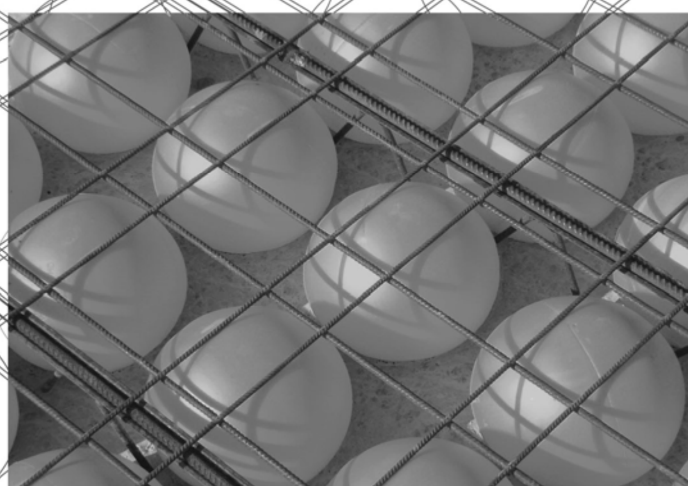
BubbleDeck Voided Flat Slab Solutions



Technical Manual & Documents

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Contents:

BubbleDeck
Tests and Reports Summary

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Results from Tests, Studies and Reports

Tests have been carried out in Denmark, the Netherlands and in Germany.
The conclusions are unambiguous:

BubbleDeck will distribute the forces in a better way (an absolute optimum) than any other hollow floor structures.
Because of the three-dimensional structure and the gentle graduated force flow, the hollow spheres will have no negative influence and cause no loss of strength.

BubbleDeck behaves like a spatial structure - as the only known hollow concrete floor structure.

The tests reveal that the shear strength is even higher than presupposed.
This indicates a positive influence of the balls. Furthermore, the practical experience shows a positive effect in the process of concreting – the balls cause an effect similar to plasticiser additives.

All tests, statements and engineering experience confirm the obvious fact that BubbleDeck:-

in any way acts as a solid deck – and therefore

will follow the same rules/regulations as a solid deck (with reduced mass), and further,

leads to considerable savings

Contents

A. Bending Strength and Deflection Behaviour

Report from the Eindhoven University of Technology / the Netherlands – Enclosure A1.

The dissertation is executed hand in hand with the Technical University of Delft / the Netherlands.

BubbleDeck is compared to a solid deck both practically and theoretically.
The behaviour is exactly as for a solid deck, both for short- and long-term situations.
The results are shown for the deck thickness 230 mm (type 1) and 455 mm (type 2).

Test Report ("Biegung") from the Technical University of Darmstadt / Germany - Enclosure A2.

The results from practical tests are compared with a theoretic analysis, concluding consistency between theory and practice.
The differences in deflections are very small, and explained by a slight difference in stiffness:

*In % of a **solid** deck*	**BubbleDeck vs. solid deck**		
	Same strength	Same bending stiffness	Same concrete volume
Strength	*100*	105	150 *
Bending stiffness	87	*100*	300
Volume of concrete	66	69	*100*

* On the condition of the same amount of steel. The concrete it self has 220 % greater effect.

Based on the report, articles have been published in the: *"Darmstadt Concrete" (Annual journal on Concrete and Concrete Structures) - Enclosure A3*, summarising several tests.

B. Shear Strength

The results of a number of practical tests confirm - once again the obvious - that the shear strength depends only on the effective mass of the concrete.
For calculating, a factor 0.6 is used on the shear capacity for a solid deck of identical height.
This guarantees a large safety margin.

Report from AEC Consulting Engineers Ltd. / Professor M.P. Nielsen - The Technical University of Denmark – Enclosure B1.

The shear strength as well as punching shear was tested.
The tests were carried out on test elements with thickness on 188 mm.
The shear capacity was measured for the ratio of a/d (distance from imposed force to support divided by deck thickness) on 1.4.

Shear strength (bending)
The shear capacity is measured to 81 % compared to a solid deck.

Punching shear
The average shear capacity is measured to 91 % compared to the calculated values of a solid deck.

Report from A+U Research Institute / Professor Kleinmann - the Eindhoven University of Technology / the Netherlands – Enclosure B2.

A solid deck is compared with two types of BubbleDeck – one with loose girders (type L) and one with secured girders (type V) – deck thickness of 340 mm.
The shear capacity is measured for two ratios of a/d (distance from imposed force to support divided by deck thickness). The results were:

Shear capacity (in % of solid deck)	a/d = 2.15	a/d = 3.0
Solid deck	*100*	*100*
BubbleDeck, secured girders	*91*	*78 (81)*[1]
BubbleDeck, loose girders	*77*	

[1] Corrected for test-elements with longer time for hardening

"Darmstadt Concrete" (Annual journal on Concrete and Concrete Structures) - Enclosure B3.

Based on the test report ("Biegung") from the Technical University of Darmstadt in Germany (Enclosure A2). a/d = 3.7
The shear capacity is measured to 72 – 78 % of a similar solid deck.

Report "Optimising of Concrete Constructions" / John Munk & Tomas Moerk - The Engineering School in Horsens / Denmark – Enclosure B4.

A solid deck is compared with BubbleDeck – no girders were used (only binding wire) in the samples – deck thickness of 130 mm.
The shear capacity is measured for a ratio of a/d (distance from imposed force to support divided by deck thickness) of a/d = 2.3. The results were:

Shear capacity (in % of solid deck)	a/d = 2.3
Solid deck	*100*
BubbleDeck, no girders	*76*

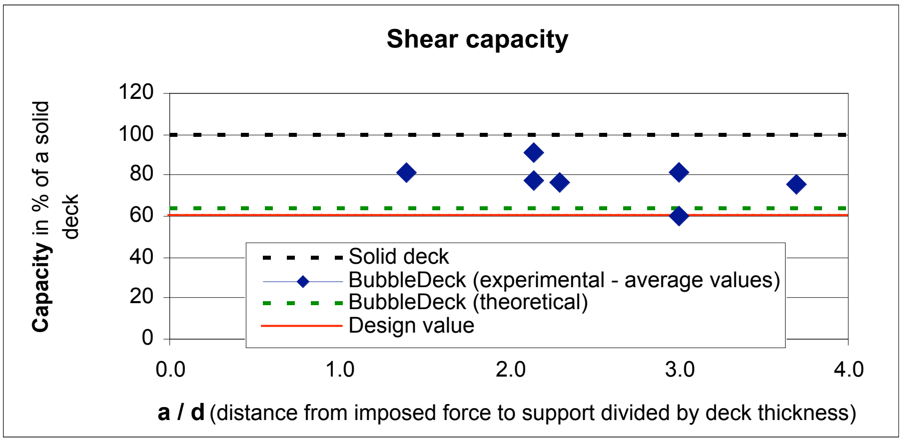
Punching Shear

Report from AEC Consulting Engineers Ltd. / Professor M.P. Nielsen - The Technical University of Denmark – Enclosure B1. The average shear capacity is measured to 91 % compared to a solid deck.

"Darmstadt Concrete" (Annual journal on Concrete and Concrete Structures) - Enclosure B3.

Tests were conducted with BubbleDeck slabs of 230 and 450 mm.
Local punching did not occur. The crack pattern was similar to the crack pattern of solid decks.

The test results are summarized in the following graph:



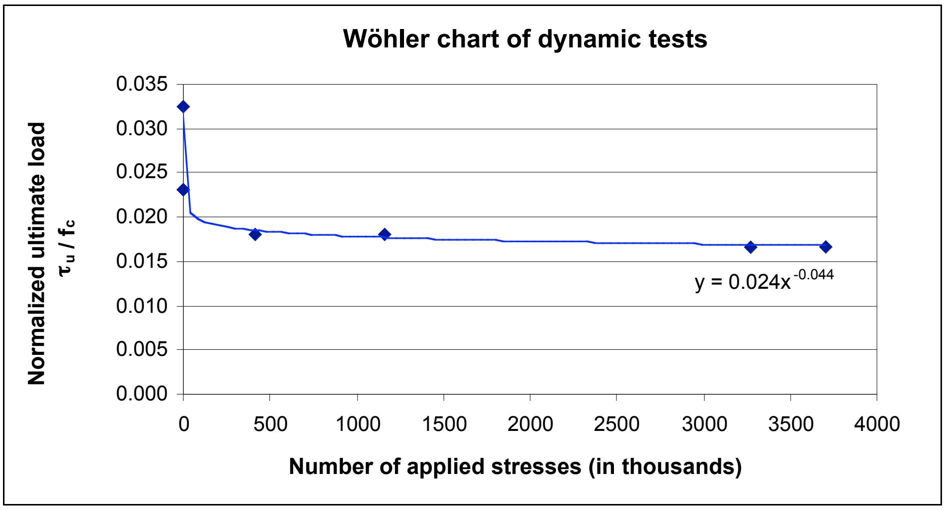
Dynamic Punching Shear

Report, "Punching Shear Strength of BubbleDeck" by Anders Brønden, Jens Christian Haukohl and Martin Hoft Jørgensen - The Technical University of Denmark – Enclosure B5.

The normalized ultimate load is defined by:

$$\frac{\tau_u}{f_c} = \frac{P}{u \cdot h \cdot f_c} \quad ; \quad \text{control perimeter, } u = \pi \cdot (d + h)$$

which, used on the experimental results, gives the following Wöhler chart:



C. Anchoring

Test Report by Koning & Bienfait b.v. / The Netherlands – Enclosure C1.

Three test blocks of BubbleDeck are compared with three test blocks of a solid deck – same reinforcement. The anchoring in the two types is identical.
The balls do not influence the anchoring.

Report BYG·DTU R-074 2003 by Gudmand-Høyer - Technical University of Denmark – Enclosure C2

A guidance to calculate the Moment Capacity in a BubbleDeck joint.

D. Fire

TNO-Report for the Weena Tower / Rotterdam – Enclosure D1

330 mm deck is fire-safe 60 minutes with concrete cover 20 mm.

TNO-Report - Enclosure D2

230 mm deck is fire-safe in 120 minutes with concrete cover 35 mm.

TNO is waiting for the CUR-Committee presenting new proposals to concrete regulations considering BubbleDeck. As soon as CUR present their conclusions for BubbleDeck, TNO will change their recommendations.

Until then TNO recommends to follow the present "hollow core" regulations, saying:

Separation: No separation due to fire for at least 120 minutes.

Bending moment: Satisfactory up to a span of 6.6 m with reinforcement ø08/ø10. The span can be increased following an increase in steel dimensions.

Shear stress: No problems are expected.

TNO admits that BubbleDeck is closer to the solid deck than to the "hollow core" deck.

A draft calculation over the internal pressure from heated air - Enclosure D4

Heated air imposes no influence (no danger) on the construction.

German Test Certificate Number P-SAC 02/IV-065 according to DIN 4102-2 - Enclosure D3

The German "Materialforschungs- und Prüfungsanstalt für das Bauwesen Leipzig e.V." has issued the German Test Certificate Number P-SAC 02/IV-065 concerning Fire Resistance according to DIN 4102-2 (in accordance with ISO 834-1).

Design recommendation: The minimum concrete cover of the lower reinforcement as a function of the period and the fire resistance:

Steel stress	Steel utilization	**Fire resistance (min)**				
		30	60	90	120	180
190	66 %	17 mm	17 mm	17 mm	17 mm	---
286	100 %	17 mm	29 mm	35 mm	42 mm	55 mm

E. Creep

Report from the Eindhoven University of Technology / the Netherlands – Enclosure E1.

No significant difference between BubbleDeck and a solid deck.

"Darmstadt Concrete" (Annual journal on Concrete and Concrete Structures) - Enclosure E2.

No significant difference between BubbleDeck and solid deck.
Differences can be due to the fact that the tests only were considered in a one-way-span.

F. Sound

Report from Adviesbureau Peutz & Associes bv: Comparison of BubbleDeck vs.Hollow core – Enclosure F1.

A comparison was made between BubbleDeck and hollow core deck prior to the construction of Weena Tower. Deck types of similar height were compared.
The noise reduction with BubbleDeck was 1 dB higher than hollow core.

The main criteria for reducing noise is the weight of the deck and therefore BubbleDeck evidently will not act otherwise than other deck types with equal weight.

German Test Certificate Number P-SAC 02/IV-065 concerning solid and live load Sound insulation – Enclosure F2.

The German "Materialforschungs- und Prüfungsanstalt für das Bauwesen Leipzig e.V." has issued the German Test Certificate Number P-SAC 02/IV-065 concerning solid and live load Sound insulation according to DIN EN ISO 140 / DIN ISO 717.

The results for 230 and 340 mm decks are:

	Sound insulation dimension	Additional spectrum adaptation values (DIN ISO 717-1)						Standard impact sound level
Deck	R_w (C;Ctr)	$C_{50\text{-}3150}$	$C_{tr50\text{-}3150}$	$C_{50\text{-}5000}$	$C_{tr50\text{-}5000}$	$C_{100\text{-}5000}$	$C_{tr100\text{-}5000}$	$L_{c,w\ (C1;C50\text{-}3150)}$
mm	dB	dB	dB	dB	dB	dB	dB	dB
230	55 (-2 ; -7)	-2	-8	-1	-10	-1	-8	78 (-11 ; -12)
340	57 (-2 ; -7)	-3	-9	-2	-7	-2	-9	76 (-13 ; -13)

Test Report from Adviesbureau Peutz & Associes b.v.: Sound Resistance. March 2004 - Enclosure F3.

Field tests in a raw building in Leiden, the Netherlands, concerning “Air and Contact Noise-resistance”. The slabs were BD 230 mm with a fixed floor layer of 30 mm.
The measurements and ratings were carried out in regulation with ISO 717-1:1996 and NEN 5077:2001

Weighted Sound Reduction (vertical)	R'_w (C;Ctr)	=	54 (-1; -14)
Reduction Index for Airborne Sound	I_{lu}	=	+ 3
Impact Resistance Level (vertical)	$L'_{n,Tw}(C_1)$	=	72 (-14)
Reduction Index for Impact Sound	I_{co}	=	+ 2
Impact Resistance Level (horizontal)	$L'_{n,Tw}(C_1)$	=	63 (-13)
Reduction Index for Impact Sound	I_{co}	=	+ 10

Test Report from Ian Sharland Ltd : Airborne and Impact Sound Insulation. Nov 2005 - Enclosure F4.

Field tests in Le Coie Housing Development in St. Helier, Jersey, concerning “Airborne and Impact Sound Insulation”. The slabs were BD 285 mm, part of a standard party floor with ceiling and screed. The measurements and ratings were carried out in regulation with ISO 140-4:1998, ISO 140-7:1998, ISO 717-1:1997 and ISO 717-2:1997.

The Vertical Impact (mean) was measured to:	L_{nTw}	=	44 dB
The Vertical Airborne (mean) was measured to:	D_{nTw}	=	59 dB

The results show that the floor structures tested meet and significantly exceed the requirements of the British Building Regulations (2000)

G. In General

Report from the Eindhoven University of Technology / the Netherlands -
Broad comparison of concrete floor systems. December 1997 – Enclosure G.

The advantages of BubbleDeck were not known at this early stage but nonetheless, BubbleDeck ended on the top of the list.
Afterwards it is recognised that "Flexibility", "Execution" and "Environment" are at a higher level (giving BubbleDeck at least 4 more points in this comparison).

H. Comparison of Cost Price

Report from the Eindhoven University of Technology / the Netherlands – Enclosure H1.

In connection to the general tests, a total cost price calculation of the Town Hall in den Haag is carried out. The Town Hall was built with pre-stressed monolithic elements. The complete construction has been evaluated in order to make a reliable comparison.

Two types of comparisons were made:

1. BubbleDeck and a solid deck was compared in three various arrangements - alteration of placement of columns. The calculations were made for increasing spans in the x-direction. For a given combination of span and deck thickness, BubbleDeck was 5 – 16 % less expensive than a solid deck.

   It is important to emphasize that the optimal combination of deck thickness and placement of columns with BubbleDeck differs from a solid deck. A correct comparison must take this fact into consideration, which was made in the second comparison:

2. Two variants of BubbleDeck were entered into the comparison.
   The result was clear – the BubbleDeck building was significant less expensive than the traditional system. The total savings was in the order of 20 %.

Report from AEC Consulting Engineers Ltd. / Professor M.P. Nielsen - The Technical University of Denmark – Enclosure H2.

Comparisons are made between BubbleDeck and solid decks.
Only differences in materials concerning the slabs are considered. Advantages in the building design and building process are *not* taken into account.

For the same amount of steel and concrete, BubbleDeck has 40 % larger span and is furthermore 15 % cheaper.

For the same span, BubbleDeck reduces the amount of concrete with 33 %, and reduces the price with 30 %.

Miscellaneous

Savings
Savings can be expected in many respects. In the case of Weena Tower, the experience shows a reduction of 35 % in necessary time using the cranes. A very important aspect, especially concerning tall buildings, because of the large amount of downtime due to wind.

Furthermore, the erection-cycle was reduced from 5 to 4 days per storey. This is confirmed through the following projects where reductions of 1-2 days per storey were obtained.

Subsequent works are simplified. Savings can also be expected throughout the buildings lifetime due to the high degree of flexibility.

Approval by Authorities

Dutch Standards

From November 2001, BubbleDeck is incorporated in the Dutch Standards (by CUR – Civieltechnisch Centrum Uitvoering Research en Regelgeving) under CUR Recommendation 86 – BubbleDeck slabs.

UK Standards

BubbleDeck can be treated as a normal flat slab supported on columns (BS 8110) according to CRIC (Concrete Research & Innovation Centre under the Imperial College of Science, Technology & Medicine), 1997.

Danish Standards

BubbleDeck can be calculated from recognized principles and within existing standards - Directorate of Building and Housing, Municipality of Copenhagen, 1996.

German Standards

BubbleDeck can be used according to existing technical standards according to Deutsches Institut für Bautechnik, 1994.

Conclusion

BubbleDeck will distribute the forces in a better way (an absolute optimum) than any other hollow floor structures.
Because of the three-dimensional structure and the gentle graduated force flow the hollow areas will have no negative influence and cause no loss of strength.

BubbleDeck behaves like a spatial structure - as the only known hollow concrete floor structure.

The tests reveal that the shear strength is even higher than presupposed.
This indicates a positive influence of the balls. Furthermore, the practical experience shows a positive effect in the process of concreting – the balls cause an effect similar to plastification additives.

All tests, statements and engineering experience confirm the obvious fact that BubbleDeck,

> ***in any way act as a solid deck*** – and consequently
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> ***will follow the same rules/regulations as a solid deck*** (with reduced mass), and further
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> ***leads to considerable savings***

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Appendix 1 - Tests and Reports References

A1 Report from the Eindhoven University of Technology / the Netherlands
A2 Test Report ("Biegung") from the Technical University of Darmstadt / Germany
A3 "Darmstadt Concrete" (Annual journal on Concrete and Concrete Structures)

B1 Report from AEC Consulting Engineers Ltd. / Professor M.P. Nielsen - The Technical University of Denmark
B2 Report from A+U Research Institute / Professor Kleinmann - the Eindhoven University of Technology / the Netherlands
B3 "Darmstadt Concrete" (Annual journal on Concrete and Concrete Structures)
B4 Report "Optimising of Concrete Constructions" - The Engineering School in Horsens / Denmark
B5 Report "Punching Shear Strength of BubbleDeck" - The Technical University of Denmark

C1 Test Report by Koning & Bienfait b.v. / The Netherlands
C2 Report BYG·DTU R-074 2003 - Ph.D. Gudmand-Høyer - Technical University of Denmark

D1 TNO-Report for the Weena Tower / Rotterdam / the Netherlands
D2 TNO-Report for 230 mm deck - fire-safe in 120 minutes
D3 German Test Certificate Number P-SAC 02/IV-065 according to DIN 4102-2
D4 A draft calculation over the internal pressure from heated air

E1 Report from the Eindhoven University of Technology / the Netherlands
E2 "Darmstadt Concrete" (Annual journal on Concrete and Concrete Structures)

F1 Report from Adviesbureau Peutz & Associes b.v.: Comparison of BubbleDeck vs. Hollow core
F2 German Test Certificate Number P-SAC 02/IV-065: Solid and live load Sound insulation
F3 Test Report from Adviesbureau Peutz & Associes b.v.: Sound Resistance. March 2004.
F4 Test Report from Ian Sharland Ltd : Airborne and Impact Sound Insulation. Nov 2005.

G1 Report from the Eindhoven University of Technology / the Netherlands - Broad comparison of concrete floor systems. December 1997

H1 Report from the Eindhoven University of Technology / the Netherlands
H2 Report from AEC Consulting Engineers Ltd. / Professor M.P. Nielsen - The Technical University of Denmark

BubbleDeck Voided Flat Slab Solutions

Technical Paper

BubbleDeck® Slab Properties

April 2006

This BubbleDeck® Technical Paper has been written to provide more detail on common issues raised by Engineers covering 1) BubbleDeck slab thickness, 2) Shear properties near columns, 3) Rebar quantity, and 4) Deflections.

1. BubbleDeck® increases slab thickness over and above alternative solutions

In fact BubbleDeck consistently realises thinner overall structural zones than alternative structural solutions. Given equivalent spans and loadings BubbleDeck consistently realises a thinner slab compared to a solid flat slab. The properties of a BubbleDeck slab compared to other slab types should be considered in terms of a) flexural strength, b) deflection, and c) shear which are each considered below:-

Flexural strength: BubbleDeck is conceived to omit a significant volume of concrete (compared to a solid slab) in the central core where the slab is principally un-stressed in flexure. 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 expounded in EC2 and BS8110. Sometimes, in heavily stressed slabs, the stress block will encroach slightly within the bubble zone but studies and tests have demonstrated this has an insignificant effect on the resistance of a BubbleDeck slab in normal design situations and, to control this criterion, the BubbleDeck design guide gives a check to test whether the design is within this valid range. Therefore a BubbleDeck slab is equivalent, in terms of concrete stress, to a solid slab under all normal building loadings.

It is important to note the voids formed by the bubbles are not prismatic as in hollow core systems – they are discrete volumes in a 2 dimensional array so they do not detract from the slab strength and stiffness in the way that a prismatic void would.

In a section at or approaching flexural ULS, the section will be fully cracked and the concrete below the plastic neutral axis will be ineffective in any slab. Therefore this cannot be a source of supposed disadvantage. If the Engineer plots a stress and strain diagram through a typical BubbleDeck slab section, it will be seen the redundant concrete (i.e. that which is not stressed) occurs wholly or almost wholly in the bubble zone. Since typically 30% or more of the weight of the slab has been omitted, this can be used in the design to justify either higher applied loadings or longer spans so, in terms of the slab system, BubbleDeck is substantially more effective than a solid slab – flat slab or otherwise.

Deflection: Due to the bubbles a BubbleDeck slab is not as stiff as a solid slab – but this effect is small. Studies and tests have shown that BubbleDeck has approximately 87% of the flexural stiffness of a solid slab. If no other measures were taken, this would mean marginally higher deflections at SLS than in an equivalent solid slab in direct proportion to this ratio.

However, the effect can be compensated for by adding a modest amount of steel even though the deflection is significantly mitigated by the fact that BubbleDeck is lighter and in long term SLS, where frequently the load combination comprises 100% permanent load and a proportion such as 33% imposed load, the permanent weight saving maximises BubbleDeck's effect. Long term SLS is frequently the governing criteria for flat slab designs.

Calculated long-term SLS combinations therefore favour BubbleDeck and frequent combination imposed-load-only deflections can, if they are critical, be dealt with by a modest addition of steel (or even be ameliorated by rigorous design). It is therefore apparent BubbleDeck's substantial reduction of permanent load, considering deflection criteria, gives a substantial advantage compared with alternative solid slab.

Final design of BubbleDeck projects almost always incorporates 'rigorous' deflection checking using FE modelling with modified element properties to take account of cracked section properties and long-term material effects. This technique is well known to produce more accurate results and these generally produce more favourable results for BubbleDeck slabs compared with simplistic calculations.

Shear: In any flat slab, design shear resistance is usually critical near columns. The shear stresses remote from the columns diminishes rapidly and outside the column zones it has been demonstrated by testing and calculation the transverse and longitudinal shear stresses are within the capacity of the BubbleDeck system.

Near the columns, bubbles are left out so in these zones a BubbleDeck slab is designed exactly the same way as a solid slab. The BV girders incorporated within a BubbleDeck slab add to the longitudinal shear resistance of the slab and it has been demonstrated the rate of change in bending moment near supports does not induce shear stresses in excess of the interface shear capacity. This is part of the design process in critical cases.

2. How is shear accounted for & calculated near columns.

This follows on from the answer to the last issue. Bubbles are omitted around the columns in an area, rectangular circular or oval as convenient, which is defined by the punching shear perimeter where the applied shear stress is exceeded by the capacity of the BubbleDeck voided slab. The shear capacity of the voided slab has been shown to be approximately 2/3 the capacity of an equivalent solid section – in design a conservative factor of 60% is usually employed.

It must be noted again that the voids formed by the bubbles are not prismatic.

Within the perimeter so defined, BubbleDeck is left solid without bubbles and the shear resistance is calculated on the basis of a solid slab in the code dependent customary way. Shear reinforcement may be added, exactly as in a solid slab. Given equivalent spans, BubbleDeck has the capacity to produce lighter column loads and alleviate shear stresses. However, the advantages of BubbleDeck are usually employed to maximise spans well beyond what can be achieved with a solid slab.

As stated above, in critical situations, the interface shear is checked and the interface can be reinforced more heavily by adding additional girders if necessary. The applied shear is obtained, as explained in EC2 or BS8110, by calculating the forces each side of the interface from the rate of change of bending moments at each end of the segment considered and averaging this to deduce a longitudinal force per unit length. Both BS8110 and EC2 have methods for determining the shear resistance of interfaces between 'concrete poured at different times'.

3. BubbleDeck® needs more rebar than alternative solutions

One of the greatest advantages of BubbleDeck is that it removes the need for much, often poorly controlled, site operations by fabricating a large proportion of the slab off site in a factory under controlled conditions and using production techniques that are, through organised process, far more productive than site work. This leads to large units that are simply transported to site, lifted into position and the concrete poured.

However, for practical manufacturing, handling and transport reasons there is a limit to the size of the units and part of the BubbleDeck system is the splicing together of elements across the joints (retaining BubbleDeck's 2 way spanning characteristic) along the sides and ends of the precast individual elements, which is done using straight rebar and mesh. This does represent a modest increase in rebar use over that strictly required for overall slab strength and stiffness. However BubbleDeck, through its weight saving and rigorous design, is relatively frugal with steel and very frugal with concrete and this more than compensates for the small addition of splice bars.

With BubbleDeck PT very modest rebar content can be realised as the advantages of post tensioning combined with weight saving are a winning combination.

4. Deflections – justify modifying the BS8110 span/depth criteria

The modification of span / depth ratios given in BubbleDeck's product literature and presentations is only used for preliminary design scoping, to reflect the longer achievable spans resulting from BubbleDeck's lower slab dead weight. We do point out the adjusted ratios are only an approximate guide, useful at feasibility stage for determining appropriate spans for a given BubbleDeck slab depth. Experience has shown this 'rule of thumb' adequately reflects the results following full engineering calculation of a BubbleDeck slab.

