d_n)}{(d - d_n)} \] , where

- \( \varepsilon_{s,\text{splice}} \) is the strain at the depth of the splice steel at the caused by the service moment at the splice
- \( \varepsilon_{s,\text{max}} \) is the maximum strain of the main steel caused by the service moment
- \( d_s \) is the depth to the splice steel
- \( d \) is the depth to the main steel
- \( d_n \) is the depth of the elastic neutral axis

Alternatively, the maximum crack width can be determined using the expression proposed by Gergely & Lutz (1968) and adopted by the ACI Code and reproduced by Rangan et al in "Concrete Structures", p265. This should be limited to 0.3mm as recommended by BS 8110.

\[ w_{\text{max}} = 0.0132 \cdot z \leq 0.3 \text{mm}, \text{ where} \]

- \( z \) is a factor \( z = ( h \cdot A_b )^{1/3} \cdot \sigma_{st} \cdot 10^{-3} \)
- \( h \) is the cover to the outermost bar - here the cover to the splice bar
- \( A_b \) is an effective concrete area surrounding each bar \( A_b = ( 2 \cdot b \cdot t ) / \text{( No bars / m width)} \)
- \( b \) is 1000 (1m width)
- \( t \) is twice the cover to the bar
- \( \sigma_{st} \) is the steel stress under the service moment

References
6. Investigation of BubbleDeck Slabs, Professor Dr. Ing. Martina Scnellenbach-Held, Darmstadt University.