Beyond preliminary design scoping we do not rely on span-depth ratio methods (as is the case for flat slab design) except for slabs of minor importance or for cursory checks. A BubbleDeck slab should be calculated in the normal way as far as deflection is concerned. Certainly, any irregularity of spans in a flat slab design would largely invalidate span-depth ratio methods. Tests carried out at the full scale Cardington European Concrete Building Project demonstrate the poor correlation of results from various methods of deflection calculation with measurements on a real structure.

Again it must be noted the voids formed by the bubbles are not prismatic so one must not assume that the second moment of area of the un-cracked section is significantly depleted by the presence of prismatic voids.

During calculation of a BubbleDeck slab there is no modification to the span depth ratios recommended in BS8110 or EC2 except that the normal span depth ratio should be adjusted to reflect the 87% reduction in flexural stiffness ("EI" stiffness). In BS8110, the basic ratios are used with modifiers for overprovision of tensile and compressive reinforcement which affect the steel service stress. In EC2, the basic ratios are also used and the modifiers obtained from equations 7.16a & 7.16b which operate on the reinforcement ratios and concrete modulus implied by the characteristic concrete strength.

When calculating deflection rigorously, it should be noted that the recommended level of cracking moment in BubbleDeck is slightly lower (80%) than in an equivalent solid slab. This does not affect ULS calculation since the section is fully cracked in that case.

Eur Ing R A Beeton BSc CEng MICE, BubbleDeck's Design Engineer, April 2006

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BubbleDeck
Engineering Design & Properties Overview

June 2007

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Design of BubbleDeck slabs

Before a detailed explanation, it should be pointed out that fully documented tests and many completed projects have demonstrated that BubbleDeck can be designed exactly as a solid slab with very few differences. The differences in terms of shear and deflection can be summarized as follows:-

1. There is less weight than a solid slab so deflection is less.
2. The flexural stiffness is approximately 90% of a similar thickness solid slab but this is overwhelmingly compensated by the weight reduction in terms of deflection.
3. It is recommended that a crack moment of 80% of a similar thickness solid slab is used.
4. Shear resistance of the solid zone (through the balls) is conservatively taken as 60% of a similar thickness solid slab (67% or more is demonstrated in tests).

Shear:

The shear resistance of BubbleDeck is (a slightly conservative value for design purposes) 0.6 times the shear resistance of a solid slab of the same thickness. If this is exceeded by the applied shear, at a column for example, we leave out the balls and use the full solid shear values. Using Eurocode 2 (or any other code for that matter, with slight differences) we would calculate the applied shear at 2d and subsequent perimeters from the column face as per the code requirements, as well as at the column face itself. We would then compare this to our calculated resistance.

- If the applied shear is less than the un-reinforced hollow slab resistance, no further check is required.
- If the applied shear is greater than the hollow slab resistance we omit balls and make it solid then check the solid part.
- If the resistance is still greater than the solid slab resistance and less than the maximum allowed, we provide shear reinforcement.

All this is exactly as solid flat slab design. Additionally we place bottom bars as per CIRIA report R89 and R110, designed to protect against progressive collapse.

Some typical figures for perspective: As an approximate or typical example of ultimate resistance values, a BD280 slab with about 1% tensile reinforcement would have a shear resistance, v_{RDc}, of approximately 0.72MPa and, with an effective depth of 243mm would produce 175N/mm in the solid part. This would reduce to about 105N/mm in the hollow part with balls. The maximum allowed v_{RDmax} at the column face would be 5.3MPa. With a small amount of shear reinforcement, such as Lenton Steel Fortress, shear stud rails or rebar, at the rate of 75mm^2 at 200c/c each way, the shear resistance would rise to about 540N/mm.

Deflection:

Span depth ratio calculations for deflections are very approximate and are not appropriate in flat slabs of irregular layout except for the most simple or unimportant cases. We use FE modelling, including non-linear cracked section analysis to calculate the deflection using normal structural concrete with a Young's Modulus (secant) E_{cm}, multiplied by 0.9 (see above) and a tensile strength, f_{ctm} multiplied by 0.8 (to reduce the crack moment as mentioned above).

Where accurate deflections are required, the software runs iteratively, calculating modified and cracked element properties at each stage of the iteration, using the applied reinforcement, until convergence is reached. The deflection using this method has been shown to have good agreement with tests conducted at the ECBP at Cardington (see Concrete Society Technical Report TR 58).

Short term loadings are usually patterned (chequerboard or parallel strips depending on the characteristics of the project) using the "Frequent Combination" set out in Eurocode 0 and using combination factors "ψ_1" as appropriate. For long term loading, the "Quasi-permanent combination" is used with combination factors "ψ_2" as well as creep coefficient and shrinkage curvature parameters. All this procedure is exactly as would be used with a rigorous design of an ordinary in-situ flat slab structure.

Some typical figures for perspective: As an approximate or typical example an office building with a design life of 50 years, we may use "Category B" and ψ_0 = 0.7, ψ_1 = 0.5, ψ_2 = 0.3. The creep coefficient may typically (depending on environment, type of concrete and constituents) be in the range 1.8 to 2. Shrinkage curvature *effects may be calculated using a free shrinkage strain, again depending on environment and materials, of 0.045%.*

A further feature of the weight reduction in BubbleDeck slabs is that the effect on long-term loading is more pronounced since the imposed loading is multiplied by ψ_2 and thus the permanent load reduction has a comparatively greater beneficial effect in the calculation.

Contact between Bubbles & Reinforcement

The potential for any contact is only theoretical because the balls do not perfectly fit between reinforcement bars and moves slightly during assembly / site concrete compaction so that some grout surrounds it and provides a measure of passivation. However, even if there were contact between the ball and the steel, the environment inside the void is very dry and protected - there is also no breach (apart from micro cracking) of the concrete to the outside air. It is a better situation than exists with inclusion of plastic rebar spacers within solid slabs that create a discontinuity within the concrete between the outside air and the rebar in solid r.c. slabs. We therefore have a situation that is better than existing with plastic rebar spacers and these have been permitted for many years. We have had balls cut open in Holland and Denmark and there has been no sign of significant corrosion.

Effect of Bubble Voids upon Stiffness

Unlike hollow core units, BubbleDeck voids are discrete balls and not prismoidal voids running the length of the span. This makes a huge difference to the performance compared to hollow core sections. Tests carried out in Denmark, Germany and Holland (See Reports A1 & A2) show that the flexural stiffness is approximately 87% to 93% of the same thickness of solid slab - In design we use an average of 90% and, in addition, we factor the cracking moment by 80% as recommended in Dutch research. In fact one of the major benefits of the system is its virtue of reducing deflections for a given span because the one-third weight reduction overwhelmingly more than compensates for the very small reduction in stiffness.

Flexural Strength

In slabs, the depth of compressed concrete is usually a small proportion of the slab depth and this means that it almost always involves only the concrete between the ball and the surface so there is no sensible difference between the behaviour of a solid slab and BubbleDeck. The only elements working are the outer 'shell' of concrete on the compression side and the steel on the tension side.

In terms of flexural strength, the moments of resistance are the same as for solid slabs provided this compression depth is checked during design so that it does not encroach significantly into the ball (a 20% encroachment has been shown to be insignificant).

Shear Strength

Incorporating bubble void formers significantly affects shear strength and that is why balls are omitted in areas where the slab sustains shear stresses greater than the hollow slab resistance. Here the strength reduction used in design is 60% (test show it is actually 67% or more – see reports B1 to B4). Where the balls are omitted we revert to normal solid slab shear resistances (this applies to punching and transverse shear as well as in-plane shear where this may be an issue).

For these reasons, it is demonstrated that the design may be carried out in every way treating the slab as a solid slab, with the provisos mentioned above, which are all taken account of in the design process. We therefore use Eurocode 2, which is fully compatible with the system, for our design and which is somewhat more up to date than BS8110. We usually carry out the design from finite element software and include non-linear cracked section analysis where appropriate but we also use manual methods, such as yield line theory, in preliminary design and checking.

Durability

The durability of BubbleDeck slabs is not fundamentally different from ordinary solid slabs. The concrete is standard structural grade concrete and; combined with adequate bar cover determined in accordance with EC2 or BS8110; is what provides most control of durability commensurate with normal standards for solid slabs. When the filigree slabs are manufactured, the reinforcement module and balls are vibrated into the concrete and the standard and uniformity of compaction is such that a density of surface concrete is produced which is at least as impermeable and durable, arguably more so, to that normally produced on site.

BubbleDeck joints have a chamfer on the inside to ensure that concrete surrounds each bar and does not allow a direct route to air from the rebar surface. This is primarily a function of the fire resistance but is also relevant to durability.

Cracking in BubbleDeck slabs is not worse, and probably better, than solid slabs designed to work at the same stress levels. In fact BubbleDeck possesses a continuous mesh, top and bottom, throughout the slab and this ensures shrinkage restraint is well provided for and that cracking is kept to a minimum whether it is intrinsic or extrinsic cracking.

Unlike an off-the-shelf product, this is a system that is bespoke designed for each and every project. All the peculiarities of a project are therefore taken into account in the design, therefore there is no risk of the product being misused by applying it to uses for which it is not intended.

Technical Certification & Approval by Authorities

European Standards & Technical Approval (ETA)

CUR Recommendation 86 – December 2001

BubbleDeck has been granted its own standard as a supplement to the Dutch National Building Standards, " *CUR Recommendation 86 – BubbleDeck slabs* " by CUR – Centre for Civil Engineering Research and Codes. CUR Recommendation 86 is supplementary to Dutch NEN 6720 – equivalent to Eurocode 2 for Concrete Structures.

The BubbleDeck floor system itself and its structural behaviour was reviewed by a CUR Committee of independent and respected experts from consultants, contractors and governmental authorities. The CUR is the highest independent and well-respected authority in the Netherlands in this field. The CUR Committee reviewed all executed research over the world and has initiated experimental research, that is executed by the Technical University of Eindhoven under supervision of these experts. All executed research and the results are published in official technical reports. The results were used to write the official supplementary standards for BubbleDeck, published as CUR-Recommendation 86.

See CUR-86 translation and list of supporting reports.

Kiwa KOMO-Certificate K22722/01 – November 2002

BubbleDeck has received Technical Approval from Kiwa N. V., who are an official EOTA Member. The KOMO Certificate is based upon the most important standard for the design of concrete structures: this is the Dutch Code NEN 6720 with common design rules and CUR-Recommendation 86 with supplementary rules for BubbleDeck floors, the equivalent code of Eurocode 2 for Concrete Structures.

See KOMO-Certificate K22722/01 translation.

The status of the several standards is as follows: NEN 6720 is still formally in force until the moment that Eurocode 2 is formally introduced as final standard in EC Countries. It is expected this will not occur earlier than 2010. At that time NEN 6720 will be withdrawn alongside equivalent national standards in other EC Countries. When Eurocode 2 is brought into force CUR Recommendation 86 will be checked for consistency with Eurocode 2 and BubbbleDeck NL expect the technical content of CUR-Recommendation 86 will not need to be changed. When Eurocode 2 is introduced as formal standard the content of the KOMO-certificate will be adjusted to reflect the new standard.

UK Standards

BubbleDeck can be treated as a normal flat slab supported on columns (BS 8110) according to CRIC (Concrete Research & Innovation Centre at the Imperial College of Science, Technology & Medicine), 1997.

Danish Standards

BubbleDeck can be calculated from recognized principles and within existing standards - Directorate of Building and Housing, Municipality of Copenhagen, 1996.

German Standards

BubbleDeck can be used according to existing technical standards according to Deutsches Institut für Bautechnik, 1994.

Results from Tests, Studies and Reports

BubbleDeck will distribute the forces in a better way (an absolute optimum) than any other hollow floor structures. Because of the three-dimensional structure and the gentle graduated force flow the hollow areas will have no negative influence and cause no loss of strength. BubbleDeck behaves like a spatial structure - as the only known biaxial hollow concrete floor structure.

Tests carried out in Denmark, the Netherlands and in Germany reveal that the shear strength is even higher than presupposed. This indicates a positive influence of the balls. Furthermore, the practical experience shows a positive effect in the process of concreting – the balls cause an effect similar to plasticiser additives.

The conclusions are unambiguous:

- BubbleDeck will distribute the forces in a better way (an absolute optimum) than any other hollow floor structures.
- Because of the three-dimensional structure and the gentle graduated force flow, the hollow spheres will have no negative influence and cause no loss of strength.
- BubbleDeck behaves like a spatial structure - as the only known hollow concrete floor structure.
- The tests reveal that the shear strength is even higher than presupposed. This indicates a positive influence of the balls. Furthermore, the practical experience shows a positive effect in the process of concreting – the balls cause an effect similar to plasticiser additives.

All tests, statements and engineering experience confirm the fact that BubbleDeck:-
- ***in any way acts as a solid deck*** – and therefore
- ***will follow the same rules/regulations as a solid deck*** (with reduced mass), and further,
- ***leads to considerable savings***

From 1999 over 8 years the BubbleDeck floor system has proved itself in the Netherlands, Denmark and Belgium in many 100,000 m^2 floor slabs successfully constructed and completed in numerous buildings of all types and sizes.

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BubbleDeck Voided Flat Slab Solutions

Technical Paper

BubbleDeck® Design and Detailing Notes – guidance to engineers and detailers

October 2007

BubbleDeck - guidance to engineering designers and detailers

Engineering design

Generally:

The engineering designer should be very familiar with the principals of slab design and particularly flat slabs as well as having a good grounding in general structural engineering.

It is recommended, as a minimum, to have read a general text on flat slab design and the appropriate sections of Concrete Society Technical Report TR43[i] and TR58[ii] (Note TR43 is specifically for post tensioned slabs but there is useful and relevant material there). Essential reading also is Eurocode 2[iii] – at least the sections on flexure and shear, with particular reference to punching shear.

CIRIA Report 89[iv] and 110[v] are also important background reading (although the latter is somewhat obsolete, it contains useful material).

The engineer wishing to explore in greater depth should read Nielsen[vi]. This is especially useful text as Nielsen was an influential member of the EC2 drafting committee and to a large degree, was responsible for bringing the code up to date with recent advances in plastic theory instead of reliance on outdated empirical practices and over-reliance on elastic methods.

It is also helpful to read through the various reports of testing and studies done on BubbleDeck Slabs in Europe.

All design work should be checked or reviewed by a competent person. It is not recommended to rely on Local Authority Building Control Checking as some checking engineers lack the specialised knowledge and experience to properly check advanced RC designs.

The analysis and calculation of resistances for BubbleDeck is much the same as for ordinary slabs except for some additional criteria. It is essential that the engineering designer has an understanding of analytical manual methods, particularly yield line theory, and an understanding of the principals and application of finite element analysis. In the latter case an understanding of linear elastic and non-linear methods is necessary.

Material properties:

Shear:

The shear resistance of BubbleDeck is a slightly conservative value, taken from tests, which we use in design: 0.6 times the shear resistance of a solid slab of the same thickness. If this is exceeded by the applied shear, at a column for example, we leave out the balls and use the full solid shear values. Test conducted in Germany, Denmark and Holland have shown the resistance to vary from about 65% to 90% of a solid slab.

Flexure:

Standard strength parameters and properties are used as for solid slabs.

Deflection:

Span depth ratio calculations for deflections are very approximate and are not appropriate in flat slabs of irregular layout except for the most simple or unimportant cases. FE modelling, including non-linear cracked section analysis is used to calculate the deflection using normal structural concrete with a Young's Modulus (secant) E_{cm}, multiplied by 0.9 (see above) and a tensile strength, f_{ctm} multiplied by 0.8 (to reduce the crack moment as mentioned above – this is mainly significant in the computation of uncracked curvatures where the geometry of the concrete section is significant but is of increasingly negligible significance after cracking).

It is not presently possible to calculate for the difference in age related properties in the filigree and in-situ concrete parts. This is not considered to be a significant weakness.

Design methods:

Generally:

For ULS, elastic or plastic methods may be used to determine the applied actions. The engineer should, however, be aware of the fundamental differences between the two theories.

For flexural design, plastic theory may lead, in practice, to more efficient use of reinforcement. This is usually applied, in the case of slabs, by the yield line theory – the most celebrated exponent of this being K W Johansson. Johansson[vii] published a comprehensive work on the practical use of yield line theory as well as his original work on the theory itself. Kennedy and Goodchild[viii] have published a useful and very readable introduction to the use of yield line theory also. Yield line theory is a very powerful tool by virtue of the relatively simple procedures involved leading to economic reinforcement quantities. It is not without need for caution, however, and care needs to be excercised not to overlook SLS concerns.

The main reason for the economy of yield line design is that collapse mechanisms are found (usually in an upper bound analysis) that involve the whole, or a very large part, of a reinforcement zone in yielding since it can be shown that the whole must fail before the structure can fail globally (bearing in mind there may be many upper bound mechanisms that need to be checked). This is in contrast to elastic design, which usually results in a fairly heterogeneous moment field for which the designer attempts to fit a practical arrangement of reinforcement. In fact, Nielsen[vi] states that the elastic theory can lead to an optimal arrangement of reinforcement and, in addition, that there is no philosophical objection to the use of plastic theory in designing the reinforcement for applied actions determined from the elastic theory. It is evident from this that the use of elastic theory and, in this particular context elastic moment results for slabs, it is only significantly uneconomic if the designer is too un-conservative about how the moment result field is covered by the reinforcement provision and if there is no allowance for yielding and redistribution.

For example, FE results for a slab may show a small but irregular area of high design moment. The designer may apply rebar to this in a rationalised zone, probably rectangular, which actually extends over areas where the design moments are very low. It would be possible, with appropriate experience and judgment to adjust downwards the quantity of reinforcement so that it actually yields at the intense moments and redistributes moment to the less utilised areas. This might be checked by utilising a work equation in the same way as yield line design is carried out. It is obviously important to ensure that the work done, the dissipation, in yielding a reinforcement zone balances the work done by the external loads. In an approximation, one could check that the dissipation of the applied reinforcement exceeds that of the required reinforcement from the elastic results.

Codes and published methods often give weight to concepts of column strips and middle strips but these are usually difficult to apply in irregular slabs. TR43 gives guidance on this, for example, and suggests that the column strip is determined as 0.4 of the distance from the column centreline to the zero shear line. Some methods further divide the column strip into an inner column strips and outer column strips. It is recommended in most UK practice to concentrate most of the reinforcement, say 2/3 of that in the column strip in to the inner column strip so that the reinforcement provision will be greatest where the service moments, tending to the elastic end of the spectrum, are greatest and thus where most needed to resist cracking and limit rotation contributing to deflection. To prevent absurd concentration of rebar, one may take the reinforcement for the average moments for the inner column strip and provide this for the full width of that strip.

Another phenomenon tending to produce in economy from yield line design is that it utilises the technique of allowing support and span moments to yield according to the reinforcement chosen in such a way that the relative quantities in the top of the slab at supports and in the bottom at mid-span are optimised to what is available and practical.

There is, however, an important feature of yield line design that must not be overlooked: It design for ULS only and assumes that a collapse mechanism can exist which mobilises all the concrete and steel used. This implies that the slab is sufficiently ductile in all respects and requires that steel can reach the strains required without exceeding the ultimate strain and that the concrete does not crush or crack excessively. Furthermore, it does not check the conditions at SLS and if over-reliance is placed on ductility it can sometimes lead to excessive cracking if large rotations occur at SLS which cannot be accommodated by the rebar supplied without large tensile strains in the concrete.

There may also be an important point to observe if excessive yielding occurs where one relies on shear resistance – punching shear at columns for example. According to modified compression field theory, it can be shown that shear softening may be significant.

This is where the advantage of FE analysis and design are most significant; even if linear elastic models are used (there are non-linear plastic FE methods in existence but they are not widely used). Modern software is available which simulates non-linear behaviour, including the effects of cracking in an iterative process on the elastic stiffness method and these have been shown to give good results. Using these tools, checks on the SLS behaviour, including cracking and deflection can be carried out as well as a ULS design indicating where yield limits may occur.

The FE methods also have the advantage of combining all the static analyses and checking into one process that can be efficiently managed.

Shear:

The shear resistance of BubbleDeck is taken as 0.6 times the shear resistance of a solid slab of the same thickness. If this is exceeded by the applied shear, at a column for example, we leave out the balls and use the full solid shear values. Using Eurocode 2[iii] (or any other code for that matter, with slight differences) one may calculate the applied shear at 2d and subsequent perimeters from the column face as per the code requirements, as well as at the column face itself. This would then be compared to the calculated resistance.

- If the applied shear is less than the un-reinforced hollow slab resistance, no further check is required.
- If the applied shear is greater than the hollow slab resistance we omit balls and make it solid then check the solid part.
- If the resistance is still greater than the solid slab resistance and less than the maximum allowed, we provide shear reinforcement.

All is exactly as solid flat slab design. Additionally one places bottom bars as per CIRIA report R89[iv], designed to protect against progressive collapse – these bars may be checked using Rasmussen's dowel calculation so that they can sustain, say 75% of the accidental limit state shear force.

Punching shear, in difficult or complex cases, may also be checked using methods described in Nielsen[vi]. Indeed it is always a good idea to check using more than one method or theory as this can expose anomalies or mistakes that must be checked.

In calculating the shear resistance care and judgment should be exercised in employing formulae which include a scale factor. The scale factor in shear is real but, according to Regan[iv], there is evidence that it is diminished if the aggregate is also scaled. It must be remembered that aggregates are often smaller for the smaller BubbleDeck slabs and thus it is prudent to set the scale factor to the value it would have for a slab of 450mm thickness – in EC2 this amounts to setting k = 1.7.

At edge and corner columns, as well as at eccentric loaded columns and transfer loads, torsion and moment capacity should be checked. Nielsen gives methods for this. The designer should be aware that the resistance of the slab at edge and corner columns may be governed by torsion and flexure as well as punching shear. In fact it is possible that flexural/tensional resistance at edges and corners will make punching shear calculations at these positions irrelevant.

If shear reinforcement is required, a conservative assumption is to design the reinforcement to sustain the entire shear without the concrete contribution. This will assist in avoiding complications with strain softening in intense shear situations at the expense of slightly more shear steel.

The valid detail for the joint at columns or walls is to arrange the filigree to embed into the columns or walls so that shear over the full section can be mobilised. Sometimes there will be requests for a joint around the column where the filigree does not reach the face of the support – usually by a distance of 40mm to 50mm – this is highly undesirable and complicates the shear design and there is no validated method of design. If the detail is unavoidable one may, with care, be able to design the section assuming the filigree to be ineffective near the column or wall but this will produce a greatly reduced shear resistance. At some distance from the column, if the shear reinforcement elements are properly anchored in the filigree and in-situ parts (that is anchored outside the main reinforcement planes), it may be argued that the section can be re-combined and the full section used for these outer perimeters. This leaves some scope for engineering judgment.

Punching shear reinforcement may take the form of purpose made rebar, studs or Lenton Steel Fortress, according to practical constraints. It is important to apply these properly and to pay attention to the anchorage requirements of any system. Shear heads may also be design for extreme situations and may be structural steel or rebar beams. ACI318-05[ix] gives recommendations for the design of these.

Longitudinal shear is only critical at high rates of change of applied moments (which is of course gives the maximum transverse shear). Within the span, the rate of change of moment tends to be less than close to the supports. The areas close to supports are usually solid however, and the filigree is in compression, so the intensity of longitudinal shear near the supports is mitigated. If a check is necessary, EC2 gives values for shear between concrete cast at different times as well as the method for calculating the applied shear (the change in moment divided by the distance between the section considered and the point of zero moment, on average). The girder webs may be taken into account in reinforcing the interface but only one diagonal in every pair unless otherwise can be justified due to the web angle.

Flexure:

A standard method may be used provided that the depth of concrete in compression does not overlap the ball zone by more than 20%. This is almost always the case in all but extremely heavily stressed slabs.

The maximum moments are usually over the columns or supports. This means the compression is in the slab bottom here, and this is usually in a solid zone, so the restriction on the depth of compression need not necessarily apply at columns and supports.

A rectangular stress distribution or other appropriate distribution may be used in the concrete. EC2 contains a useful and simple method but other plastic methods may be used.

Steel should be ductility class B, especially if plastic design is used, unless special calculations prove class A to be satisfactory. This should ensure that the yield strain limit is not reached prematurely in the reinforcement.

The engineer should exercise a degree of judgment when interpreting the results of FE analysis, especially if it is a linear elastic analysis. There are many mathematical anomalies that can occur which can distort the results one way or another. Singularities, for example, can occur at concave corners and point loads and supports – these lead to absurdly high design moments. Some software uses peak smoothing algorithms and, if these are not available, manual averaging or taking moment at the support face may be an expedient choice.

Even with cracked section iterative analysis, high concentrations of moment and/or torsion can occur in corners and, although this is a reflection of reality, they can lead to very high reinforcement requirements. In manual analysis, and indeed in yield line methods, these peaks are averaged out by implied yielding. This is legitimate provided always that the structure, or any substructure, is globally elastic at SLS and provided that the rotations implied by the yielding do not lead to excessive cracking (and consequent increase in deflection), particularly at the top of the slab at columns. Excessive cracking here may also indicate that shear strength is compromised. For these reasons, it is recommended that the top tension steel is bunched toward the centre of such supports – such practice is mentioned in several codes and literature.

Deflection:

Span depth ratio methods are not recommended, except in checking and approximate or relatively unimportant cases. FE analysis is recommended for all slabs as there is no practical manual method that can be used with confidence. Even unidirectional spans can be very tedious in the computation of deflections.

Where accurate deflections are required, the software runs iteratively, calculating modified and cracked element properties at each stage of the iteration, using the applied reinforcement, until convergence is reached. The deflection using this method has been shown to have good agreement with tests conducted at the ECBP at Cardington (see Concrete Society Technical Report TR 58[ii]).

Short term loadings cases are usually patterned, subject to engineering judgment, (chequerboard or parallel strips depending on the characteristics of the project) using the “Frequent Combination” set out in Eurocode 0 and using combination factors “y1” as appropriate. For long term loading, the “Quasi-permanent combination” is used with combination factors “y2” as well as creep coefficient and shrinkage curvature parameters if necessary. Shrinkage curvature is generally of low order compared to extrinsic effects – L/1500 has been quoted as an order of magnitude of the defection component due to this.

The combination factors now available in EC0 represent a statistical method of estimating which part of the imposed load is variable and which is invariable.

For simplicity, and where it can be justified, the engineer may estimate long term loadings using the total permanent load and 50% of the imposed load without great loss of accuracy. This is likely to be good enough for most ordinary building projects.

Non-linear, iterative analysis can take a long time on complex or large slab models so it is not generally efficient to run such an exacting analysis on every slab and every load case. Partial models can be constructed to model limited parts of slabs and reasonably good results can be obtained with the exercise of some prudence. It is recommended to calibrate such partial models by comparing them to the full model under comparable conditions so that the approximation represented by the partial model can be validated. In a similar way, elastic results may be used as a broad approximation provided they use a modified elasticity and that this is calibrated against a more rigorous analysis.

Creep and shrinkage have been shown by tests to be only marginally higher than a solid slab of similar dimension. Due to the precision of serviceability calculations this small difference is usually ignored.

Vibration:

RC slab structures are generally less susceptible to vibration problems compared to steel framed and light weight skeletal structures, especially using thin slabs.

However, BubbleDeck is light and is not immune from vibration in all cases so this must be checked just as it should be in appropriate solid slab applications.

Where deflections are large, as indicated by the static design, it is often an indication that the structure is sensitive to vibration SLS issues.

The lighter weight of BubbleDeck may be exploited if it can usefully alter the modal frequencies of a slab – generally raising them compared to a solid slab. The most effective weapons against vibration, particularly resonant vibration, are stiffness and damping. If we consider damping to be similar to solid slabs, and concentrate on stiffness, we may observe that a BubbleDeck slab can provided over 2_ times the stiffness obtained from a solid slab for the same quantity of concrete used. This can be exploited in vibration sensitive applications.

At the present time, the static modification to the flexural stiffness is applied. However, future work may show that the static stiffness is not the same as the flexural stiffness in BubbleDeck slabs but the difference is thought to be minor compared to the effects of inaccuracies in modelling vibration problems.

TR43[i] should be used for the procedures for determining vibration sensitivity and modal superposition may used to determine the response for given excitation.

Fire resistance:

The fire resistance of the slab is a complex matter but is chiefly dependent on the ability of the steel to retain sufficient strength during a fire when it will be heated and lose significant strength as the temperature rises. The temperature of the steel is controlled by the fire and the insulation of the steel from the fire. The degree to which weakening of the steel is significant is related to the service stress at FLS.

The design then reduces to a determination of the combination of the amount of steel and amount of concrete cover to attain a balance of steel temperature and stress that allows the structure to remain stable at FLS.

Advance or more complex design and analysis may include the determination of temperature profiles in the time domain, of cooling and the even effects of quenching by fire fighting water.

A basic design may make use of the data tabulated in the BD technical manual for cover required for various fire resistance periods and steel stress. An analysis may be carried out for FLS loading (roughly 0.7 of the ULS loading but this should be calculated according to EC2-1-2) and the applied moments obtained. This will allow the designer to check various sections, using calculated moment curvature relationships, to determine the steel stress corresponding to the FLS moments. When these steel stresses are known they may be interpolated in the tabulated data and cover or fire resistance thus estimated.

A question that frequently arises concerns the pressure in the bubbles during heating. Calculations have been carried out by Jørgen Breuning to show that this is not a serious issue. In any case, all concrete is cracked and, in a fire, it is likely that the air would escape and the pressure dissipated.

If the standard bubble material is used (HDPE), the products of combustion are relatively benign, certainly compared to other materials that would also be burning in the vicinity. In an intense, prolonged fire, the ball would melt and eventually char without significant or detectable effect.

Seismic design:

This is a specialist area outside the scope of this brief technical note. However, the concerns in Seismic design are largely similar to any flat slab structure.

Punching shear under seismic conditions is the most critical issue and damage at the slab-column junction during sway reversals should be properly considered as well as amplification of the punching shear due to the vertical component of ground acceleration.

In computing the building's response, the seismic designer should be closely engaged with determination of the mass and the effect of this on modal spectrum. Using BubbleDeck a significant reduction of mass in the floor plate may be realised together with an increase in modal frequency and reduction in the sway forces due to lateral acceleration.

Detailing:

BubbleDeck demands more from the detailer than normal flat slab design – of this there is no doubt. The geometrical discipline required to coordinate the layering and spacing of factory fixed and site fixed rebar as well as the bubble module is far more demanding and requires an attention to detail greater than ordinary detailing.

The BubbleDeck geometry is founded on the module size which, until recently, comprised 200mm, 250mm, 300mm, 350mm, and 400mm. Larger sizes have been added but the rules applying to the geometry still apply.

- The ball diameter is always 0.9 of the module.
- The effective depth, except with heavy reinforcement may be approximated as equal to the module.
- The cover to the bubbles should be at least one ninth of the ball diameter.

The cover to the bubbles and to the reinforcement may vary, of course, and this may require adjustments to be made. Slightly more concrete that standard may be poured to achieve a range of sizes intermediate to those imposed by the fixed module sizes.

The filigree or 'biscuit' standard thickness is 60mm or 70mm depending on size of bottom steel. The bottom edges have a 6mm x 6mm bevel. At least two edges of every unit must have 25mm x 25mm bevel on the top to ensure that the splice bar has a filet of site concrete to seal it against attack by fire.

At a very early stage, the detailer should draft the sections to be used in a project and verify the reinforcement geometry and especially the mesh spacing and girder size required.

Although the edge distance of the balls to the edge of the units, at internal edges that will be concreted, may follow the natural module; the cover to the bubble may be insufficient at the outside of the slab so it may be necessary to leave out a row of balls or otherwise plan the spacing with this in mind. Fixings are frequently made to edges of slabs so a slightly wider solid edge zone is often no bad thing.

Mesh will generally need to be custom mesh and it should be noted that machine made meshes usually have one or more of the following restrictions:

- Max. bar size 16mm
- Longitudinal bar spacing increments of 50mm c/c (eg 50mm / 100mm / 150mm / 200mm, etc.)
- Min. distance from last bar to end 25mm
- Cross wire spacing sometimes in 25mm increments but may be unrestricted according to machine type.

Girders are supplied in height increments of 10mm but some suppliers may supply any size. The diagonals should be 63° approximately and must be welded securely to the longitudinal bars (See CUR86 for a useful specification).

The standard girder spacing, as outlined in CUR86 is two balls maximum. Greater spacing than this is possible but the unit may be too flexible and crack more easily during transit or handling. The longitudinal girder bar should be 10mm minimum for the 200 and 250 modules and at least 12mm for 300 modules and above. The girder web bar may usually be 7mm or 8mm and 8mm is preferred except in light applications.

The section should be drafted so that the correct ball spacing is produced and so that the bubbles are restrained against movement laterally or vertically by at least two bars at the bottom and two bars at the top. It is usually sufficient to have two long bars in the bottom mesh controlling the position and two transverse bars at the top. It is imperative that the ball cannot rise up more than a few millimetres when placed in the casting bed. The top mesh should be low enough in the section to permit the top site steel to be placed allowing for some tolerance.

The detailer should note that the ball will float up, during casting in the factory, until it is in contact with the closest top mesh bars. This means that the top mesh will usually control the height of the ball.

The top mesh does not usually fulfil an important function in the permanent state, except for an crack purposes, and is more significant in the temporary state (lifting and when spanning between props) when it has the important purpose of stabilising the top of the girder against lateral buckling. Clearly it also traps the bubbles in place.

Loose bars, not welded in the mesh, may be detailed to fit between the mesh bars, secured by tying wire, to achieve localised increases in steel area.

Splice bars are placed on top of the filigree and should be detailed so that they have adequate clearance, spacing and anchorage. It will generally be more efficient to provide more of smaller bars than few of larger ones. Anti-progressive-collapse bars will also pass through columns in two directions and lay directly on the filigree.

Top site steel is detailed and placed in the normal way – as for solid slabs. A heavy zone of steel will usually occur over columns with lighter steel on the column lines between these zones. In curtailing the top steel, it is advantageous to do so in a way that does not result in the bars ending very close to a unit joint and thus complicating placement of splice mesh.

Where the top main site steel does not already form a top splice to the top mesh, narrow sheets of top mesh are used to lay over the joints to complete a continuous top reinforcement.

All edges of the slab must be fitted with U-bars, whether they are support edges or not. This provides for the tensional resistance required at slab edges and the satisfaction of the correct conditions for the development of the Kirchhoff boundary forces. This is especially important near supports, like columns, and corners.

Shear reinforcement should be long enough to achieve correct cover top and bottom but must be anchored in the top and bottom steel zones. The lateral spacing should be as close to 0.75 times the effective depth as practicable, but not greater. For radial arrangements of shear reinforcement, the circumferential spacing should be similar in the case of the first element perimeter, which should be placed at a maximum of approximately 0.375 times the effective depth from the face of support. There will almost inevitably be conflict with the mesh and site steel and the spacing should be varied by as small an amount as possible to clear this. In cases where there is doubt about the suitability of a position, and extra element may be placed adjacent.

To close the edge of the mesh and to provide transverse reinforcement to prevent separation of the filigree at the joints, the edges of the units should have Ø8mm hook bars, along the edge, hooked around the bottom mesh and top mesh edge bars.

The mesh should be welded to the top and bottom of the girders and the welds should be sufficiently close together to resist pull-out from the filigree during lifting and should provide sufficiently close spacing to the top girder bar so that it does not buckle when in compression. Triangular or three bar girders have better resistance but are more difficult to install with sufficient space for the bubbles and other steel.

It is suggested that the welds between the mesh and the girders should be at a maximum spacing of 600mm spacing. The welds should not be too far apart as they may allow the girder to pull out to easily from the filigree during lifting. They may also provide insufficient restrain to the girder top bar which must be prevented from buckling, especially when it is spanning across the props on site and supporting the concrete pouring operation.

If fixings are to be made to the top of the slab when it is exposed to the weather, a hole should be drilled right through to enable trapped water to drain out.

References

[i] Technical Report 43, Post Tensioned Concrete Floors – Design Handbook, The Concrete Society.

[ii] Technical Report 58, Deflections in Concrete Beams and Slabs, The Concrete

Society.

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[iv] CIRIA Report 89, Behaviour of Reinforced Concrete Flat Slabs, P E Regan, CIRIA, 1981.

[v] CIRIA Report 110, Design of Reinforced Concrete Flat Slabs to BS8110, CIRIA, Revised Edition 1994.

[vi] Limit Analysis and Concrete Plasticity, M P Nielsen, CRC Press, 2nd Edition 1998.

[vii] Yield Line Formulae for Slabs, K W Johansson, Concrete Society

[viii] Practical Yield Line Design, Gerrard Kennedy and Charles Goodchild, British Cement Association 1st Edition 2003

[ix] ACI318-05, Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)

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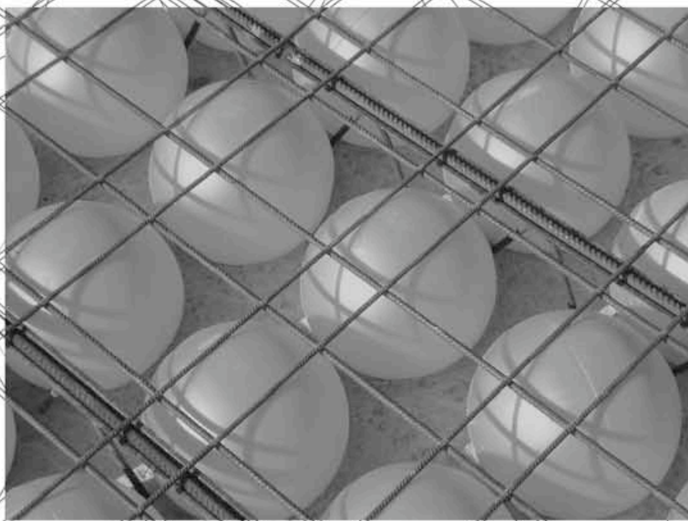
BubbleDeck Acoustic Tests and Reports



March 2006 Issue 2

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Acoustic Tests and Studies

Tests have been carried out in Denmark, the Netherlands, Germany and in UK (Channel Islands).
The conclusions are unambiguous:

BubbleDeck performs acoustically in a better way than any other hollow or solid floor structures.
Because of the three-dimensional structure and the gentle graduated force flow, the hollow spheres have a positive influence on sound insulation.

BubbleDeck behaves like a spatial structure - as the only known hollow concrete floor structure.

The tests reveal that airborne sound insulation is even higher than expected. This indicates the bubbles have a positive influence. Furthermore the combination of in-situ concreting on site with the semi pre-cast elements produce a seamless finished slab without any joints, avoiding joint discontinuity associated with fully pre-cast planks that can seriously impair sound insulation performance.

BUILDING REGULATIONS CRITERIA

Approved Document E, 2003 Edition: Floor Types 1 or 2 requires a concrete slab (cast in-situ or with permanent shuttering) to have a minimum mass of either 365 kg/m2 (Clause 3.29 Floor Type 1.1C), or 300 kg/m2 (Clause 3.67 Floor Types 2.1C and 2.1C(b)).

Our thinnest BubbleDeck 230mm slab with a mass of 368.8 kg/m^2 exceeds Approved Document Part E requirement for Floor Types 1 or 2.

All thicker BubbleDeck slab types have increased mass, exceeding Approved Document Part E requirement for Floor Types 1 or 2, as follows:-

Slab Type	Thickness	Mass kg/m^2
BD280	280mm	460
BD340	340mm	550
BD390	390mm	640
BD450	450mm	730

TESTS AND REPORTS

Report from Adviesbureau Peutz & Associes bv: Comparison of BubbleDeck vs.Hollow core – Enclosure F1.

A comparison was made between BubbleDeck and hollow core deck prior to the construction of Weena Tower. Deck types of similar height were compared.
The noise reduction with BubbleDeck was 1 dB higher than hollow core.

The main criteria for reducing noise is the weight of the deck and therefore BubbleDeck evidently will not act otherwise than other deck types with equal weight.

German Test Certificate Number P-SAC 02/IV-065 concerning solid and live load Sound insulation – Enclosure F2.

The German "Materialforschungs- und Prüfungsanstalt für das Bauwesen Leipzig e.V." has issued the German Test Certificate Number P-SAC 02/IV-065 concerning solid and live load Sound insulation according to DIN EN ISO 140 / DIN ISO 717.

The results for 230 and 340 mm decks are:

	Sound insulation dimension	Additional spectrum adaptation values (DIN ISO 717-1)						Standard impact sound level
Deck	R_w (C;Ctr)	$C_{50-3150}$	$C_{tr50-3150}$	$C_{50-5000}$	$C_{tr50-5000}$	$C_{100-5000}$	$C_{tr100-5000}$	$L_{c,w\ (C1;C50-3150)}$
mm	dB	dB	dB	dB	dB	dB	dB	dB
230	55 (-2 ; -7)	-2	-8	-1	-10	-1	-8	78 (-11 ; -12)
340	57 (-2 ; -7)	-3	-9	-2	-7	-2	-9	76 (-13 ; -13)

Test Report from Adviesbureau Peutz & Associes b.v.: Sound Resistance. March 2004 - Enclosure F3.

Field tests in a raw building in Leiden, the Netherlands, concerning “Air and Contact Noise-resistance”. The slabs were BD 230 mm with a fixed floor layer of 30 mm.
The measurements and ratings were carried out in regulation with ISO 717-1:1996 and NEN 5077:2001

Weighted Sound Reduction (vertical)	R'_w (C;Ctr)	=	54 (-1; -14)
Reduction Index for Airborne Sound	I_{lu}	=	+ 3
Impact Resistance Level (vertical)	$L'_{n,Tw}(C_1)$	=	72 (-14)
Reduction Index for Impact Sound	I_{co}	=	+ 2
Impact Resistance Level (horizontal)	$L'_{n,Tw}(C_1)$	=	63 (-13)
Reduction Index for Impact Sound	I_{co}	=	+ 10

Test Report from Ian Sharland Ltd : Airborne and Impact Sound Insulation. Nov 2005 - Enclosure F4.

Field tests in Le Coie Housing Development in St. Helier, Jersey, concerning “Airborne and Impact Sound Insulation”. The slabs were BD 285 mm, part of a standard party floor with ceiling and screed. The measurements and ratings were carried out in regulation with ISO 140-4:1998, ISO 140-7:1998, ISO 717-1:1997 and ISO 717-2:1997.

The Vertical Impact sound reduction (mean) was measured to: L_{nTw} = 44 dB
The Vertical Airborne sound reduction (mean) was measured to: D_{nTw} = 61 dB

The results show that the floor structures tested meet and significantly exceed the requirements of the British Building Regulations (2000)

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BubbleDeck Voided Flat Slab Solutions

Technical Paper

Anti-Disproportionate Collapse Properties of BubbleDeck

January 2006

This BubbleDeck® Technical Paper has been written to provide more detail about the inherent properties of BubbleDeck slabs in resisting Disproportionate Collapse.

Buildings must always be designed to be stable, robust and resistant to a reasonable degree of abuse, whether during construction or in use, regardless of their size. This is explicit in all of the design codes and Building Regulations. For buildings of medium to large size, there are special provisions to limit collapse, in accidental situations, to a limited area of the structure so that the damage is contained and the occupants or those around the building may survive and so that rescue and fire-fighters have a reasonable chance of entering the building after a damaging event. This is known as the avoidance of disproportionate or progressive collapse and is a requirement in the Building Regulations intended to limit collapse so that it is not disproportionate to the cause.

The history of this provision is vague but, for those in the UK it, is punctuated by an important accident that occurred in the 1960s – the Ronan Point collapse – which will be remembered by all those of around 50 or more. In this incident a simple domestic gas explosion provoked the collapse of an entire corner of a block of flats. Gas explosion is probably one of the causes uppermost in the mind of the designer of buildings but there are also many other events that can damage a building seriously enough for it to collapse if it is not robust enough – fire, terrorism and vehicular collision being also of concern. The result of the bombing of the Murrah building in Oklahoma in 1995 is an, admittedly extreme, example of progressive collapse.

Generally speaking designers of modern buildings seeks to simplify the system of structural support in the interests of economy and to enable rational design methods to be used. This often results in buildings with relatively few load paths from any part to the ground. In more complex buildings, such as older buildings that may be more highly compartmentalized with partitions and walls, there may be many unintended load paths from any one part of the structure to the foundations – that is to say; parts of the building not intended to bear load will take a share of load if some other, perhaps primary, part of the structure is removed or seriously damaged.

Typically, the modern building will comprise a frame of steel or concrete beams and columns or may be formed by flat slab construction on columns. Sometimes it may have load-bearing masonry instead of columns. Here there would be relatively few primary structural components. For example, a column may be removed by an accidental event and that column may have supported an entire bay for the full height of the building – if the structure above the column were not capable of spanning to the adjacent intact columns, a large volume collapse would occur.

To avoid the catastrophic effects of removal or damage to primary structural elements, the building may be designed to either span over such localized damage or it may be designed with alternative load paths to the ground. The latter solution militates against economy and use of space inside the building so it is more common to adopt the former approach. Another approach is to provide what are termed 'protected key elements', which are supports designed to resist all reasonably foreseeable accidental damaging events.

Spanning over damaged areas can be accomplished in a number of ways:

- Firstly, we may assume that the structural elements, such as beams and slabs, remain isolated and not integrated or interconnected. The means that the framework of the building, i.e. columns, walls, beams or parts of slab, must remain largely intact and bridge the damaged area by forming a temporary structure akin to a truss, Veerendeel frame or some kind of space frame. This will require that the primary structural elements must remain attached together and, to this end, the codes require that effective tying is provided around each floor of a building, across each floor, and up the height of the building. The ties so formed, usually in the form of steel bars, straps, or structural steel sections, form a framework that can support the superstructure over a damaged area, even if the deflection in the accidental event would be alarming in normal use.

Pre-cast floors have an inherent disadvantage in respect of these provisions as they are isolated elements that require tying together in various directions on site after erection. Some part filigree/in-situ systems, which are fundamentally one way spanning, have to be provided with tying across the main span to achieve the requisite tying in each direction and this also is added on site.

An extract from 'Stability of Modern Buildings'[i] is given below:

> 2.3 Effective ties
>
> The general principles of conserving stability against unforeseen hazards have been set out in Section 1 when discussing the concept of providing an alternative path for carrying the load to the ground.
>
> The basic requirement of this concept is to contain the local damage that develops in one section of the structure within a condition of partial instability and ensure that this does not lead to the total collapse or full instability of the building. This implies a capability in the structure of bridging or stringing over the damaged area of the work.
>
> To satisfy the basic requirement, the structure should be effectively tied together at each floor level on the longitudinal and transverse axes along the main column grid lines, combined with effective continuity of the vertical load-carrying members. The main constituent elements of the floors and walls should also be anchored into the main-tie system where they are not themselves used to form the main tie.
>
> Emphasis has been placed on the need for an effective tie. The magnitude is obviously related to the span of the floor and beam components and should be capable of sustaining practical catenary effects. It is also essential that the tie should be of a continuous form. Unless the joints in the tie are firmly bonded into the mass of the connected components they should be of an interlocking type. In this respect, tests on continuous floor slabs over two bays when loaded without the central support have shown a considerable increase in loading capacity in this emergency condition when the bottom reinforcement is fully lapped over the missing support.
>
> Storey-height posts in the outside walls and particularly at the corners are always relatively vulnerable, and where these occur the bridging capacity of the supported beam and members should be correspondingly improved. Reference has also been made to the potential capacity of storey-height walls to bridge across the damaged sections of their support when properly tied in their own plane at the floor level (see Section 1).

- A second method of providing resistance to disproportionate collapse is to tie spans together so that a catenary is formed by the steel if an intermediate support is removed. The sag of the slab or beam in such an arrangement may be dramatic but the intention is that it will just remain intact. This form of catenary action is difficult to realize in pre-cast construction as the necessary embedment and joining of the steel in all directions is often complex especially where the elements must be anchored to vertical ties.

  Unfortunately the catenary effect does not really work at building corners and vertical support to exterior corners are often vulnerable in many buildings due to explosion, or vehicle collision combined with the fact that windows often leave only an isolated pier or column at the corner.

- A third method is to provide a reserve of conventional spanning capacity of the horizontal elements such as floor slabs or beams. A slab may be designed to perform satisfactorily in the conventional way but also to have reserve strength to span across damaged or removed supports even if the deflection or sag is large - this is an intermediate stage toward the catenary effect mentioned above but the slab remains intact and deforms much less.

- The provision of alternative load paths was mentioned above. Providing extra beams and columns has obvious disadvantages. However, in-situ slab structures are inherently robust in this respect since reinforcement is usually provided in at least two directions and, even if the slab is primarily intended to span one way only, it has the ability to span the other way and across a wide band so it may, with little or no need for modification, span onto other vertical elements if the primary vertical support is removed.

  Consider for example a corner room of a building supported on four sides: If the corner column or a wall were removed so that the end support to a span was removed, such as at Ronan Point, an in-situ slab could span across the remaining walls. The span may form between remaining opposite walls or across adjacent walls meeting at a corner. Not only can the span remain intact but it can, since it is usually tied into the rest of the floor across the whole width in both directions, provide lateral restraint to the remaining walls or columns to prevent buckling or bursting outward. Even non- structural infill wall may form a temporary support when primary walls are removed although the designer would have to bear this in mind when specifying the infill materials and form of construction.

- Protected key elements are a viable option in most buildings but usually employed when 'all else fails'. They are required to resist very intense loadings applied directly to them and through anything attached to them so they can become inevitably complex or large and costly. The load to be applied to them as prescribed in current codes is largely derived from tests on contained gas-air explosions. Designing for vehicle impact can be complex and onerous exercise if economic sizes are to result.

BubbleDeck is fundamentally an in-situ RC slab and its benefits in respect of disproportionate collapse are realised in all of the four mechanisms described above. No other floor system except solid in-situ floor construction has this as an inherent feature. The filigree elements are relatively wide and are joined together with splice bars that are designed to develop the full moment of resistance required at any joint and, in this way, are functionally equivalent to an ordinary in-situ two way spanning slab.

One way spanning systems, with or without filigree soffits must be made to fulfil the secondary spanning function by adding components such as tie bars. Such secondary reinforcement may be intended to provide catenary action described above or may be intended to provide true spanning action in the secondary direction. However, despite the superficial similarity to the BubbleDeck jointing, since the added bars are placed on top of the relatively narrow filigree units, the spanning capacity is limited by the reduced effective depth to the reinforcement across the whole slab width.

The BubbleDeck slab is not greatly affected by the reduced effective depth of the splice bars and deflection performance is only marginal affected, since these bars are at infrequent intervals and the increased curvature due to reduce effective depth takes place of a very short distance local to the joint. The curvature contributing to flexural deflection therefore would comprise a very small proportion of span length corresponding to the joints and a very large proportion of the span remote from the joints with full effective depth.

[For the technical reader, the deflection can be expressed as a function of the double integral of the curvature on the member length – a method of calculating deflections is to integrate the curvature twice upon the span length and add this to the support tangent. The reader may verify that that the length over which the curvature is effective is fundamental to the result so resulting deflection will be dominated by curvature over the majority of the span length rather than proportionally very small lengths of high curvature. For a more detailed exposition of the theory, see the classic text in Timoshenko[ii] or other of the various treatments of the subject.]

The splice bars in a BubbleDeck slab are usually given sufficient lap length, often the full tension lap, to ensure that full tying and spanning is obtained. Since the slab is reinforced in both directions across its whole width it can span over damaged areas in one direction or another and can, in major damage, form part of a catenary system.

The BubbleDeck slab can be locally reinforced with higher concentrations of rebar to enable walls to be supported on it and this can also be enhanced by leaving out rows of balls at strategic positions. Since effective tying is continuous, walls and slab may be designed compositely to form deep beams or employ tied arching to span across larger opening formed by removal of support below.

BubbleDeck slabs can be designed in the conventional way to resist sagging or hogging moments with equal facility. In terms of disproportionate collapse this can be utilised to assist in the support of support corners by cantilevering diagonally as well as orthogonally.

Eur Ing R A Beeton BSc CEng MICE

BubbleDeck UK Senior Engineer, January 2006

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[i] Stability of Modern Buildings, Institution of Structural Engineers, September 1971.
[ii] Strength of Materials, Timoshenko, Van Nostrand Co Inc, 1956.

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BubbleDeck Voided Flat Slab Solutions

Fixings & Holes Manual

Edition 1. Revision A: June 06

Introduction

- The information in this document is given in good faith and is believed to be correct at time of publication.
- This document has been published by BubbleDeck C.I. Ltd. For further details, please contact us.
- When using Hilti equipment and products it is important that standard safety procedures are observed and regulations adhered to. For further information please refer to Hilti technical and safety documents

Fixings into BubbleDeck slabs

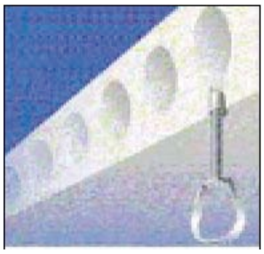
Lightweight Fixings

There is a minimum of 20mm concrete below the centre of each bubble, but just a short distance away from the bubble centre the concrete depth quickly increases to 70mm plus up the side of each bubble. Therefore fixings for attaching lightweight articles can be made using normal methods (plug & screw / expanding anchors, etc.) to provide adequate fixings for wiring conduits, small cable trays, small ventilation ducts and the like.

Medium & Heavy Weight Fixings

Where stronger fixings are required to resist higher pull out (downward) forces from medium / heavy loads to be suspended from the soffite we recommend our Bubble layout drawings are inspected to determine where fixings will occur directly below or close to the edge of a bubble. Where fixing locations and lengths are likely to project into a bubble void we recommend Hilti HIT HY20 Injection Resin Anchor with HIT sieve, item no. 00068613, are used. Hilti also produce a range of other fixing systems designed for fixing through into voids.

Hilti Fixings into BubbleDeck Voids

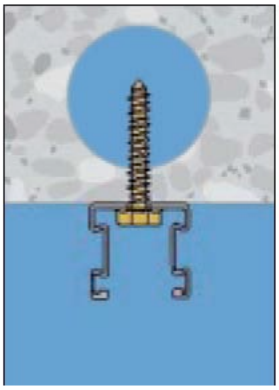



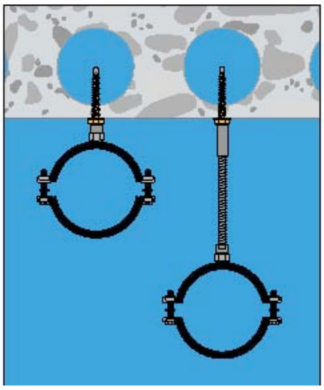
HKD-S Anchor

This is a shallow embedment depth anchor with an internal thread suitable in thread forms M6, M8 and M10. The fixing is flush with the surface after setting. The anchor is placed in a drilled hole in the precast unit and set by driving a central expansion cone into the the anchor with a purpose designed setting punch. The setting tool leaves a visible impression on the end of the anchor to enable easy inspection to see that the fixing has been correctly installed.

This fixing is commonly used with set screws or threaded rod for plumbing, heating, air conditioning and ventilation installations, pipe suspension, air ducts, etc, and also for securing channels, rails, plates, brackets and suspended ceilings. Variations in thickness fastened can be accommodated using different length set screws or threaded rods. It can be loaded as soon as it has been installed. A fire rated fixing can be achieved by using HKD-S anchors.

HUS Screw Anchor



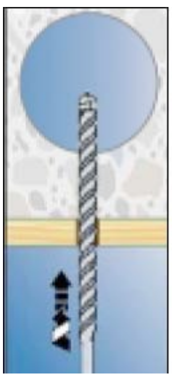


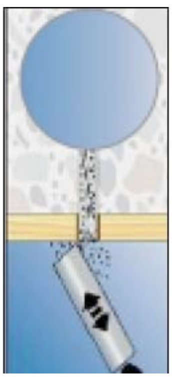
This screw fastening is made straight into the concrete without using a separate plug. A 6mm diameter hole is drilled into the concrete slab and the HUS screw is driven straight into the hole using an appropriate tangential impact screwgun.
This fixing is available with the following head types:

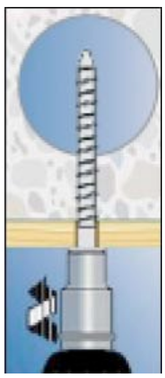
- Flat head for fixing timber and soft materials.
- Hexagon head used for fixing for instance light duty installation channels, light duty steel angles, mechanical installation components and parts for interior panelling or cladding.
- Metric connection thread for fastening light duty pipe rings. This fixing can be loaded as soon as it has been installed. The HUS Screw fixing comes in a range of lengths to allow for variations in thickness fastened. A fire rated fixing can be achieved by using HUS Screw fixings.

HUS Universal		**HUS-H Hexagon head**		**HUS-A Metric connection thread**	
Item no.	Description	Item no	Description	Item no	Description
00071260	HUS 7.5x35	00309340	HUS-H 7.5x35	00309346	HUS-A 7.5x45 M8
00071261	HUS 7.5x45	00309341	HUS-H 7.5x45	00309347	HUS-A 7.5x65 M8
00071278	HUS 7.5x60	00309342	HUS-H 7.5x60	00309348	HUS-A 7.5x45 M10
00071279	HUS 7.5x80	00309343	HUS-H 7.5x80	00309349	HUS-A 7.5x65 M10
00071280	HUS 7.5x100	00309344	HUS-H 7.5x100		
00071281	HUS 7.5x120	00309345	HUS-H 7.5x120		
00071282	HUS 7.5x140				
00071283	HUS 7.5x160				
00071284	HUS 7.5x180				
00309350	HUS 7.5x200				
00309351	HUS 7.5x220				

For all sizes, the hole diameter is 6mm.







HIT HY 20 Injection Resin Anchor

This is a chemical anchoring system for hollow units which produces a keyed fixing. This fixing is particularly suitable for use with the thinner BubbleDeck slab types, BD230 & BD280.

It consists of a specially formulated chemical mortar, a sieve and a fastening insert, which can be either an anchor rod or a threaded socket.

A hole is drilled into the unit penetrating the hollow core. The sieve is placed in the drilled hole and filled with the mortar. The fastening element is then pushed into the mortar, which displaces some through the sieve to form a mechanical key into the hollow section. The resin mortar must be allowed to cure before the fixing is loaded.

A fire rated fixing can be achieved by using HIT HY 20 fixings.

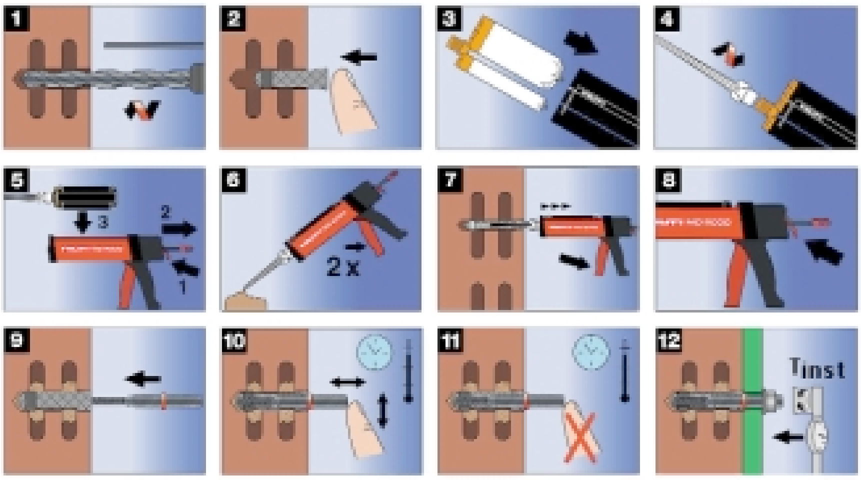
System Components
Hilti HIT HY 20 Injection Technique

HIT HY 20 resin
Item no. 00229156

HIT sieve		HIT-A anchor rod		HIT-IG threaded socket	
Item no.	Description	Item no.	Description	Item no.	Description
00068613	HIT-S 16	00049767	HIT-A M6/65	00077485	HIT-IG M8
00068615	HIT-S 22	00334767	HIT-AN 8/100	00077486	HIT-IG M10
		00334768	HIT-AN 10/110	00077487	HIT-IG M12
		00334769	HIT-AN 12/115		

Please note that threaded sockets with HIT HY 20 have not been tested for this application.

Setting Details (e.g. in hollow brick)



Holes through slabs

Holes can easily be diamond core drilled through the completed BubbleDeck slab. Due to the two way spanning attributes of BubbleDeck slabs there are few limitations on the positioning of holes, except near columns where loads are transferred from the slab into the columns and shear forces are highest.

For holes larger than 250mm diameter, or within 500mm of a column, or multiple holes in close proximity please refer to our Technical Department for advice before forming.

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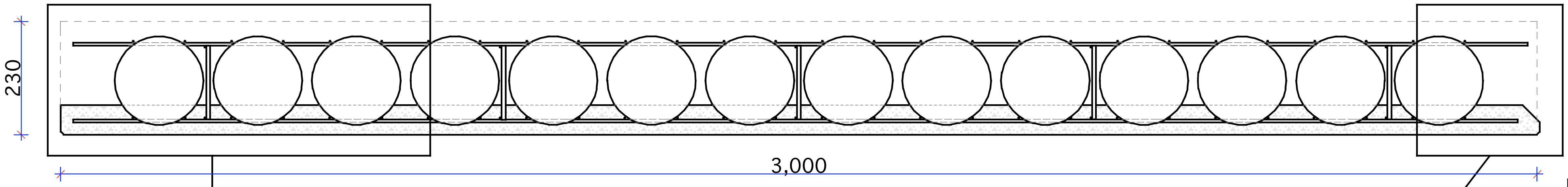
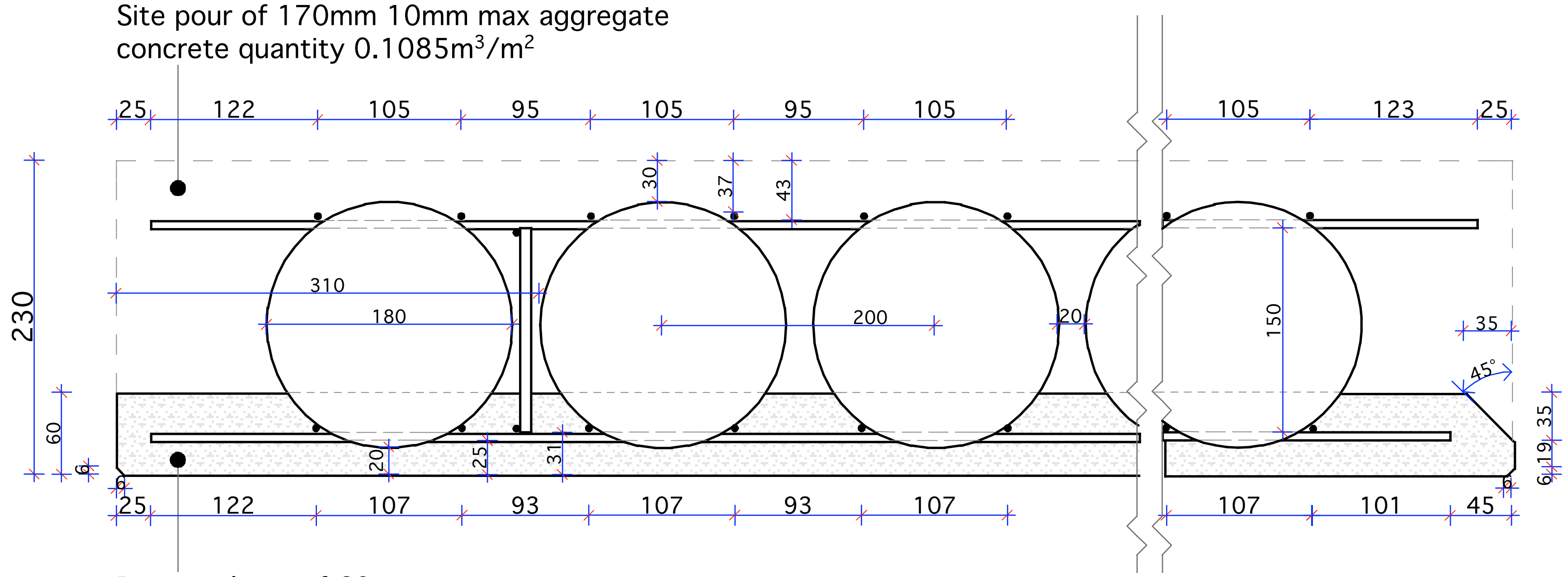
BubbleDeck Fixings & Holes Manual
Edition 1A – June 2006

REVISIONS



1:10 Element Section
BD 230A 3 metre Wide Element

Site pour of 170mm 10mm max aggregate concrete quantity 0.1085m³/m²



Pre-cast layer of 60mm concrete permanent shuttering forming part of overall slab depth

1:5 Detail Section
BD 230A End Section

BD230 SPECIFICATIONS
Precast Element Weight: 180Kg/m2
Completed Slab Weight: 370Kg/m2
Finished Slab Thickness: 230mm

COMPONENT	WELDS/M2
Bottom Reinforcement Mesh	93.3
Top Reinforcement Mesh	93.3
Girders/Mesh	33.3
TOTAL WELDS PER M2:	219.9

WELDS PER 1 LINEAR METRE-3m	ELEMENT
Bottom Reinforcement Mesh	280
Top reinforcement Mesh	280
Girders/Mesh	100
TOTAL WELDS 1 x 3 Metre:	660

BALLS	
Diameter	180mm
Max No. Per m2	25

CONCRETE	
Precast Concrete Thickness	60mm
Site Concrete o/all thickness	170mm
Site Concrete Quantity(m3/m2)	0.11
(Including allowance for areas of balls left out and grout loss)	

ALTERNATIVE MESH REINFORCEMENT CONFIGURATIONS											
	Mesh	Cover Top Main	Cover Top Secondary	Cover Bottom Main	Cover Bottom Secondary	Cover Top Bubble	Cover Bottom Bubble	Spacing Top Main	Spacing Top Secondary	Spacing Bottom Main	Spacing Bottom Secondary
As drawing	6	37	43	31	25	30	20	105/95	87/113	107/93	112/88
Optional	8	31	39	33	25	30	20	80/120	92/108	82/118	95/105
Optional	10	25	35	35	25	30	20	61/139	98/102	72/128	98/102

FIRE RESISTANCE To BS 8110-1:1997		
	Bottom Concrete Cover	FR Period
As drawing	20mm	1 Hour
Optional	25mm	1.5 Hours
Optional	30mm	2 Hours

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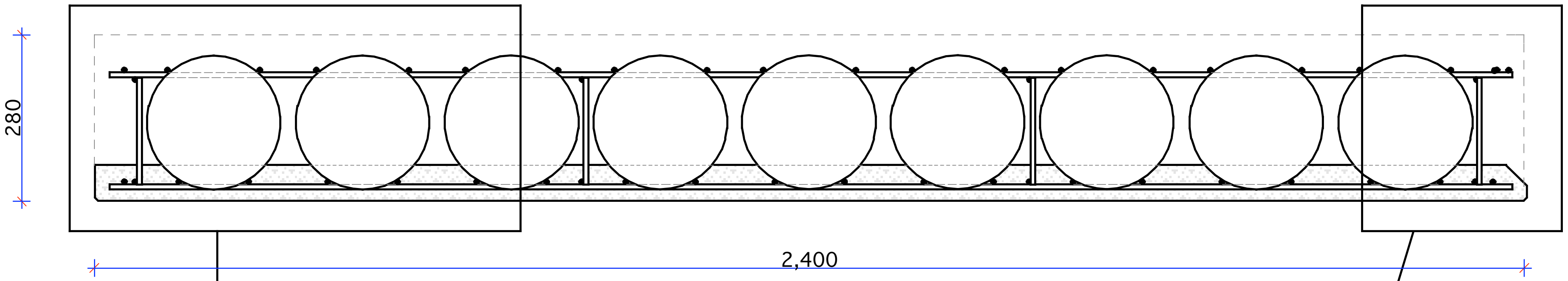
PROJECT / LOCATION	DRAWING	
BubbleDeck Standard Element Detail	BD 230 Element Sections Type A - Filigree Element 3m wide option	
CLIENT	DRAWN	CHECKED
	MT	
DATE	SCALE	DRAWING NUMBER
February 2006	1:5/1:10	BDS001/11

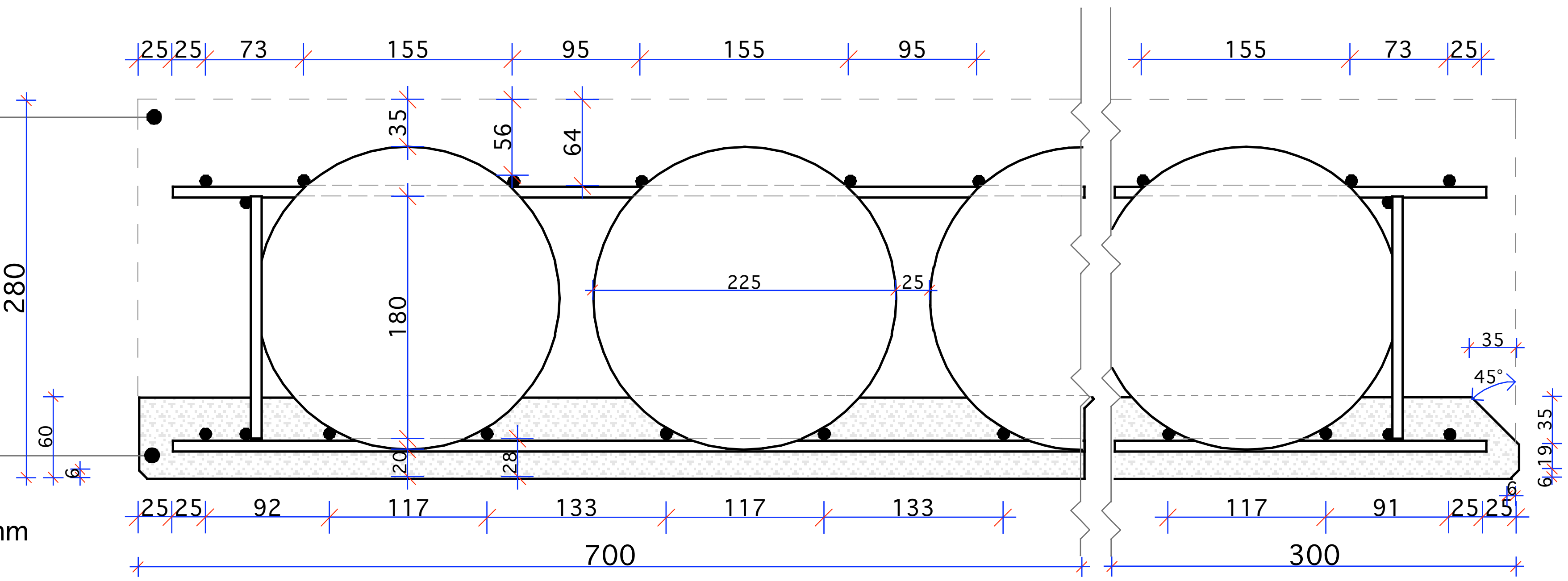
REVISIONS



1:10 Element Section

BD 280A 2.4 metre Wide Element

Site pour of 220mm
10mm max aggregate
concrete quantity
0.1392m³/m²



Pre-cast layer of 60mm
concrete permanent
shuttering forming part
of overall slab depth

1:5 Detail Section

BD 280A End Section

BD280 SPECIFICATIONS
Precast Element Weight: 180Kg/m2
Completed Slab Weight: 460Kg/m2
Finished Slab Thickness: 280mm

COMPONENT	WELDS/M2
Bottom Reinforcement Mesh	66.7
Top Reinforcement Mesh	66.7
Girders/Mesh	26.7
TOTAL WELDS PER M2:	160.1

WELDS PER 1 LINEAR METRE-2.4m	ELEMENT
Bottom Reinforcement Mesh	160
Top reinforcement Mesh	160
Girders/Mesh	64
TOTAL WELDS 1 x 2.4 Metre:	384

BALLS	
Diameter	225mm
Max No. Per m2	16

CONCRETE	
Precast Concrete Thickness	60mm
Site Concrete o/all thickness	220mm
Site Concrete Quantity(m3/m2)	0.14
(Including allowance for areas of balls left out and grout loss)	

ALTERNATIVE MESH REINFORCEMENT CONFIGURATIONS

	Mesh	Cover Top Main	Cover Top Secondary	Cover Bottom Main	Cover Bottom Secondary	Cover Top Bubble	Cover Bottom Bubble	Spacing Top Main	Spacing Top Secondary	Spacing Bottom Main	Spacing Bottom Secondary
As drawing	8	56	64	28	20	35	20	155/95	171/79	117/133	83/167
Optional	10	50	60	30	20	35	20	145/105	167/83	131/119	95/155

FIRE RESISTANCE To BS 8110-1:1997

	Bottom Concrete Cover	FR Period
As drawing	20mm	1 Hour
Optional	25mm	1.5 Hours
Optional	30mm	2 Hours

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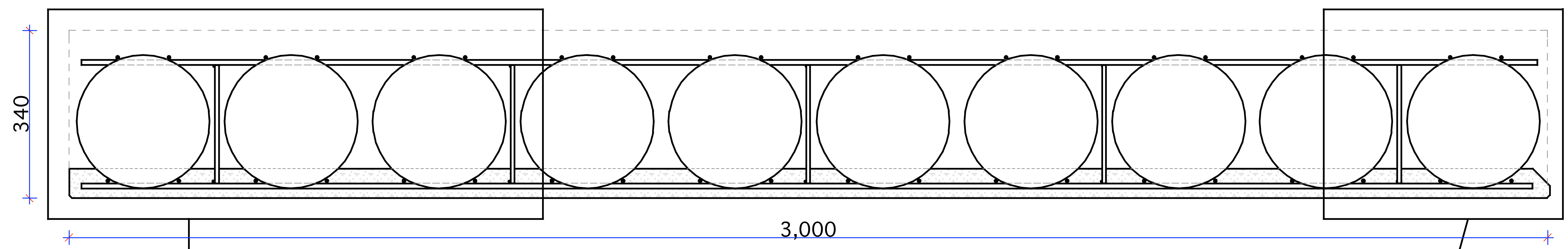
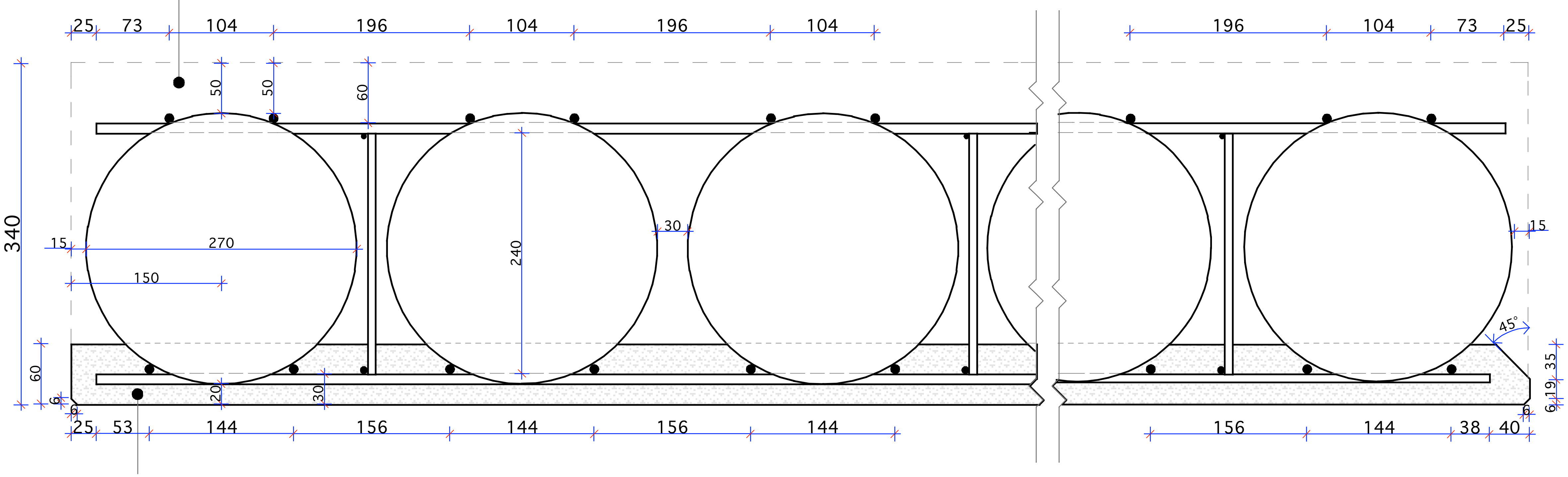
PROJECT / LOCATION	DRAWING	
BubbleDeck Standard Element Detail	BD 280 Element Sections Type A - Filigree Element 2.4m wide option	
CLIENT	DRAWN	CHECKED
	MT	
DATE	SCALE	DRAWING NUMBER
February 2006	1:5/1:10	BDS001/2



1:10 Element Section
BD 340A 3 metre Wide Element

Site pour of 280mm 10mm max aggregate concrete quantity 0.1809m³/m²



Pre-cast layer of 60mm concrete permanent shuttering forming part of overall slab depth

1:5 Detail Section
BD 340A End Section

ALTERNATIVE MESH REINFORCEMENT CONFIGURATIONS	Mesh	Cover Top Main	Cover Top Secondary	Cover Bottom Main	Cover Bottom Secondary	Cover Top Bubble	Cover Bottom Bubble	Spacing Top Main	Spacing Top Secondary	Spacing Bottom Main	Spacing Bottom Secondary
Optional	8	56	64	28	20	50	20	120/180	149/151	131/169	95/205
As drawing	10	50	60	30	20	50	20	104/196	144/156	144/156	104/196
Optional	12	44	56	32	20	50	20	81/219	138/162	157/143	114/186

FIRE RESISTANCE To BS 8110-1:1997	Bottom Concrete Cover	FR Period
As drawing	20mm	1 Hour
Optional	25mm	1.5 Hours
Optional	30mm	2 Hours

This drawing must be read in conjunction with all other drawings, details and specifications issued by the Architect, Structural Engineer and other Consultants or approved specialists. Discrepancies between any other drawings, details and/ or specifications must be referred to BubbleDeck C.I. for verification at least 7 days prior to commencement of the work. It is the Main Contractor's responsibility to ensure all work is carried out in accordance with all statutory requirements and to the approval of the Building Control Officer.

The Main Contractor is reponsible for all setting out of the works. Use written dimensions only, do not scale off drawings. All dimensions must be checked on site.

All work to be undertaken in accordance with all other BubbleDeck's documents & recommendations issued to the Main Contractor
If in any doubt refer to BubbleDeck C.I. prior to commencement of the work.

REVISIONS

BD340 SPECIFICATIONS
Precast Element Weight: 180Kg/m2
Completed Slab Weight: 550Kg/m2
Finished Slab Thickness: 340mm

COMPONENT	WELDS/M2
Bottom Reinforcement Mesh	46.7
Top Reinforcement Mesh	46.7
Girders/Mesh	23.3
TOTAL WELDS PER M2:	116.7

WELDS PER 1 LINEAR METRE-3m	ELEMENT
Bottom Reinforcement Mesh	140
Top reinforcement Mesh	140
Girders/Mesh	70
TOTAL WELDS 1 x 3 Metre:	350

BALLS	
Diameter	270mm
Max No. Per m2	11.1

CONCRETE	
Precast Concrete Thickness	60mm
Site Concrete o/all thickness	280mm
Site Concrete Quantity(m3/m2)	0.18
(Including allowance for areas of balls left out and grout loss)	

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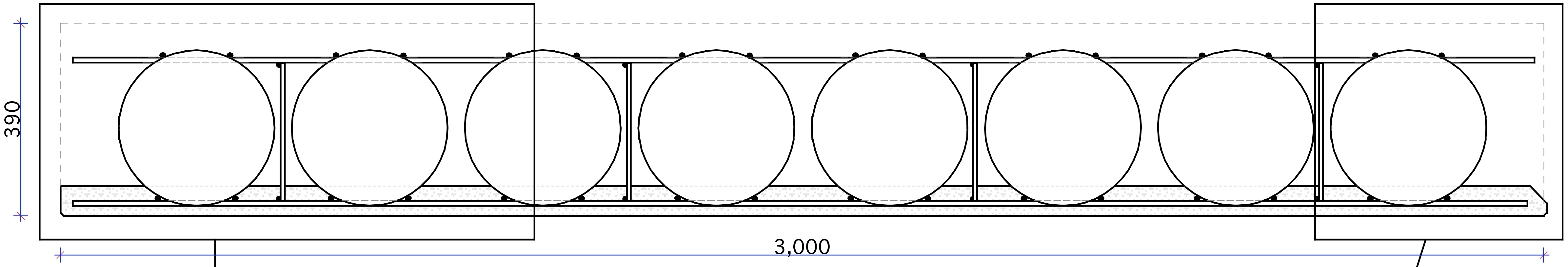
PROJECT / LOCATION	DRAWING
BubbleDeck Standard Element Detail	BD 340 Element Sections Type A - Filigree Element 3m wide option

CLIENT	DRAWN	CHECKED
	MT	

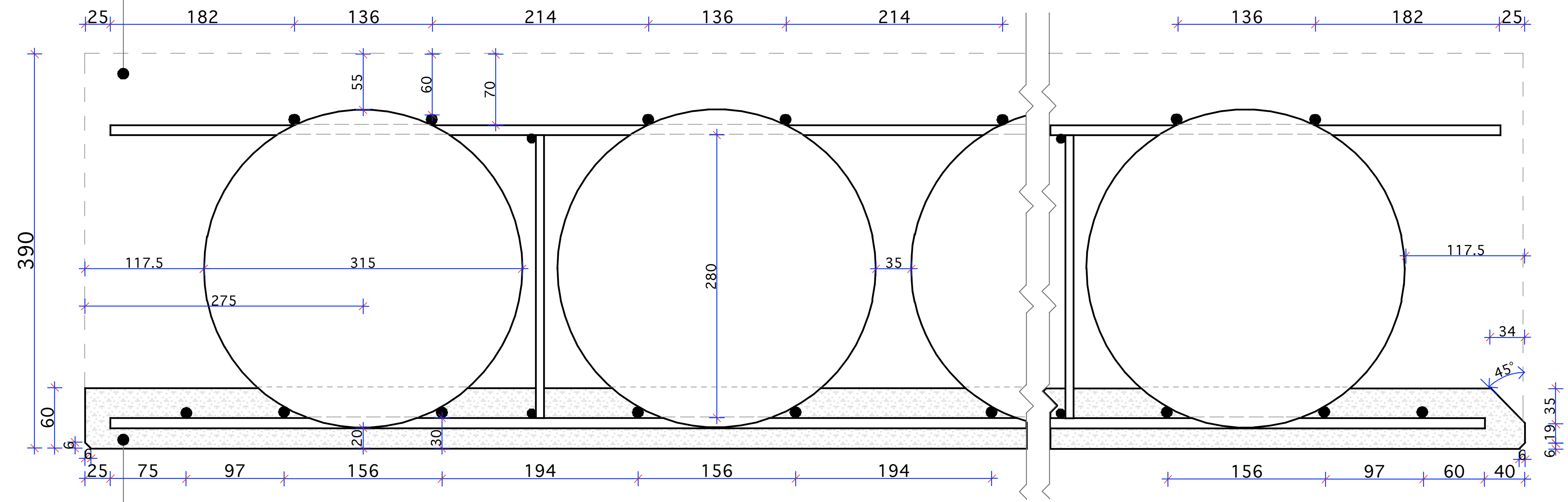
DATE	SCALE	DRAWING NUMBER
February 2006	1:5/1:10	BDS001/13

REVISIONS



1:10 Element Section
BD 390A 3 metre Wide Element

Site pour of 330mm 10mm max aggregate concrete quantity 0.2125m^3/m^2



Pre-cast layer of 60mm concrete permanent shuttering forming part of overall slab depth

1:5 Detail Section
BD 390A End Section

BD390 SPECIFICATIONS
Precast Element Weight: 180Kg/m2
Completed Slab Weight: 640Kg/m2
Finished Slab Thickness: 390mm

COMPONENT	WELDS/M2
Bottom Reinforcement Mesh	36
Top Reinforcement Mesh	32
Girders/Mesh	16
TOTAL WELDS PER M2:	84

WELDS PER 1 LINEAR METRE-3m	ELEMENT
Bottom Reinforcement Mesh	108
Top reinforcement Mesh	96
Girders/Mesh	48
TOTAL WELDS 1 x 3 Metre:	252

BALLS	
Diameter	315mm
Max No. Per m2	8.16

CONCRETE	
Precast Concrete Thickness	60mm
Site Concrete o/all thickness	330mm
Site Concrete Quantity(m3/m2)	0.21
(Including allowance for areas of balls left out and grout loss)	

ALTERNATIVE MESH REINFORCEMENT CONFIGURATIONS											
	Mesh	Cover Top Main	Cover Top Secondary	Cover Bottom Main	Cover Bottom Secondary	Cover Top Bubble	Cover Bottom Bubble	Spacing Top Main	Spacing Top Secondary	Spacing Bottom Main	Spacing Bottom Secondary
Optional	8	66	74	28	20	55	20	152/198	179/171	140/210	100/250
As drawing	10	60	70	30	20	55	20	136/214	173/177	156/194	112/238
Optional	12	54	66	32	20	55	20	118/232	167/183	171/179	123/227

FIRE RESISTANCE To BS 8110-1:1997		
	Bottom Concrete Cover	FR Period
As drawing	20mm	1 Hour
Optional	25mm	1.5 Hours
Optional	30mm	2 Hours

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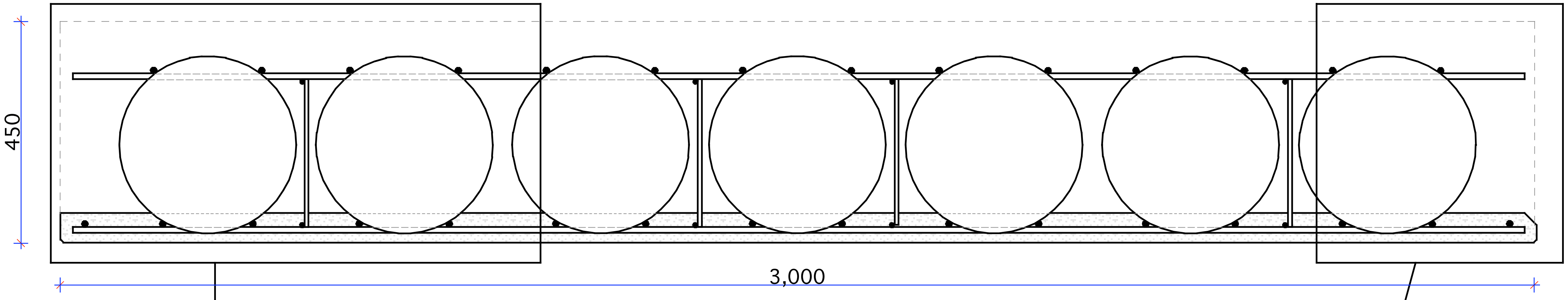
PROJECT / LOCATION	DRAWING
BubbleDeck Standard Element Detail	BD 390 Element Sections Type A - Filigree Element 3m wide option

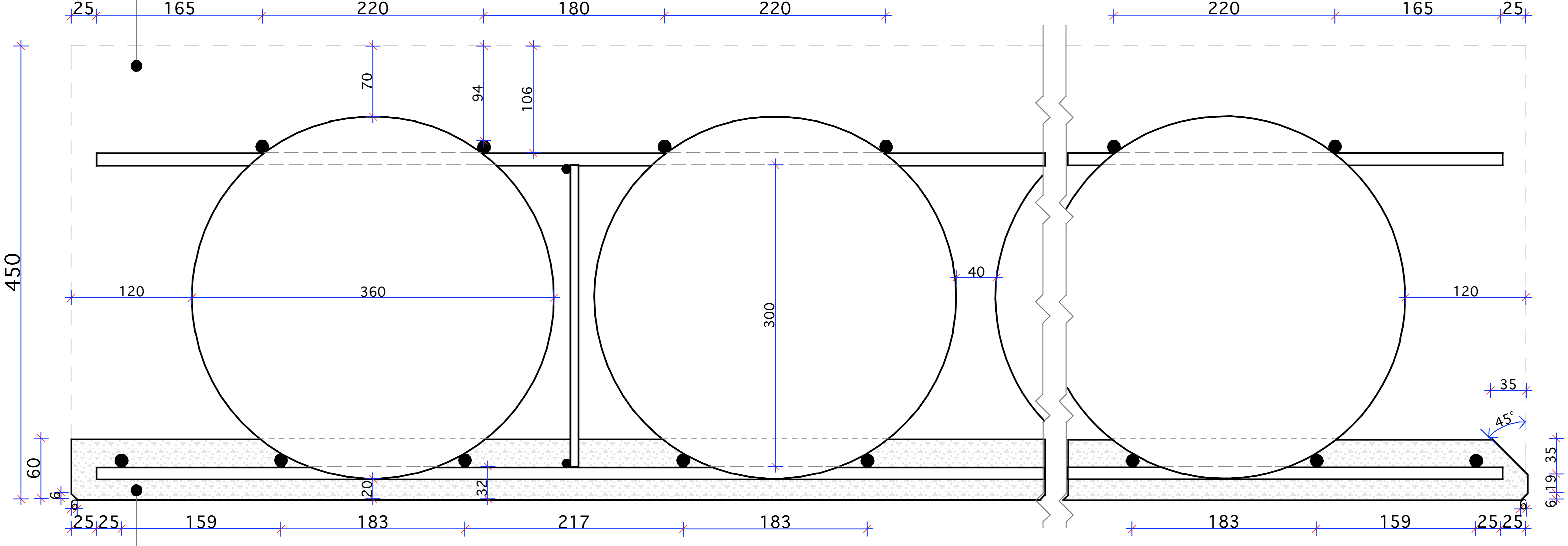
CLIENT	DRAWN	CHECKED
	MT	

DATE	SCALE	DRAWING NUMBER
February 2006	1:5/1:10	BDS001/14



1:10 Element Section
BD 450A 3 metre Wide Element

Site pour of 390mm 10mm max aggregate concrete quantity 0.2547m^3/m^2



Pre-cast layer of 60mm concrete permanent shuttering forming part of overall slab depth

1:5 Detail Section
BD 450A End Section

ALTERNATIVE MESH REINFORCEMENT CONFIGURATIONS											
	Mesh	Cover Top Main	Cover Top Secondary	Cover Bottom Main	Cover Bottom Secondary	Cover Top Bubble	Cover Bottom Bubble	Spacing Top Main	Spacing Top Secondary	Spacing Bottom Main	Spacing Bottom Secondary
Optional	8	106	114	28	20	70	20	239/161	256/144	150/250	107/293
Optional	10	100	110	30	20	70	20	230/170	253/147	167/233	120/280
As drawing	12	94	106	32	20	70	20	220/180	249/151	183/217	131/269

FIRE RESISTANCE To BS 8110-1:1997		
	Bottom Concrete Cover	FR Period
As drawing	20mm	1 Hour
Optional	25mm	1.5 Hours
Optional	30mm	2 Hours

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REVISIONS

BD450 SPECIFICATIONS
Precast Element Weight: 180Kg/m2
Completed Slab Weight: 730Kg/m2
Finished Slab Thickness: 450mm

COMPONENT	WELDS/M2
Bottom Reinforcement Mesh	42.7
Top Reinforcement Mesh	23.3
Girders/Mesh	17.3
TOTAL WELDS PER M2:	83.3

WELDS PER 1 LINEAR METRE-3m	ELEMENT
Bottom Reinforcement Mesh	128
Top reinforcement Mesh	70
Girders/Mesh	52
TOTAL WELDS 1 x 3 Metre:	250

BALLS	
Diameter	360mm
Max No. Per m2	6.25

CONCRETE	
Precast Concrete Thickness	60mm
Site Concrete o/all thickness	390mm
Site Concrete Quantity(m3/m2)	0.25
(Including allowance for areas of balls left out and grout loss)	

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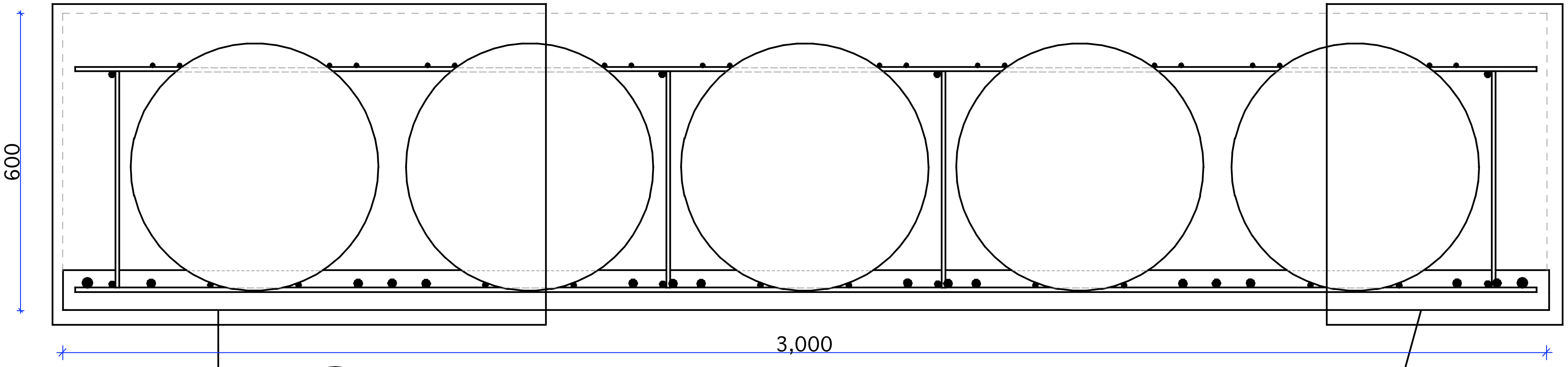
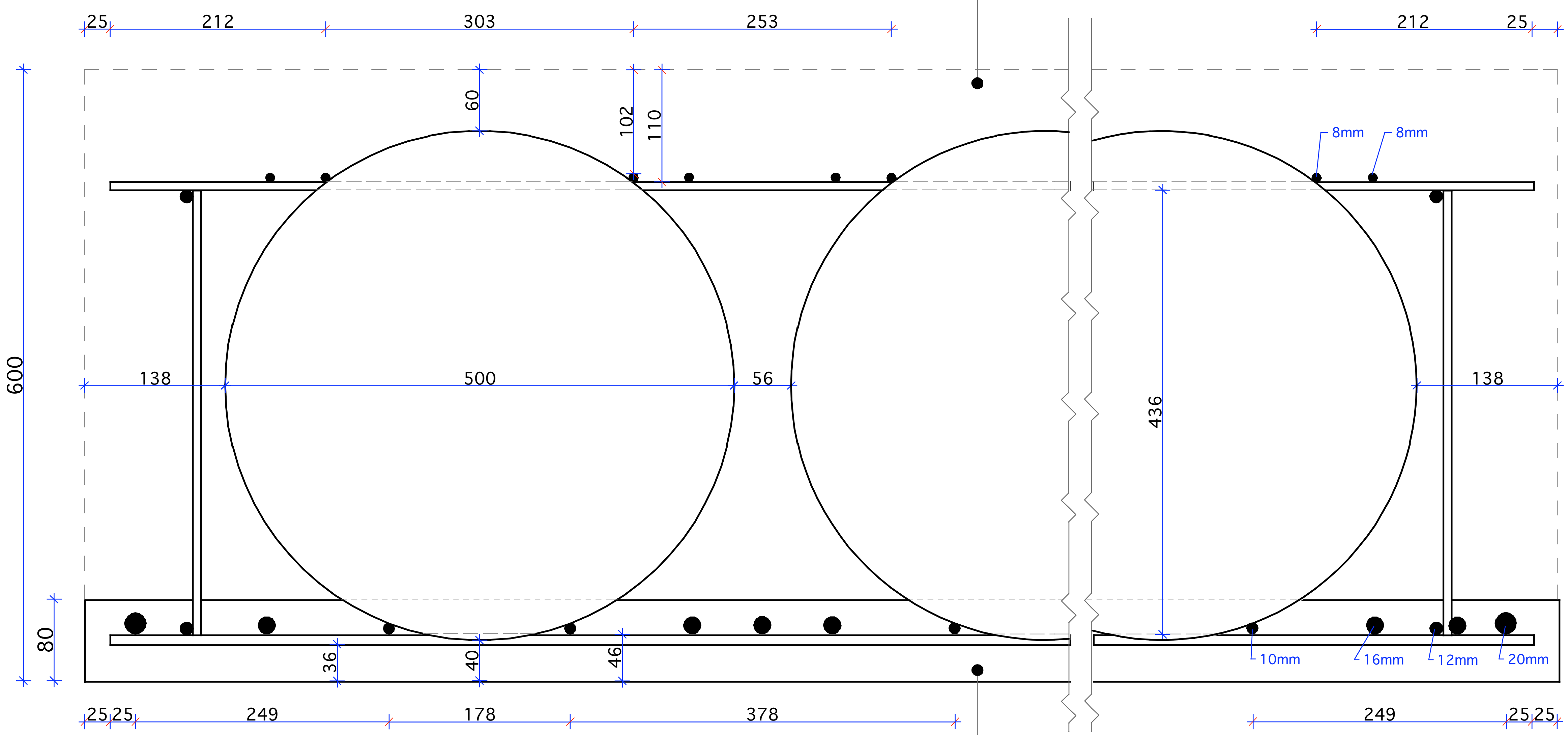
PROJECT / LOCATION		DRAWING	
BubbleDeck Standard Element Detail		BD 450 Element Sections Type A - Filigree Element 3m wide option	
CLIENT		DRAWN	CHECKED
		MT	
DATE	SCALE	DRAWING NUMBER	
February 2006	1:5/1:10	BDS001/15	



1:10 Element Section
BD 600 3 metre Wide Element



1:5 Detail Section
BD 600 End Section

MESH REINFORCEMENT CONFIGURATIONS										
	Mesh	Cover Top Main	Cover Top Secondary	Cover Bottom Main	Cover Bottom Secondary	Cover Top Bubble	Cover Bottom Bubble	Spacing Top Main	Spacing Top Secondary	Spacing Bottom Main	Spacing Bottom Secondary
As drawing	10	102	110	46	36	60	40	303/253	310/246	178/378	110/446

FIRE RESISTANCE To BS 8110-1:1997		
	Bottom Concrete Cover	FR Period
As drawing	36mm	2 Hours

This drawing must be read in conjunction with all other drawings, details and specifications issued by the Architect, Structural Engineer and other Consultants or approved specialists. Discrepancies between any other drawings, details and/ or specifications must be referred to BubbleDeck C.I. for verification at least 7 days prior to commencement of the work. It is the Main Contractor's responsibility to ensure all work is carried out in accordance with all statutory requirements and to the approval of the Building Control Officer.

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REVISIONS

BD600 SPECIFICATIONS
Precast Element Weight: Approx.185Kg/m²
Completed Slab Weight: 980Kg/m²
with Full Bubble Pattern
Finished Slab Thickness: 600mm

COMPONENT	WELDS/M²
Bottom Reinforcement Mesh	24
Top Reinforcement Mesh	26.7
Girders/Mesh	18.7
TOTAL WELDS PER M²:	69.4

WELDS PER 1 LINEAR METRE-3m	ELEMENT
Bottom Reinforcement Mesh	72
Top reinforcement Mesh	80
Girders/Mesh	56
TOTAL WELDS 1 x 3 Metre:	208

BALLS	
Diameter	500mm
Max No. Per m²	2.986

CONCRETE	
Precast Concrete Thickness	80mm
Site Concrete o/all thickness	520mm
Site Concrete Quantity(m³/m²)	0.328
(Including allowance for areas of balls left out and grout loss)	

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PROJECT / LOCATION	DRAWING
Car Park Solution 16m Long Span Element Configuration 2.5kN/m² Line Load	BD 600 Element Sections Double Span 3m Wide Element

CLIENT	DRAWN	CHECKED
	MT	PH

DATE	SCALE	DRAWING NUMBER
May 2006	1:5/1:10	BDS CP 01/02

ZURICH

Mr C Dewhirst
Bubbledeck UK
4 Firs Close
Ellesmere Road
St Martins
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SY11 3LT

Your reference

Our reference ZBG/APP/06/15

Date 31/06/06

Zurich Building Guarantee

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www.zurichmunicipal.com

Direct phone: 01252 387599
Direct fax: 01252 372989
E-mail:
carl.nelson@uk.zurich.com

Dear Mr Dewhirst

Re: Bubbledeck System

In confirmation of the technical assessment, factory visit and review of the paperwork you submitted to Zurich Building Guarantee. It gives me great pleasure to inform you that the Bubbledeck System can be used where Zurich Building Guarantee is the warranty provider.

This will only be acceptable to us as long as only correctly trained and approved contractors are used to install the product as directed in accordance with the manufacture's recommendations and Bubbledeck UK issues their certificate upon completion of the works.

Our Technical Standards and Building Regulations must be complied with. As a Designated Warranty provider and Approved Inspector we are also able to offer Building Control approval and warranty insurance on developments.

If I can be of further assistance please contact me.

Yours sincerely

C R Nelson

Carl Nelson MSc FCIOB
Technical Manager
Zurich Building Guarantee

KOMO Certificate

NL/SfB: (23) Gf2

Kiwa N.V.
Certification & Testing
Sir Winston Churchill-laan 273
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Tel: 070 414 44 00
Fax 070 414 44 20

Kiwa
Partner for progress

RvC
Accredited by the
Council for Accreditation

BubbleDeck Floor System

Number: K22722/01
Published: 15/11/2002

Certificate Holder
BubbleDeck Nederland BV

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Internet: www@BubbleDeck-UK.com

KIWA DECLARATION

This certificate is based on BRL 0203, "Self supporting floor system of pre-fabricated concrete construction", conforming to the KIWA regulations for product certificate delivered by KIWA.

KIWA declares that the BubbleDeck Floor System performs as described in this certificate, if:

- the pre-fabricated parts of the floor comply with the technical environmental and physical specifications as laid down in this certificate.
- the pre-fabricated parts of the floor are being used in compliance with the fabrication methods laid down in this certificate.
- the application conditions are complied with as described in this certificate.

For relationship between the verdicts of this certificate and the prescription of the Building Decree and the Material Decree refer to the list of quality declarations as published twice yearly by the Foundation for Building Quality (SBK) in Rijswijk.

Ing. B Meekma,
Director Certification and Testing, Kiwa N.V.

Users of this certificate are advised to ascertain with Kiwa if this document is still valid

KOMO brand Logo:

This certificate consists of 25 pages

CONTENTS

1. TECHNICAL SPECIFICATION

1.1 SUBJECT
Floor slab comprising structural reinforced concrete containing hollow (bubbles) spherical concrete saving elements.

Parts of the floor construction are pre-fabricated. There are three alternatives:
- Reinforcement Modules in which the bubbles are trapped between the upper and lower reinforcement mesh;
- Filigree Elements as above, but also with a pre-fabricated concrete biscuit cast onto the bottom reinforcement mesh (permanent formwork);
- Pre-cast Finished Planks in which the reinforcement modules have been cast into concrete to full finished depth.

The three alternatives are pictured in *figure a* on the drawing sheets of this certificate.

With the first two alternatives using partially pre-fabricated floor elements, the reinforcement modules or filigree elements are connected together with splice bars and joint mesh and afterwards concreted to full depth.

For the completely pre-cast Finished Planks only the joints between the planks are filled.

1.2 PRE-FABRICATED ELEMENTS
The pre-fabricated elements of the Bubble Deck floor have to be delivered according to the KOMO product certificate. In the product certificate concerned a reference will be made to this certificate.

This is a guarantee that the pre-fabricated parts meet the requirements included in the certificate, and that the manufacturer of the pre-fabricated parts uses materials that comply with the requirements in the product certificate concerned and with the requirements in 1.2.1 up to 1.2.2

Depending on the pre-fabricated part, the directive can vary on the basis of which the KOMO product certificate concerned has been given. This is mentioned in 1.2.3 up to 1.2.5

This certificate is only valid in combination with the related KOMO product certificates.

1.2.1 Bubbles
The bubbles are made of material that doesn't react chemically with the concrete and/or the reinforcement steel. The bubbles are non-porous and possess enough strength and stiffness to carry applied loads safely in the phases before and during the pouring of the site concrete.

1.2.2 Girder Reinforcement
Girder Reinforcement used as cross element reinforcement has to comply with the requirements in clause 9.11.1.7 of CUR-Recommendation 86.

When the Girders are not being used as cross element reinforcement then they only have to comply with the functional requirements during production, transport and pouring.

1.2.3 Reinforcement Modules
The reinforcement modules have to comply with KOMO product certificate on the basis of BRL 0503.

1.2.4 Filigree Elements
The Filigree Elements have to comply with KOMO Product certificate on the basis of BRL 0203.

1.2.5 Pre-cast Finished Planks
The completely pre-cast Finished Planks, where the reinforcement module is completely cast into full depth concrete, have to comply with KOMO product certificate on the basis of BRL 0203 or BRL 2813.

1.2.6 Form and Dimensions
The dimensions of the parts of the floor are specified in table 1 and in the drawings in this certificate.

Table 1 – Dimensions

	Nominal Dimensions
Slab Depth	230, 280, 340, 390, 450 mm
Thickness of the concrete biscuit	≥ 60 mm
Bubble Diameter	180, 225, 270, 315, 360 mm
Diameter Tolerance	+ 0/- 6mm
Distance between Bubbles	≥ 1/9 of bubble diameter

1.3 FLOOR CONSTRUCTION

1.3.1 Form and Composition
The floor is made as Bubble Deck floor corresponding to CUR-Recommendation 86.

1.3.2 Adjoining Structures
The structure in which the floors are used belong to one of the categories according to 2.2 of NEN 6720.

1.3.3 Erecting the Pre-Fabricated Elements
When either the Reinforcement Modules or the Filigree Elements are being used before erection they have to be supported on temporary propping or permanent supporting surfaces, according to the supplier's regulations.

In the case of permanent supporting surfaces, corresponding to details 1a and 2a, Class 1 floor load can be taken to be achieved if, according to the design, the supporting length *a* is at least equal to:
- 90mm in case of being supported on top of blockwork;
- 80mm in case of being supported by reinforced or un-reinforced concrete;
- 70 mm in case of being supported by steel beams or adequately rigid steel flanges.

When applying details 1 and 2, the level of the supporting surface has to comply with figure b. In case of larger measured deviations use d_{aanw} for the effective height.

In floor load class 2, according to NVN 6725, it is possible that after constructing the supporting structure, a pressure-distributing supporting material will be required. For instance, this can be the case with supporting brickwork surfaces or cast concrete surfaces.

In case of supporting structures corresponding to detail 1 and 2, it can be necessary to shutter the joint between the floor slab and the side with a wooden batten, or equivalent, before casting site concrete. Expanding foam is not allowed for this purpose. See figure c.

1.3.4 Joint Filling
The joints between the completely pre-cast Finished Planks are filled with concrete or cement / sand mortar with the following characteristics:
- strength class ≥ B15,

- largest aggregate grain dimension ≤ 8 mm
- where the joint width is ≥ 50 mm, then the largest aggregate grain dimension can be 16 mm.
- gravel, sand and cement meeting the requirements of NEN 5905 and NEN 3550.

The values for the characteristic push and/or pull strength should not be bigger than the values that apply for B15 if the joint width ≤ 50 mm.

1.3.5 Holes, Pipes, Conduits and electrical boxes

The pre-fabricated (Finished Planks) can be supplied with pre-fabricated holes having a minimum dimension of 250 x 250 mm^2 and incorporating electrical boxes.

The maximum dimension of the holes varies dependant upon the project and has to be determined together with the builder.

During manufacture a row of balls can be omitted to install pipes or service conduits.

In consultation with the builder, holes less than 250 x 250 mm^2 have to be drilled on site.

1.3.6 Site poured Concrete

The site poured concrete to complete the floor slabs:
- has to comply with NEN 5950
- has to comply with the respective application agreed environment class according to 4.3 of NEN 5950
- has an aggregate grain dimension, D_{max} ,of maximum 3 to 15 mm

Note: where 230mmor 280mm thick floor slabs are used, because of the small dimensions between bubbles, the use of aggregate with D_{max} = 10 mm dimension will be necessary.

1.3.7 Mass of the Floor

For strength and bending calculations a mass of reinforced concrete of 2400 kg /m^2 is used. The exact weight of the floor will be given for specific projects.

Table 2 - Mass with full bubble pattern (kg/m^2)

Floor Type	Element Width 3000 mm	Element Width 2400 mm
BD 230	380	390
BD 280	450	490
BD 340	550	580
BD 390	650	660
BD 450	740	720

2. USE

2.1 GENERAL

During use conform to the requirements of 1.3

2.2 LIFTING, STORAGE AND TRANSPORT

Work in connection with lifting, storage and transport should not give the possibility of damaging and/or splitting of the floor elements. The elements have to be stored raised off the floor on bearers or directly on top of each other on top of wooden packing.

The Reinforcement Modules and Filigree Elements have to be lifted by the vertical girders with the lifting hooks inserted underneath the angle between the girder web reinforcement.

For lifting of Filigree Elements use a chainset with four chains having a minimum length of 6 metres each connected to another pair of chains terminating in hooks, so that it can be lifted in 8 places. Filigree Elements smaller than 4 metres long can be lifted with 4 hooks, with the exception of elements with a pre-cast concrete layer of ≥ 70 mm and where the element profile requires more than 4 points. The lifting and the use of aids for pre-fabricated elements must be undertaken according to the respective supplier's instructions.

2.3 PERMANENT SUPPORTING SURFACES

For supporting structures according to details 1a and 2a, the points of support have to be flat and even in the area of the supporting surfaces.

For practical reasons it can be necessary to omit the first row or rows of bubbles. Refer to the details on the drawing sheets in this certificate.

2.4 TEMPORARY PROPPING

Temporary propping of the elements has to be undertaken according to the supplier's instructions. The propping beams have to be erected parallel and at right angles to the vertical reinforcement girders - see figure d. When placing the props, strips of formwork should be applied at the construction joints between the transverse joints and if necessary between the longitudinal joints of the elements

The top and diagonal, web, bars of the vertical reinforcement girders must not be cut without the supplier's agreement. If these reinforcement bars are cut extra propping has to be provided in the area of the break to prevent bending of the elements during pouring of site concrete.

Until the site concrete has gained adequate strength the temporary propping is required to provide temporary support and cannot be removed. The temporary propping must not be too quickly removed and dismantling must be undertaken with due care.

Temporary propping must only be erected on a stable platform without any movement and cannot be erected on insulation material.

2.5 JOINT FILLING

The water-cement ratio for mortar joint filling the completely pre-fabricated Finished Planks has to be specified so the mortar does not leak between the elements. If necessary the joints have to be cleaned before hand and be dampened with water.

In case of low outside temperature and frost, the measures as mentioned in 9.4.4 of NEN 6722 are also applicable on the joint filling.

2.6 HOLES

Forming holes as described in 1.3.5 has to conform to the supplier's instructions.

2.7 MEASURES IN CASE OF FROST

Avoid drilling of the plastic bubbles, for instance when fixing temporary formers such as construction joint, perimeter or column shuttering.

Rainwater can run into the plastic bubbles through drilled holes and during cold weather could freeze causing damage to the floor.

Note: If a bubble has been drilled, a small hole has to be drilled through the concrete skin on the underside of the bubble to allow water to drain out.

2.8 SITE POURED CONCRETE

The concrete has to be poured with even distribution to avoid heaps of concrete mortar.

Due to the limited space between the bubbles to compact the concrete and remove any entrained air during pouring a thin vibrating poker has to be used. Avoid separation occurring due to the vibrating of shuttering and/or bubbles that can result in segregation of the concrete mix.

3 FEATURES

3.1 Construction Safety

3.1.1 Floor Construction Strength, BB article 2.1 up to and including 2.3, 2.4b, 2.5, 174.1 up and including 174.3, 174.5b, 174.6, 359.1, 359.2, 359.3, 359.5b and 359.6

It can be determined if the floor construction meets the above mentioned requirements of the Building Decree according to NEN 6720 and CUR-Recommendation 86.

3.1.2 Drawings and Calculations

Floor drawings and calculations have to be drawn up by the supplier for each floor to demonstrate the floor construction meets the above mentioned requirements of the Building Decree.

3.2 FIRE SAFETY

3.2.1 Strength in case of Fire, BB article 2.6, 2.7, 174.7, 174.9, 359.7

The floor constructions fire resistance against collapse time period is determined according to above mentioned articles of the Building Decree and CUR-Recommendation 86.

3.2.2 Limitation of events causing Fire, BB article 12.1, 12.2, 12.5, 184.1, 184.2, 184.5

A floor comprising un-insulated concrete floor elements is fireproof as mentioned in NEN 6064 so that it meets the above mentioned requirements of the Building Decree.

Note: Insulated floor elements are only used for ground level above crawl spaces or spaces of limited height.

3.2.3 Limitation of Fire Development, BB article 13.1, 13.2, 13.4, 185.1, 231.1, 231.3, 231.6, 256.1, 256.3, 256.6 and 369.1

The top and bottom of the floor complies with at least class T1 (determined according to NEN 1775) or class 2 (determined according to NEN 6082) for resistance against fire spread, taking into account this is not valid for the underside of an insulated floor.

Note: Insulated floor elements are only used for ground level above crawl spaces or spaces of limited height.

3.2.4 Limitation of the Spread of Fire / Escape in case of Fire, BB article 14.1, 14.2, 14.3, 15.1, 232.2, and 257.3

The fire resistance of the floor in connection with the separating function is equal to the smallest value calculated according to 3.2.1 of this certificate with product certificate or 120 minutes.

Note: For ground floors above crawl spaces or spaces of limited height, the requirement in connection with fire resistance of the floor construction in connection with the separating function is not relevant.

3.2.5 Limitation of the Cause of Smoke, BB article 16.2, 16.3, 187.1, 233.1, 233.2, 233.4, 258.1, 258.2 and 258.4

The smoke density of the floor elements determined according to NEN 6066 is smaller than 10 m^1 on both sides, knowing that this decision is not valid for the underside of insulated floor elements.

Note: The Building Decree does not require this property for a side of a floor facing a crawl space or a space of limited height.

3.2.6 Limitation of the Spread of smoke, BB article 16.5, 234.9 and 259.7

The smoke resistance of the floor determined according to NEN 6075 equals 1.5 x the value of the fire resistance in connection with the separating function.

Note: For ground floors above crawl areas of areas of limited height the requirement in connection with the smoke resistance of the floor construction is not relevant.

3.3 HEALTH

3.3.1 Protection against External Noise

Protection against external noise has to be determined for the complete construction to determine if this meets the specified requirements. Use, amongst others, the mass per surface of the floor construction for this.

3.3.2 Sound Proofing Between Spaces, BB article 24 and 267

Application examples of joint and connection details are included on the drawing sheets in this certificate with product certificate. These details guarantee that sound transmission via the floors will meet the requirements of the Building Decree.

For testing of the requirements in the Building Decree, a judgement of the total construction is required. Apart from the floor elements and their connection details other construction parts are also important including their connection details. For these connection details refer to the quality declarations of the different building parts.

NPR 5070 includes application examples that comply with article 24 and 267 of the building decree.

When using NPR 5070 take into consideration the conditions for the use of this practice directive according to supplement C ”Explanation use NPR 5070 for areas where you remain.”

The reservations included in supplement C of NPR 5070 are, as far as it concerns the noise transmission via the floor, not applicable if the dwelling separating floor has a mass per surface of at least 600 kg/m^2 and, if present, the dwelling separating wall has a mass per surface of at least 550 kg/m^2.

For the definition of the mass per surface of the floor construction in connection with noise, use table 1 of NPR 5070 for the mass of the site concrete and/or the pre-cast concrete.

For materials which are not mentioned in NPR 5070, the volume mass needs to be determined with the materials equilibrium humidity according to NEN 5967.

3.3.3 Damp Proofing from the Outside, BB article 26.4 and 197.4
When the floor elements are being used as roof slabs, the water tightness has to be guaranteed with additional waterproofing layers.

For the use of waterproofing layers refer to NPR 2652.

Openings made for manholes, pipes and closures of joints and other openings in the ground floor need to be airtight in order to get a sufficient low level of the volume airflow. More information can be found in NPR 2652.

For the examples of joints and connection details of the ground floor on the drawings of this product certificate the floors specific volume airflow according to NEN 2690 can be established without counting pipes and openings.

3.3.4 Damp Proofing from the Inside, BB article 271, 27.2, 198.1, 198.2, 268 and 309
The requirement concerning the intrusion of damp from the toilet or bath area into the construction has to be met through the application of a floor waterproofing finish.

In the table in connection with the temperature factor is the temperature factor given for relevant details or detail combinations. There is a difference between the temperature factor of the detail drawn (f_{2d}) and the temperature factor of a combination of two details in an external corner (f_{3d}, meeting between transverse facade, longitudinal facade and ground floor). The values concern the temperature factor of the inner surface of the external separation construction determined according to NEN 2778.

The detail where the dwelling separating wall, end gables and the ground floor meet always has a greater temperature factor than the detail in the area of the external corner (f_{3d}), given that the head of the dwellings separating wall is provided with an equal thermal insulation. All details mentioned of the drawings in this certificate concerning the meeting of the external separation construction (head-or end gable) with a dwelling separating floor have to comply with $f_{3d} \geq 0.65$, given that the gable is insulated up to a level of Rc equal to 2.5 m^2K/W determined according to NEN 1068.

For details or detailed combinations not included in the table, or if the table doesn't provide a definite answer, in so far as this is relevant for the detail concerned and if the calculation conforms with NEN 2778, it has to be proven that it meets the requirement.

3.4 Use

3.4.1 Floor Construction Stiffness, BB article 68.1, 68.2b, 68.3, 225.1, 225.3b and 225.4
If the floor construction agrees with above mentioned articles of the Building Decree and article 3.3 of BRL 0203 can be determined according to NEN 6720.

3.4.2 Drawings and Calculations
For each floor, drawings and calculations will be compiled by the supplier showing that the floor construction complies with the above mentioned articles of the Building Decree and article 3.3 of BRL 0203.

3.5 ENERGY SAVING

3.5.1 Thermal Insulation, BB article 70.1, 70.3, 227.1 and 227.3
The heat resistance of the floor determined according to NEN 1068 is shown in table 2.

Table 2 - Thermal Insulation

R_c value (m^2K/W)	Thickness of insulation material
2.5	100 mm
3.0	120 mm
4.0	160 mm

The values of the heat resistance mentioned in table 2 are determined with the value for the heat conductive coefficient of the insulation material. $\lambda = 0.040$ W/m^2K

3.5.2 Air Permeability, BB article 71.1 and 228.1
The airflow leakage of the floor construction, determined according to NEN 2686, follows from the total of the air volume stream of the floor breakthroughs. The airflow leakage through the floor is negligible if the joint and connection details are executed according to the drawings of this certificate.

4 TIPS FOR THE USER

4.1 Inspect on delivery products mentioned under "technical specification" if:
4.1.1 what has been delivered has been agreed;
4.1.2 the brand and the method of branding are correct;
4.1.3 the products do not have visual faults due to transport or otherwise.

4.2 Inspect on delivery products mentioned under "application" if they comply with the stipulated specification.

4.3 If you decide to reject the products contact:
4.3.1 BubbleDeck C.I. Ltd.;
and if necessary:
4.3.2 Kiwa N.V.

4.4 Store, transport and apply according to the instructions mentioned under "application".

4.5 Take care of the application conditions mentioned under "Features".

5. LIST OF DOCUMENTS MENTIONED*

BRL 0203	Self supporting floor system of pre-fabricated concrete construction.
BRL 2813	Concrete building elements.
BRL 0503	Reinforcement constructions and bent and woven work for reinforcement centres.
NEN 1068	Thermal insulation of buildings.
NEN 1775	Definition of the contribution to fire-spreading of floors.
NPR 2652	Damp-proofing in residential buildings – Internal Damp-proofing - External Damp-proofing - Examples of building constructions.
NEN 2686	Air permeability of buildings. Measuring methods.
NEN 2778	Damp-proofing in buildings. Definition methods.
NEN 3550	Cement. Definitions, requirements and testing.
NPR 5070	Noise-proofing in residential buildings - examples of wall and floor constructions.
NEN 5905	Materials for concrete. Materials with a volume mass of at least 2000 kg/m^2
NEN 5950	Instructions Concrete. Technology (VBT). Requirements, manufacturing and testing.
NEN 5967	Concrete. Definition of the volume mass.
NEN 6008	Concrete steel.
NEN 6064	Definition of the fire resistance of building materials.
NEN 6066	Definition of the smoke production in case of fire of building material (combinations)
NEN 6068	Definition of the resistance against fire breakthrough and fire spreading between areas.
NEN 6075	Definition of the resistance against smoke spreading between areas.
NEN 6082	Fire safety of buildings. Houses and residential buildings. Performance requirements.
NEN 6702	Technical foundations for building constructions. Loads and transformations.
NEN 6720	Prescriptions concrete. Constructive requirements and calculation methods (VBC).
NEN 6722	Prescriptions concrete execution (VBU).
NVN 6725	Self supporting floor system of pre-fabricated concrete.
CUR Recommendation 86	Bubble Deck Floors, December 2001
Building Decree	The Building Decree

*For the correct verdict of the norms mentioned, refer to the last amendment sheet to BRL 0203.

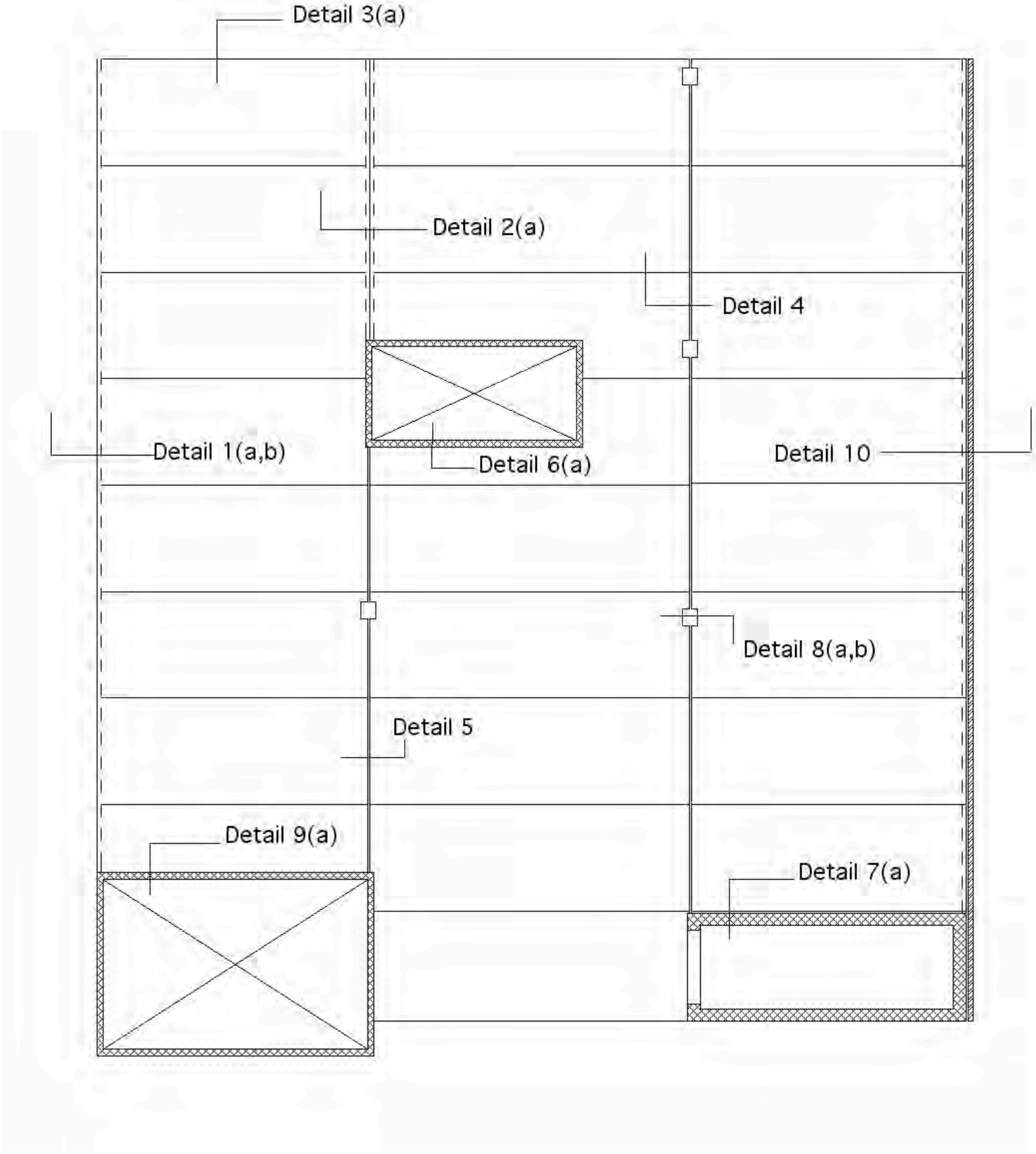
6. DRAWINGS

The unnamed materials (shaded wall and cavity walls) relate to a masonry material with a mass per surface of minimum 170 kg/m^2 and a calculation value of the heat conduction coefficient λ of minimum 0.8 and maximum 2.0 W/mK, like for instance chalk sandstone, traditional masonry and concrete.

Table 3 - Overview of the details and temperature factors for floor constructions with a heat resistance $R_c \geq 2.5$ m^2 K/W

detail	$f_{2d} \geq 0.5$	$f_{3d} \geq 0.5$	$f_{2d} \geq 0.65$	$f_{3d} \geq 0.65$
1/BG	sufficient	sufficient in combination with detail 1 BG	insufficient	insufficient
2/BG	sufficient	sufficient in combination with detail 2 BG	insufficient	insufficient
2/BG	sufficient	sufficient in combination with detail 1 BG*)	insufficient	insufficient
3/BG	sufficient	sufficient in combination with detail 1 BG	insufficient	insufficient

*) In this situation, the insulation layer against the cavity wall internal masonry leaf has to be continued to the underside of the floor construction. The insulation layer has to be carried out up to a minimum of 75 mm above the foundation in a damp-resistant material.


Detail 3(a)
Detail 2(a)
Detail 4
Detail 1(a,b)
Detail 6(a)
Detail 10
Detail 8(a,b)
Detail 5
Detail 9(a)
Detail 7(a)

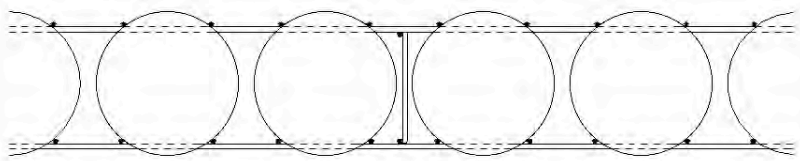


Figure a

Reinforcement Modules where the bubbles are trapped between the upper and lower reinforcement mesh.

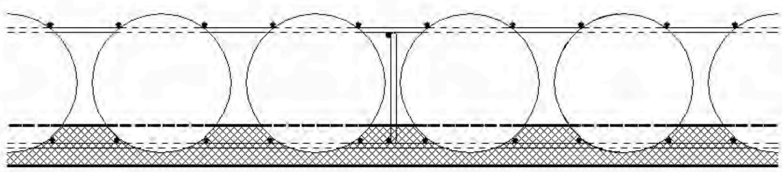


Figure a

Filigree Elements where the bubbles are trapped between the upper and lower reinforcement supplied with pre-cast concrete permanent formwork.

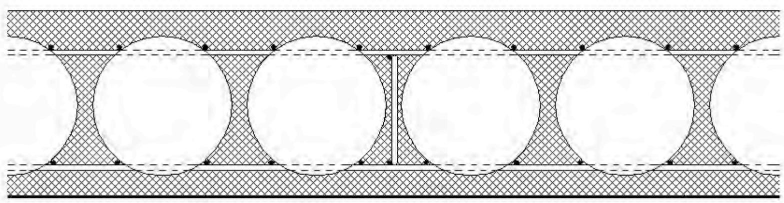


Figure a

Completely pre-cast Finished Planks where the reinforcement modules are encased with full depth concrete.

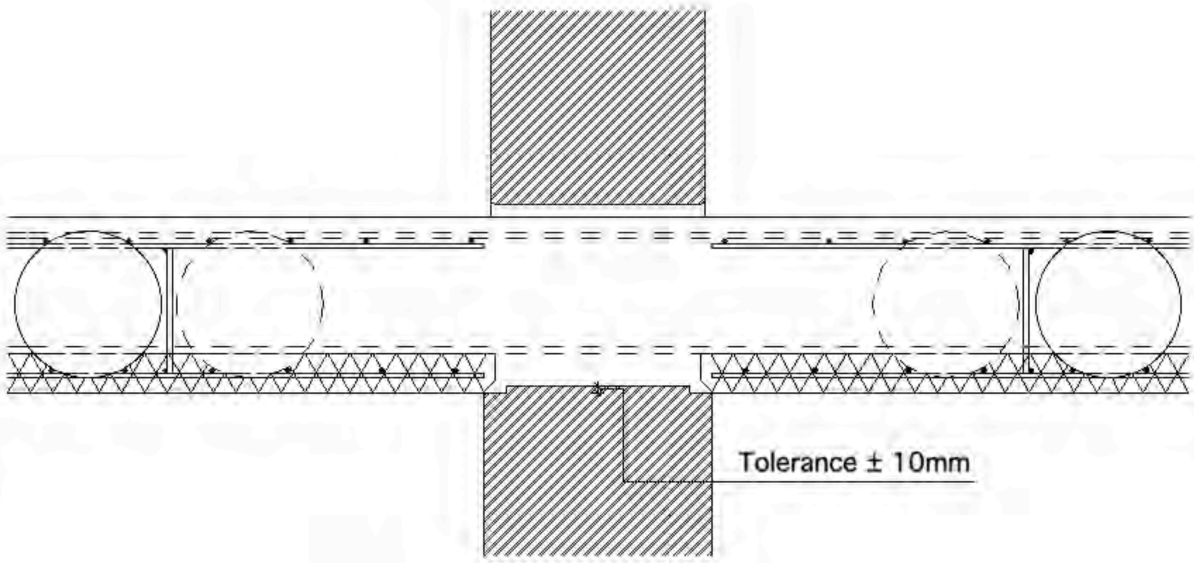


Figure b

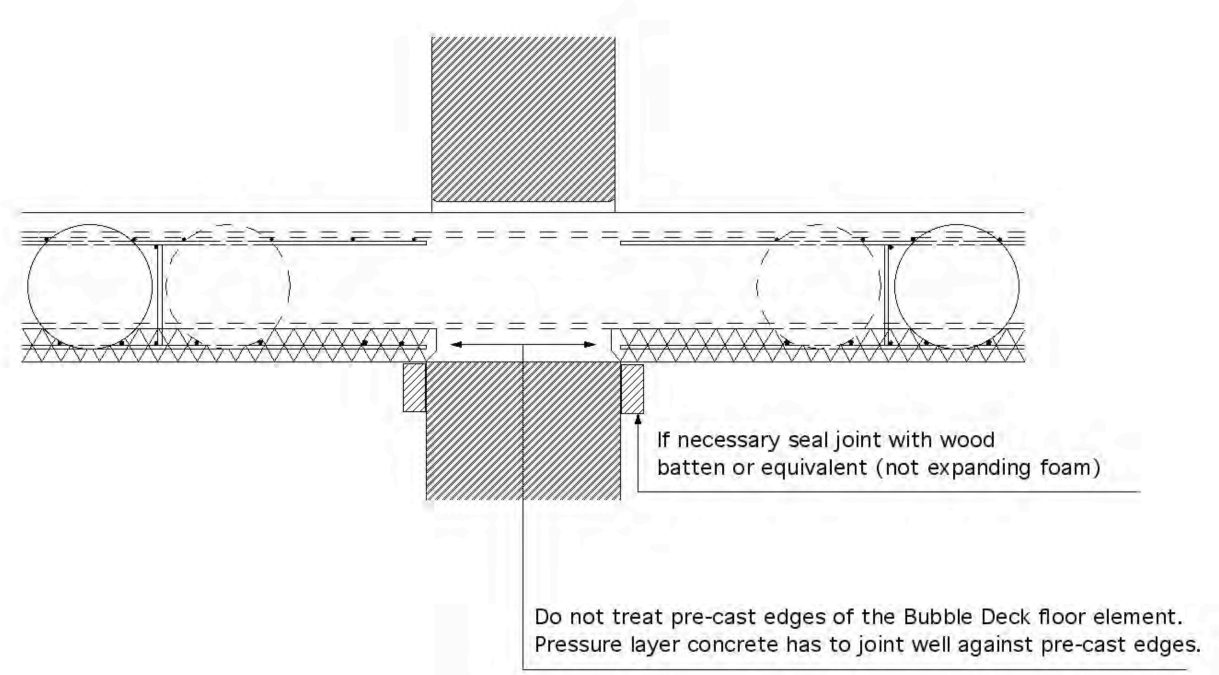


Figure c

Construction phase.

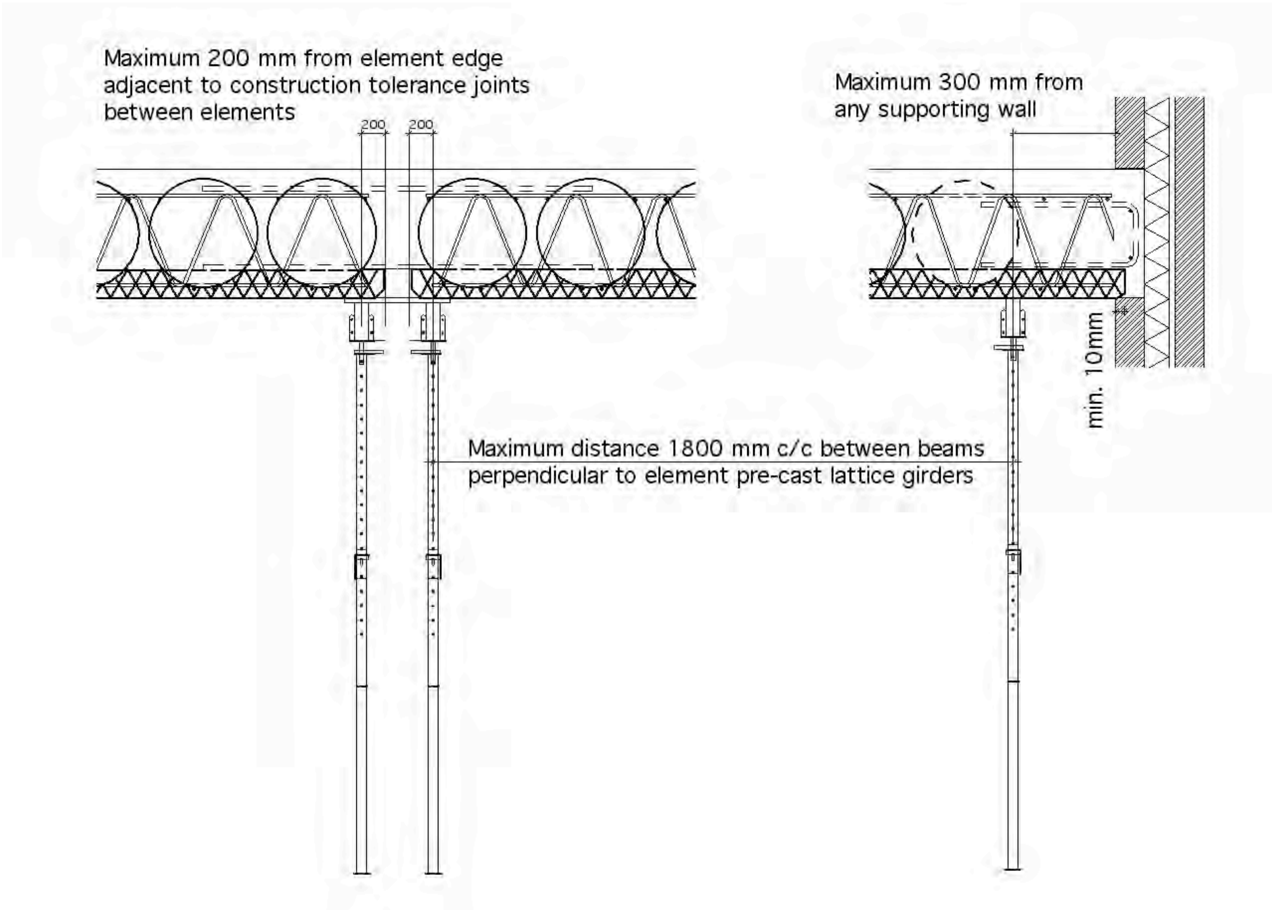


Figure d

Temporary propping of BubbleDeck elements

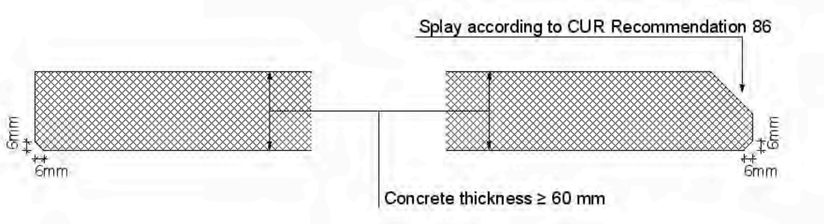


Figure e

Edge detail through longitudinal joint in pre-cast concrete biscuit of BubbleDeck Filigree Element - variant a.

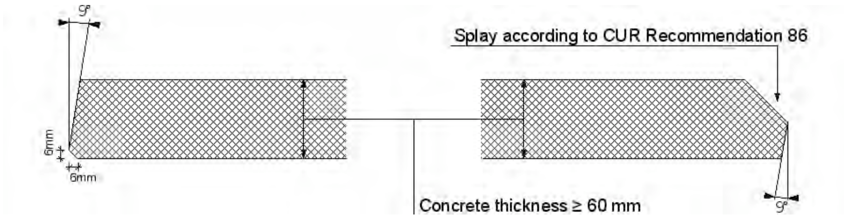


Figure e

Edge detail through longitudinal joint in pre-cast concrete biscuit of BubbleDeck Filigree Element - variant b.

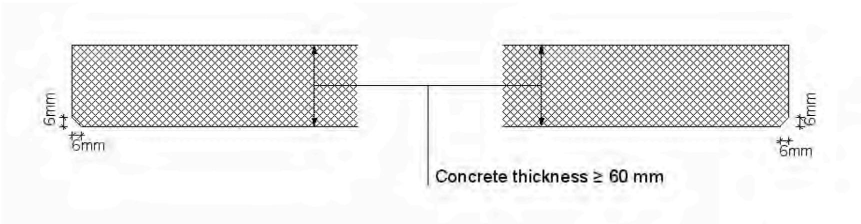
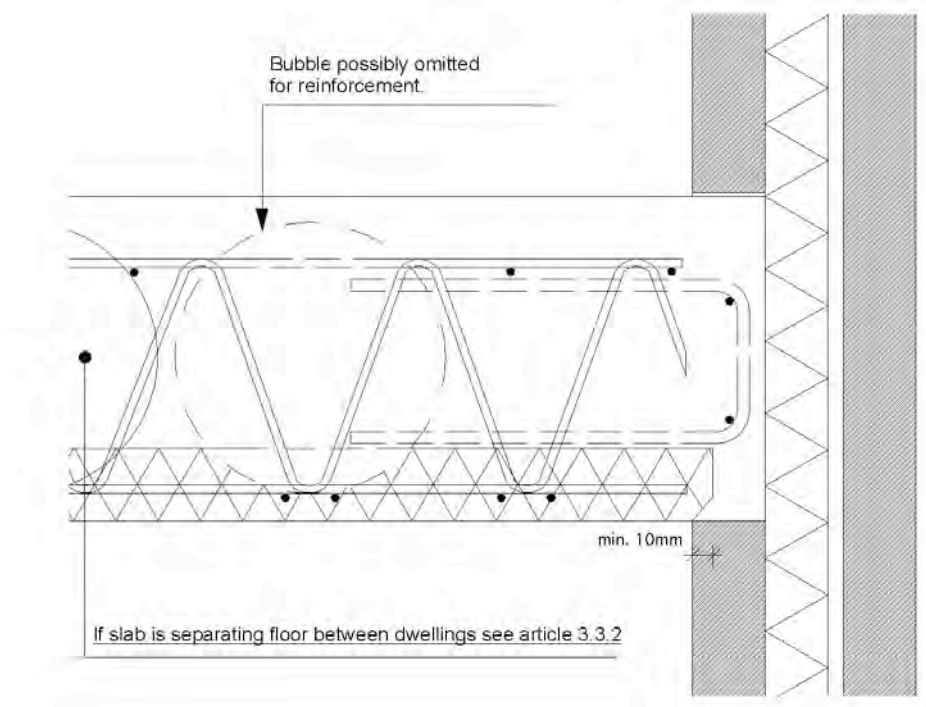
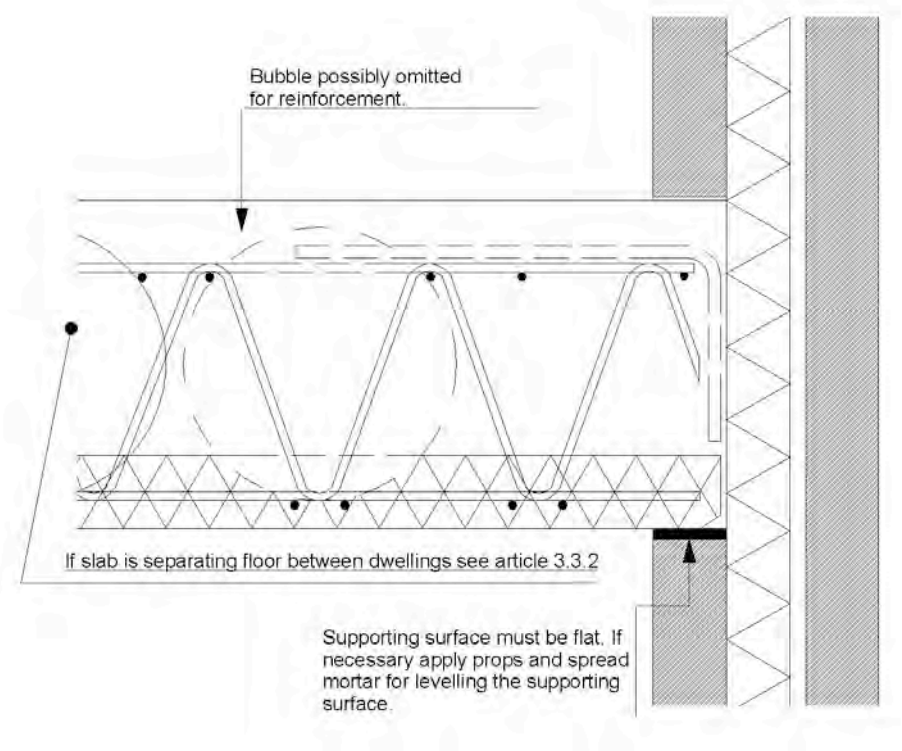


Figure e

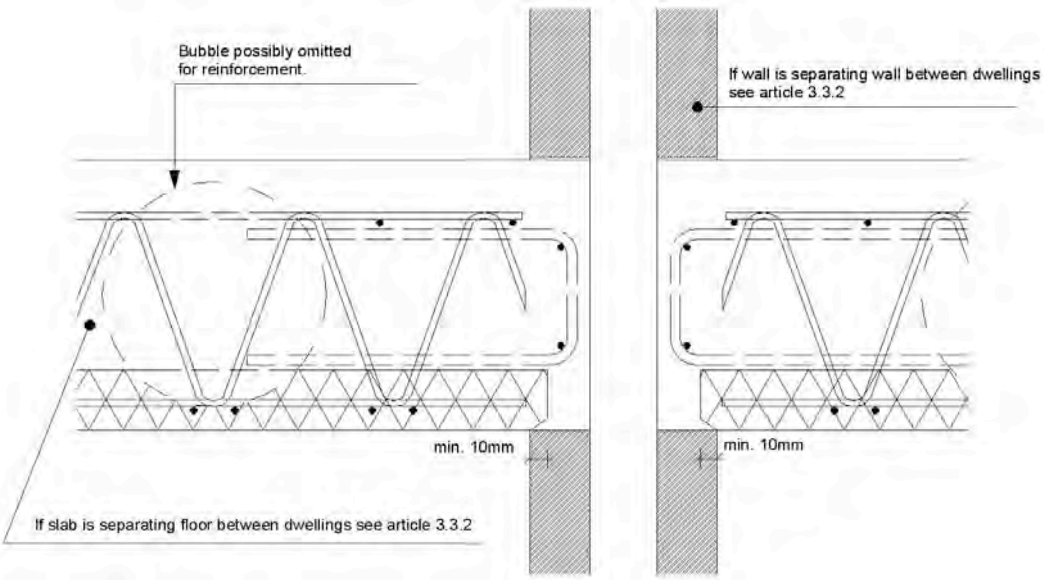
Edge detail through short edge joint in pre-cast concrete biscuit of BubbleDeck Filigree Element.



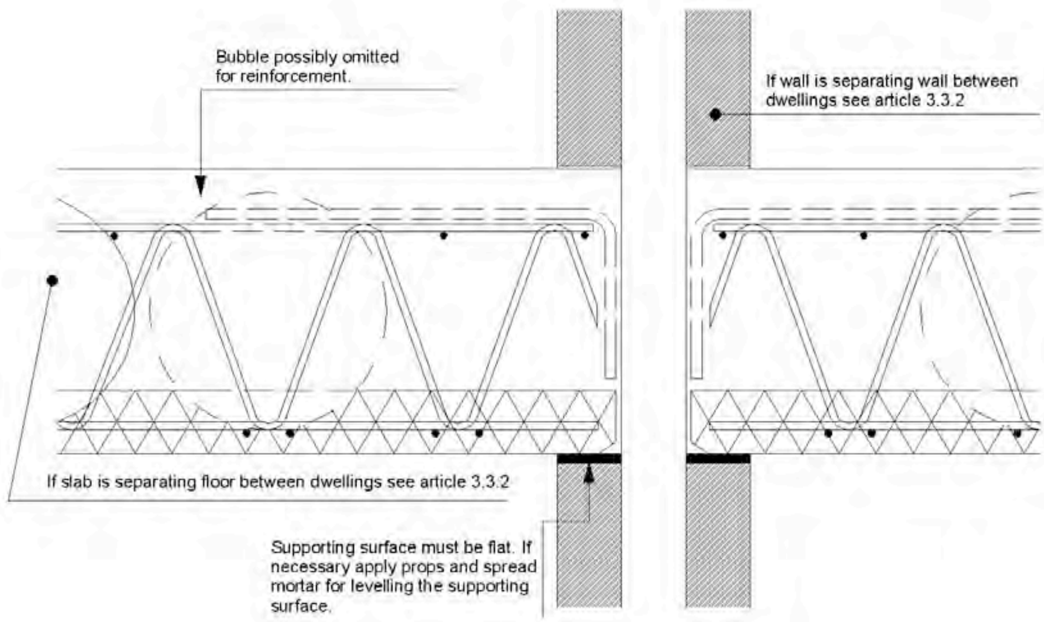
Detail 1



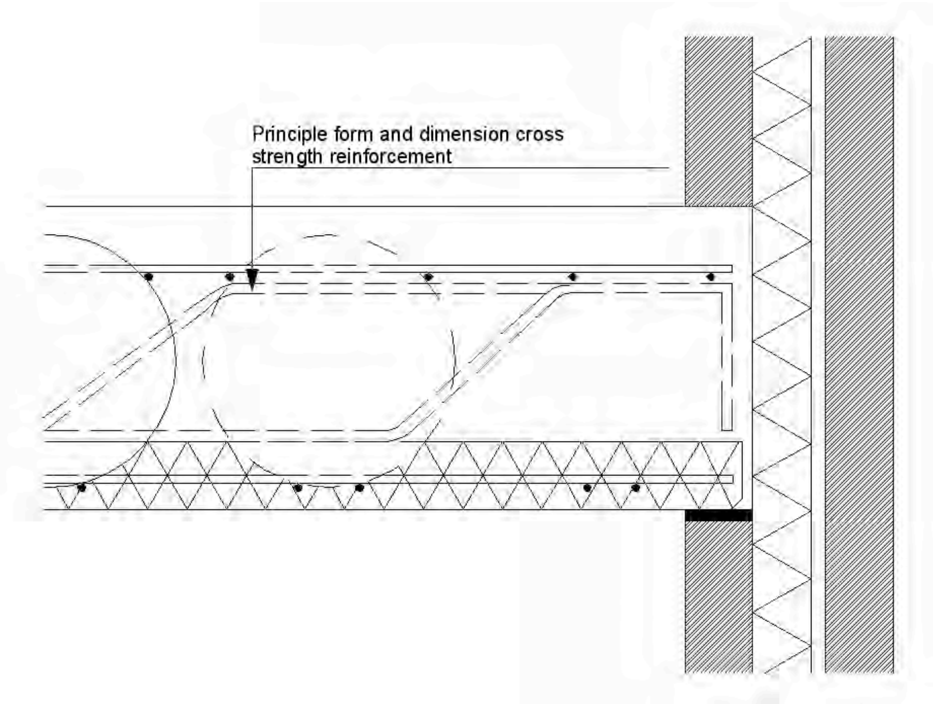
Detail 1a



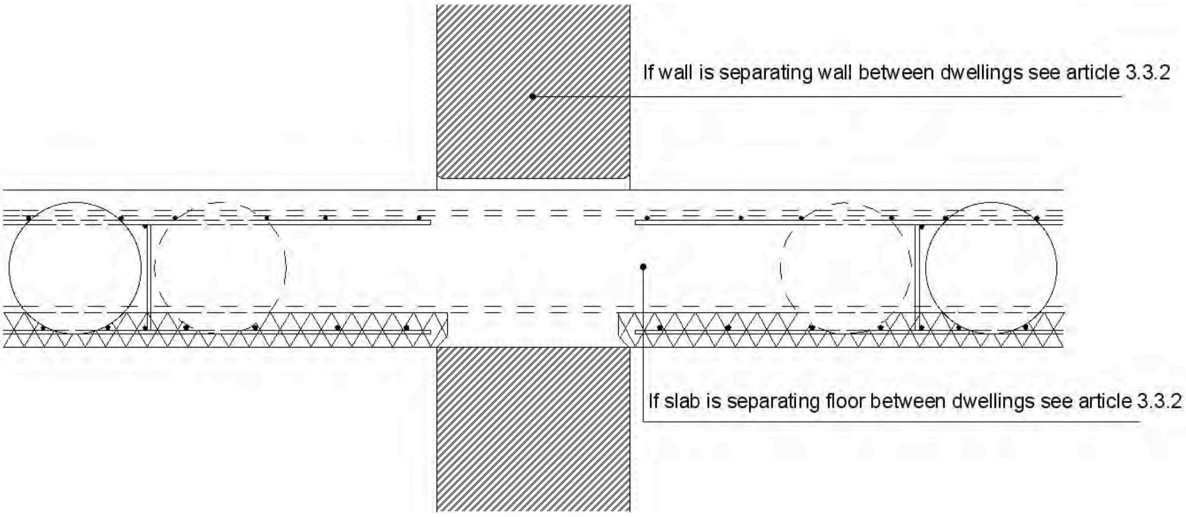
Detail 1



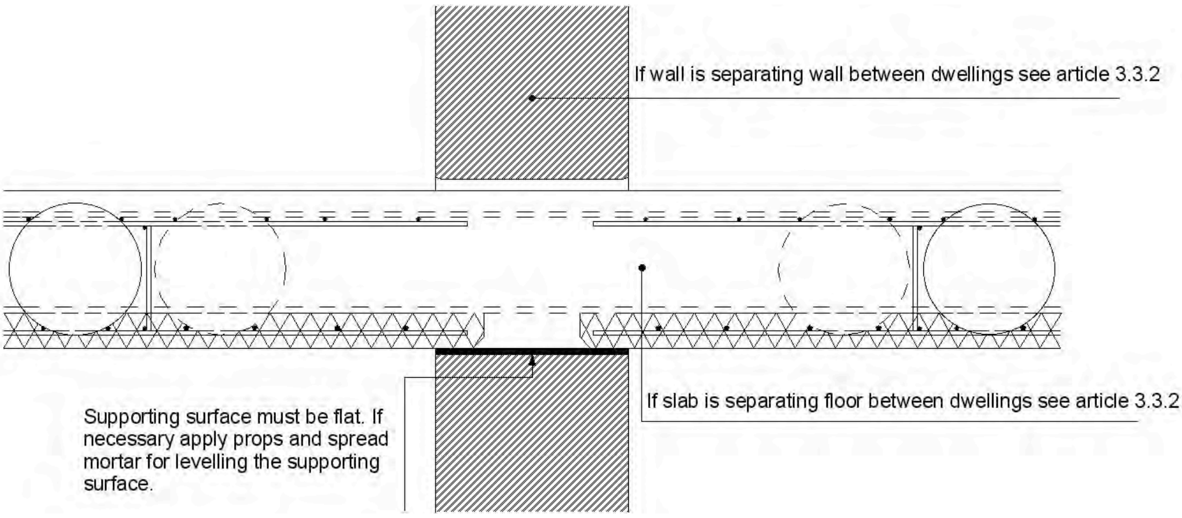
Detail 1a


Principle form and dimension cross
strength reinforcement

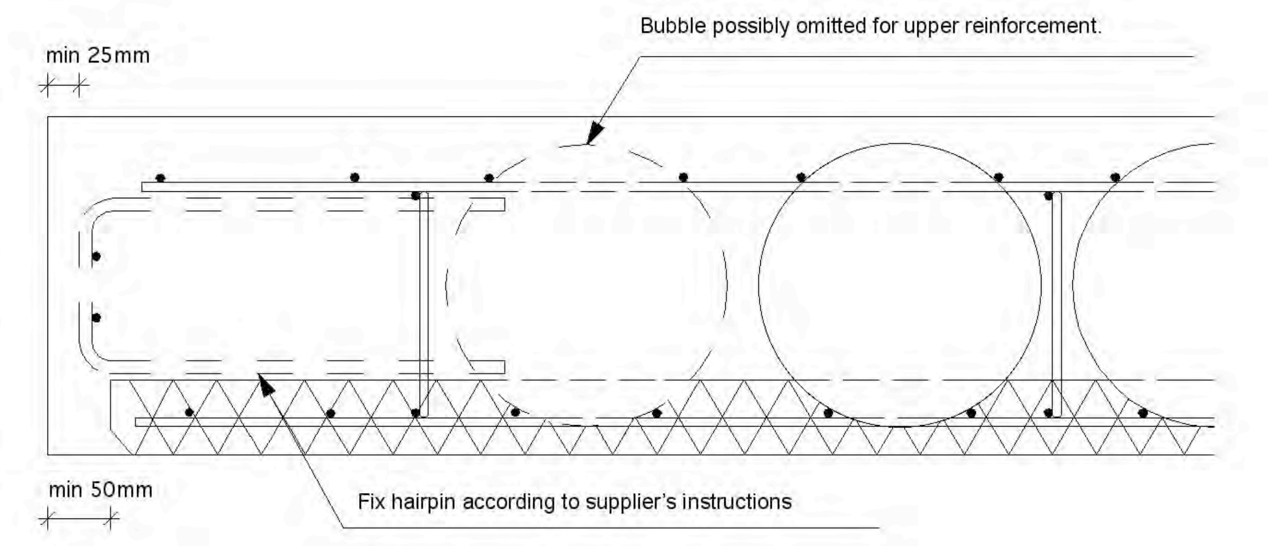
Detail 1b



Detail 2

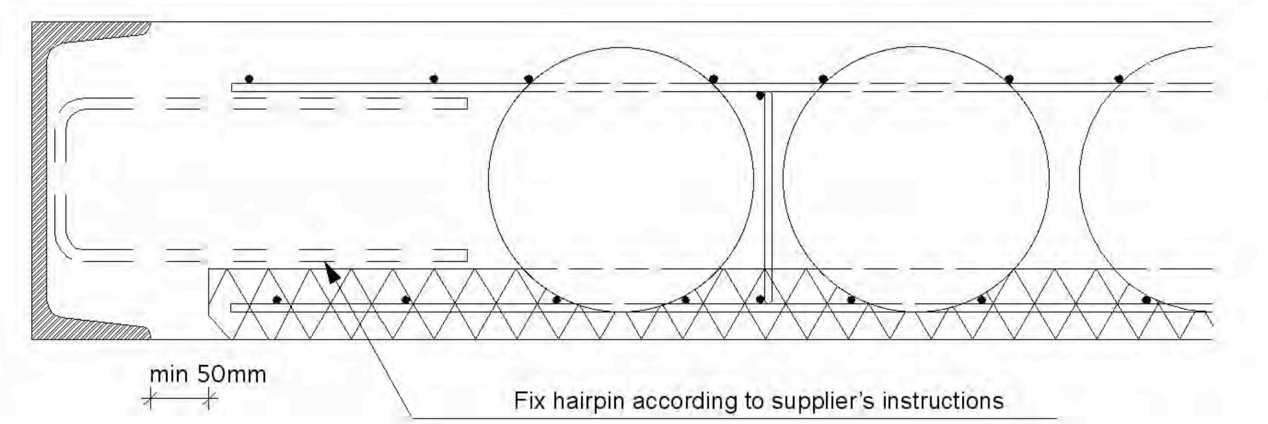


Detail 2a



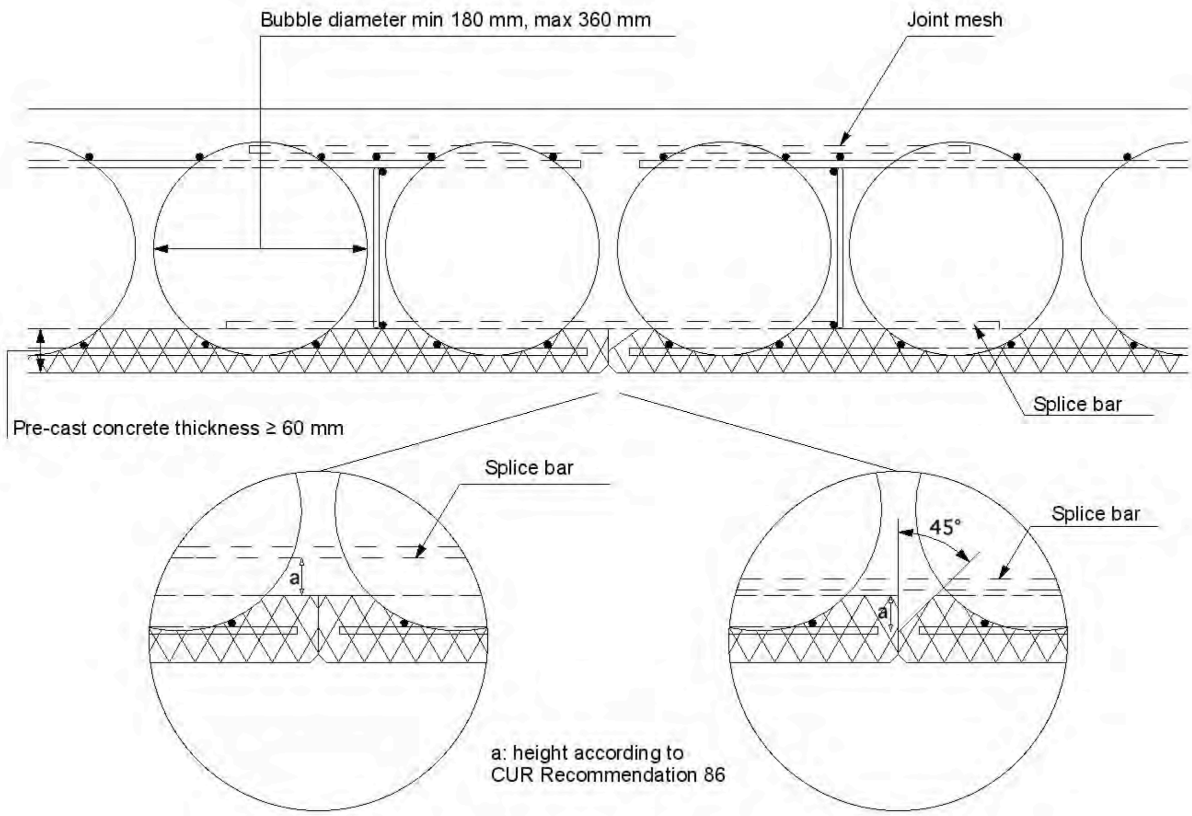
Detail 3

Floor edge



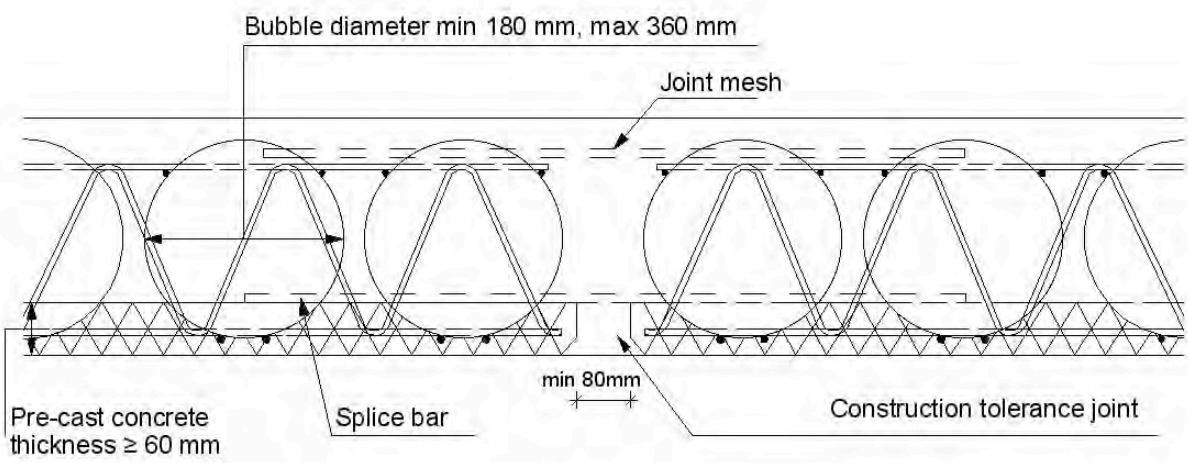
Detail 3a

Floor edge trimmed with steel profile



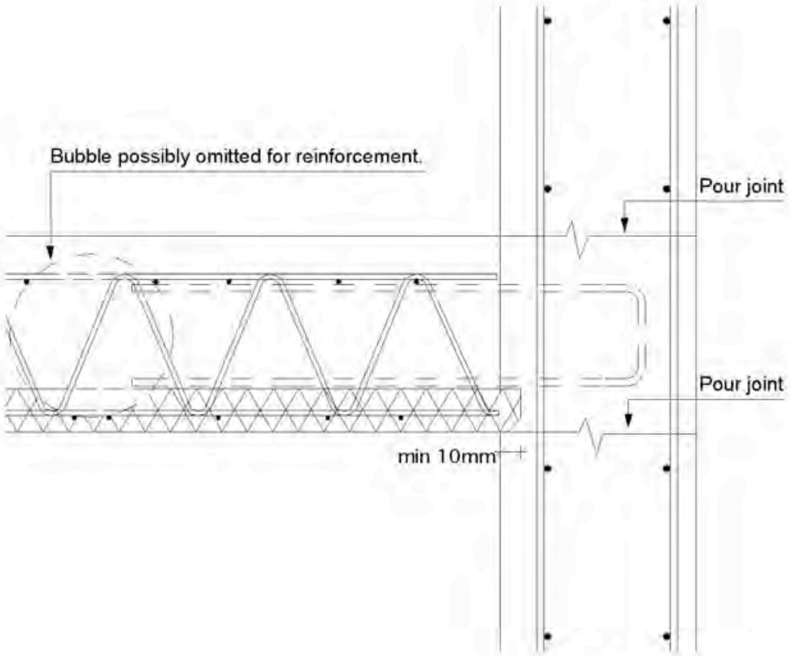
Detail 4

Detail of longitudinal joint between elements according to CUR Recommendation 86



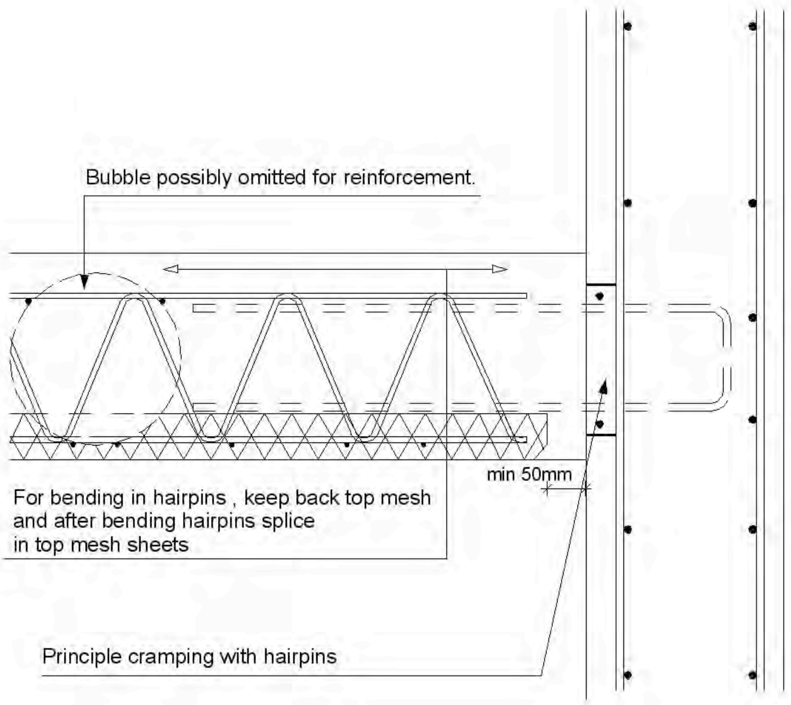
Detail 5

Section through site tolerance joint



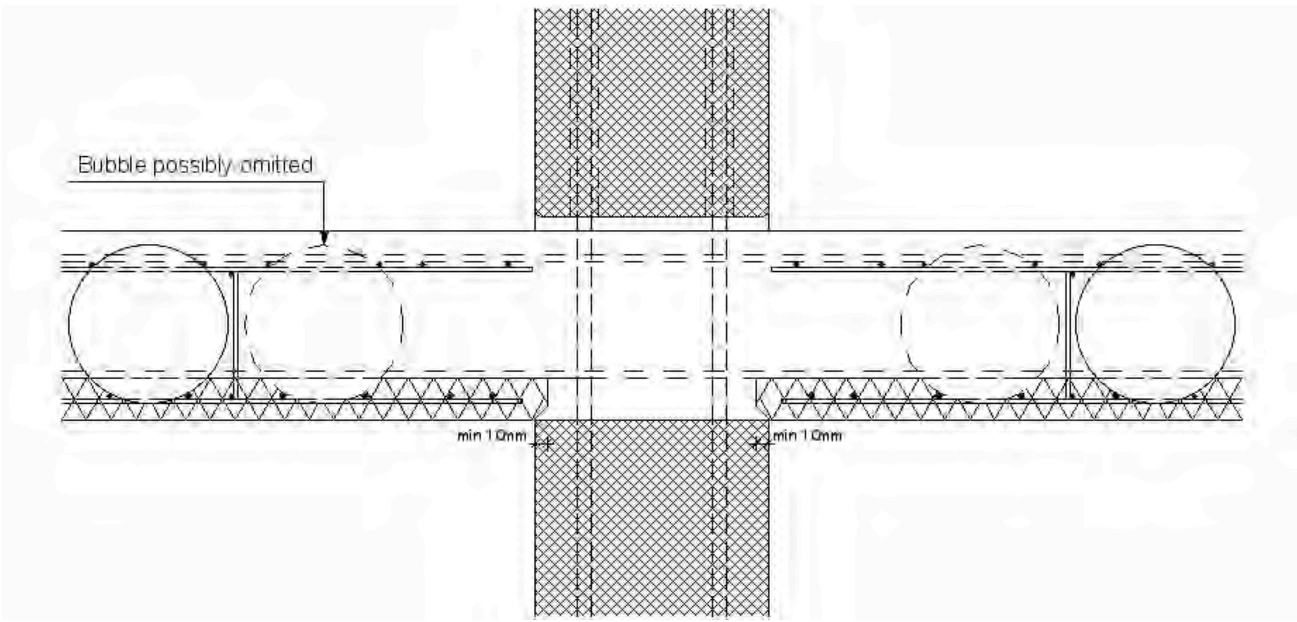
Detail 6

Concrete core wall connection



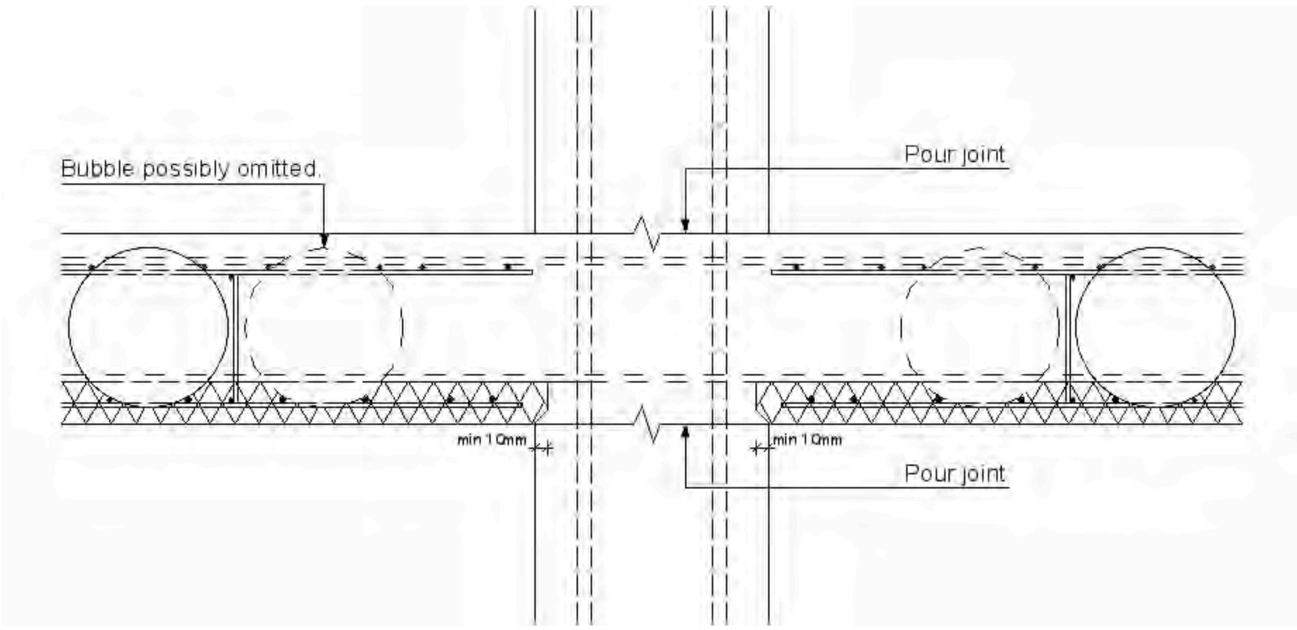
Detail 6a

Concrete core wall connection



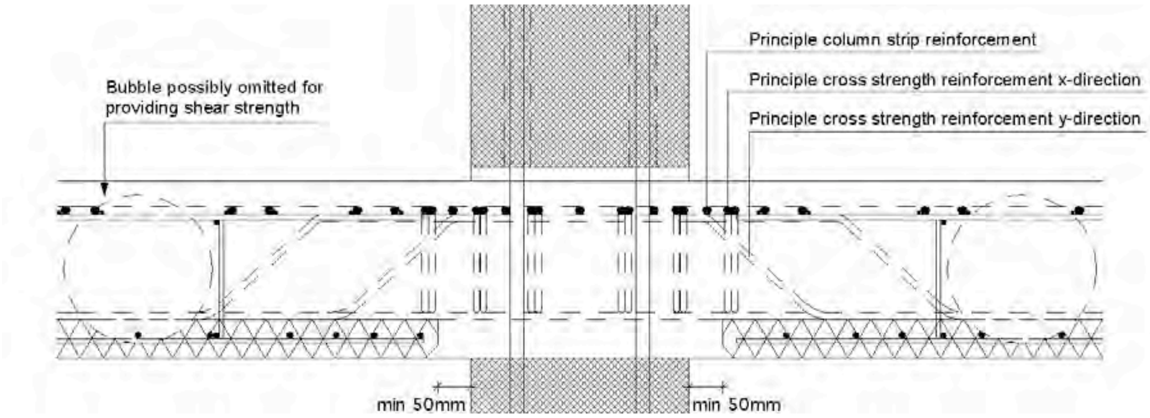
Detail 7

Connection to pre-cast concrete wall



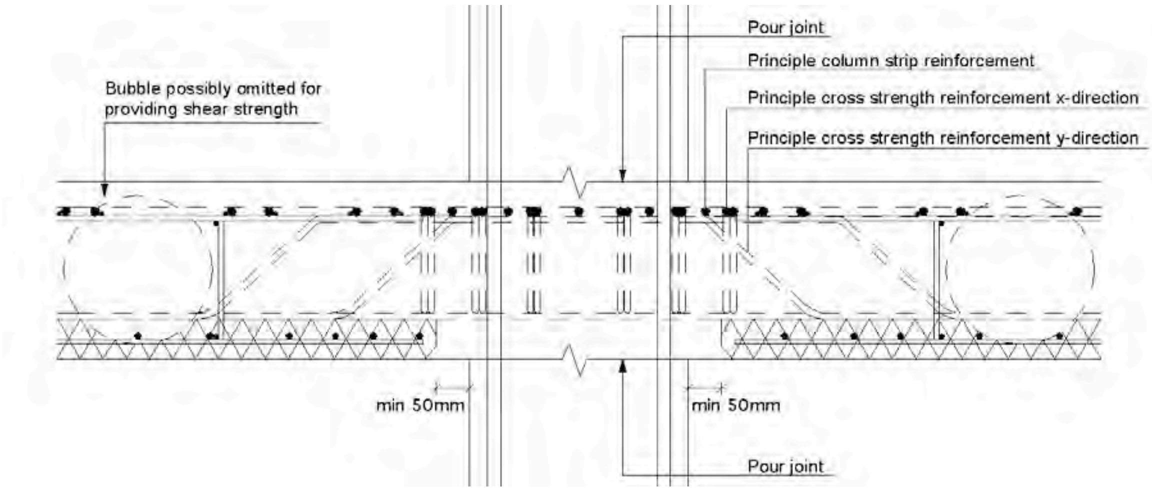
Detail 7a

Connection to in-situ concrete wall.



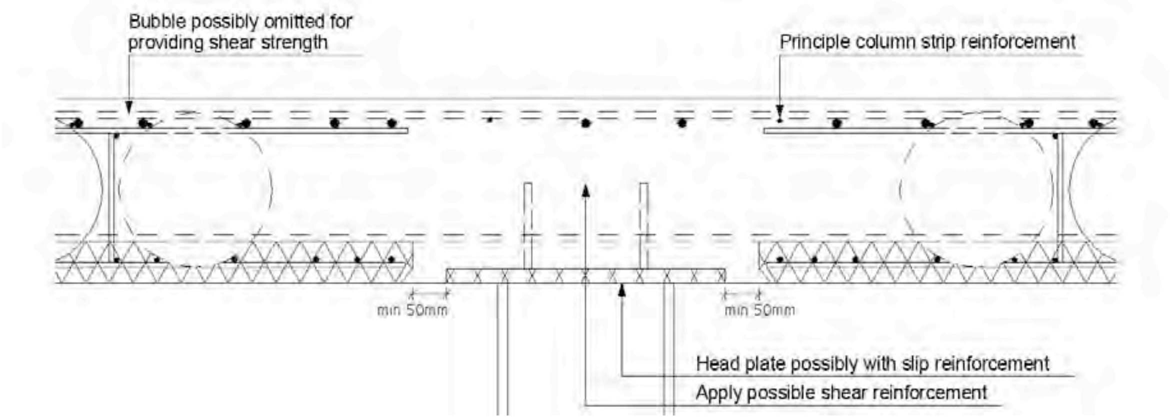
Detail 8

Connection to pre-cast concrete column



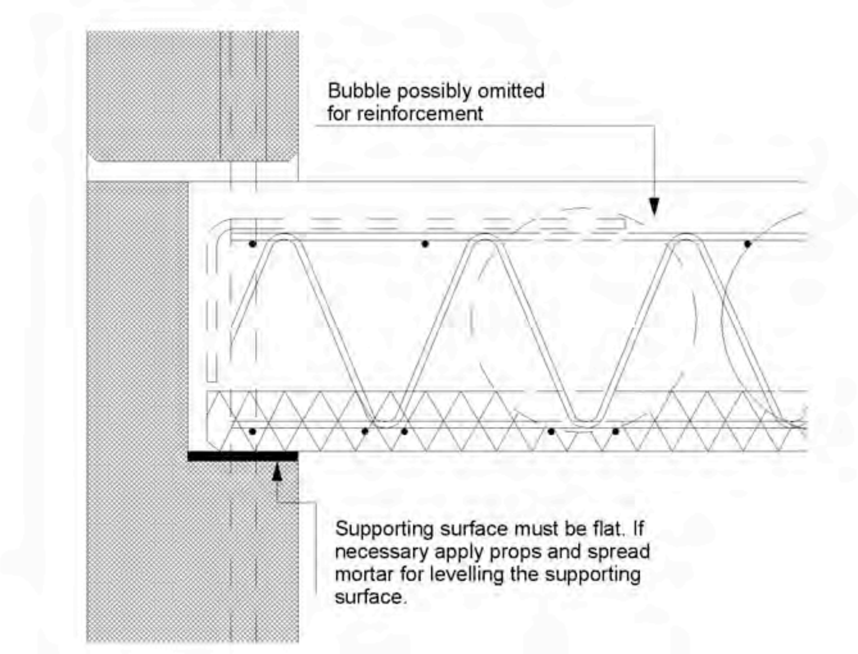
Detail 8a

Connection to in-situ concrete column



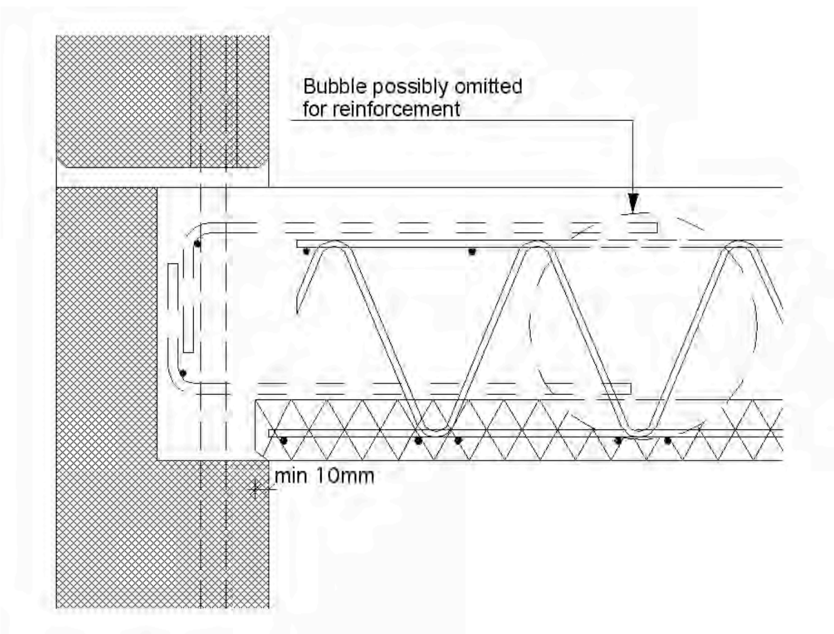
Detail 8b

Connection to steel column



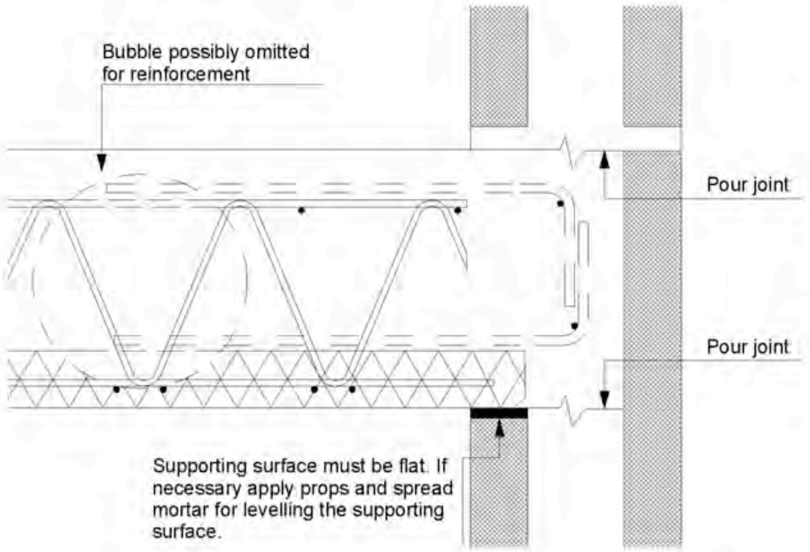
Detail 9

Connection to pre-cast concrete wall



Detail 9a

Connection to pre-cast concrete wall



Detail 10

Connection to concrete cellar wall consisting of pre-fabricated concrete inner and outer skin

Centre for Civil Engineering Research and Codes (CUR)

Recommendation 86

BubbleDeck floors®

Supplementary regulations to NEN 6720:1995 (VBC 1995)*

*) Dutch standard NEN 6720: Technical principles for building constructions TGB 1990
Regulations for concrete: Structural requirements and calculation methods (VBC 1995)

New types of concrete constructions are regularly designed and realized for which no structural requirements still exists. This is an inevitable result of innovation. While there are no requirements, rules are being set for each project by the interested parties. It is of course undesirable that this situation continues for a long time. CUR publishes CUR-Recommendations to fill this void in structural requirements.

Subject of this CUR-Recommendation are concrete floors in which hollow spheres are applicated to reduce the weight of the concrete floor.

This floor type is known under the name "BubbleDeck floor®".
At the opinion of CUR-Regulations Committee 58 "BubbleDeck floors®" in practice enough experience is achieved and information is available to draw up regulations for this type of concrete floor. This information consists mainly of test results and theoretical considerations. The Committee has carried out experimental research on shear force capacity. The information is not yet such that a uniform answer is available for all the questions. For this reason extra margins were built in design values. Also the scope has been restricted to floors with a total thickness of 230 mm up to and including 450 mm.

This CUR-Recommendation is drawn up by CUR-Regulations Committee 58 "BubbleDeck floors®". At the moment of publication the composition of the Committee was as follows:
H. Ouwerkerk (chairman), D.J. Kluft, ir J.A. Bunkers, P. de Jong (secretary/reporter),
Prof. C.S. Kleinman, Prof. Dr. J.C. Walraven, J. de Wit, M. van Iperen,
C.A.J. Sterken, F.H. Middelkoop, R. Plug, J.M.H.J. Smit (co-ordinator) and Th. Monnier (mentor).

The Committee thanks ing. M.J.A. van Niekerken who up to 1 January 2001 was the co-ordinator of the Committee.

The Recommendation has been approved by the General Regulations Committee "Concrete" and is supported by NEN/CUR-Committee 35100109/VC20 "TGB Concrete Constructions".

CUR Recommendations 86

Table of contents

1. Subject and scope of application

1.1 Subject

This CUR-Recommendation contains supplementary regulations and requirements to NEN 6720: Technical principles for building structures TGB 1990 – Regulations for Concrete – Structural requirements and calculation methods (VBC 1995) for BubbleDeck floors®. The supplementary CUR-Recommendation 37 "High strength concrete" and also CUR-Recommendation 39 "Concrete with rough light aggregates" to NEN 6720 are not applicable.

Where in this Recommendation partially supplementary and/or deviating regulations are given to NEN 6720, the rest of each of these articles shall remain valid as it is. Where no supplementary and/or deviating regulations relative to certain parts of NEN 6720 are given, NEN 6720 shall remain valid in full. Where in NEN 6720 reference is made to another article in NEN 6720, the latter being supplemented and/or amended in this Recommendation, in case of reference the statement in the supplemented and/or amended article applies.

The standards NEN 5950: Regulations for concrete technology (VBT 1995): Requirements, production and inspection, and NEN 6722: Regulations for concrete: Construction (VBU 1998) are applicable.

1.2 Scope of application

This CUR-Recommendation applies to predominantly statically loaded BubbleDeck floors®, with a floor thickness from 230 mm up to and including 450 mm, fitted with reinforcement and not prestressed.

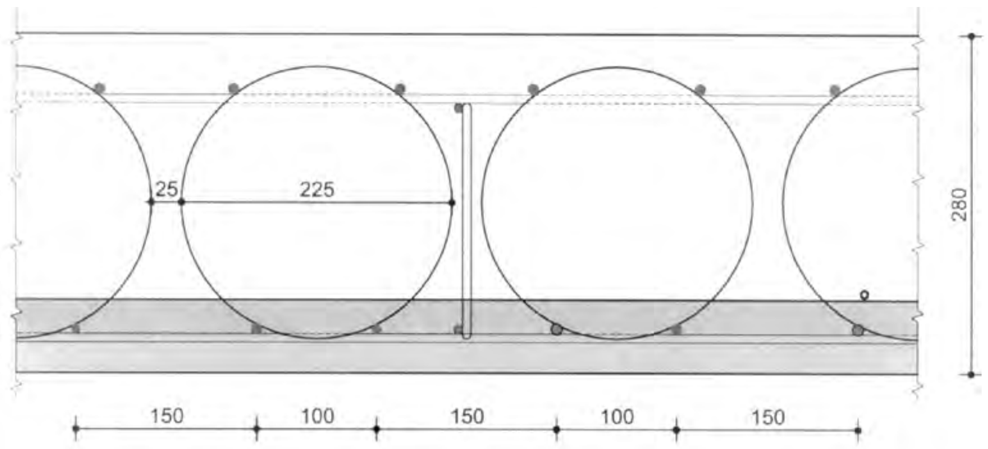
2. Terms and definitions

BubbleDeck floors®

A BubbleDeck floor® is a flat concrete slab composed of pre-fabricated reinforcement modules where between the lower and the upper reinforcement weight-saving hollow spherical elements are clamped. The pre-fabricated reinforcement modules can on the bottom be fitted with a concrete filigran slab or applied directly on the mould of the concrete floor. The reinforcement modules are connected by reinforcement bars and nets. The floor is poured on site. It is also possible to have a fully pre-fabricated BubbleDeck floor®.

Explanatory notes

Figure 1 is an example of a cross section of a BubbleDeck floor®.



BV-girder

Single leg girder for BubbleDeck floors®.

Gross concrete cross section

Area of the cross section without taking into consideration the presence of the spheres.

Net concrete cross section

Vertical cross section in the area of the middle of the spheres.

3. Units and quantities

The units and quantities used correspond with those in NEN 6720.

4. Requirements and determination methods

No supplementary regulations.

5. General conditions

No supplementary regulations.

6. Material properties

6.1 Concrete

6.1.5 Creep coefficient

For the determination of the creep coefficient and the shrinkage apply 2/3 of the fictive thickness of the gross concrete cross section as the fictitious thickness hm.

7. Schematization and distribution of sectional forces and moments

7.1 Schematization

7.1.4.2 Proportion of flexural stiffnesses

The calculated flexural stiffnesses EIvx and EIvy in this clause should be multiplied by the factor 0.8.

7.2 Theories

7.2.3 Principles of quasi-linear theory of elasticity

Method b2 can also be applied for BubbleDeck floors®. The value of I should be taken equal to the square area moment of the non-cracked gross concrete cross section. For the determination of the fictive modulus of elasticity Ef, the reinforcing percentage should be applied on the gross concrete cross section, so without taking the presence of spheres into consideration. The values of Ef following table 15 are not valid for flexure in combination with normal force. The values for flexure in combination with normal force should be multiplied by the factor 0.9.

8. Dimensioning and assessment

8.2 Shear force

8.2.1 Verification criterion

The value of t2 should be multiplied by the factor 2/3.

8.2.2 Design value of the shear resistance

In the formula for td the value of b should be 0.3 x the width of the gross concrete cross section.

8.2.3 Ultimate shear resistance
8.2.3.2 Cross sections loaded in flexure and normal force
For the determination of s'bmd the gross concrete cross section should be applied. The calculation of sbmd should be based on the net concrete cross section.

8.2.3.3 Cross sections with limited flexural tensile resistance
For the determination of s'bmd the gross concrete cross section should be applied.

8.2.4 Shear stress to be resisted by the reinforcement
Of the two diagonal legs of the BV-girder that form a triangle together, only one diagonal should be applied as shear force reinforcement. Of this diagonal only 75% of the cross section should be applied.
In the formula for ts the internal lever arm z should be maximized to the centre-to-centre distance of the upper and lower bar of the BV-girder. For the value of q the requirements in NEN 6720 are valid, provided that for q no bigger value than 45° should be applied.

Explanatory notes
The requirement in 8.2.4 of NEN 6720 that at least 50% of the total shear force reinforcement should be consist of stirrups if td is bigger than 2t1, is also valid for BubbleDeck floors®. Due to complaints about the practicality of applying stirrups it is generally recommended to omit the spheres in areas where td is bigger than 2t1, so that for these areas the requirements for shear stress resistance according to NEN 6720 are valid.

8.2.5 Shear joint surfaces of composite girders and slabs
For Bubble Deck floors® where pre-fabricated filigran slabs are applied, ks = 0.8 and kb = 0.3. If the shear force reinforcement consists of BV-girders the angle between the diagonals of the BV-girders and the shear joint surface of the filigran slab should be taken into consideration when determining tu. Asv in the formula tu should be multiplied by the factor (sin a + cos a), where a is the angle between the shear joint surface and the reinforcement bars that cross the shear joint surface. Of the two diagonal legs of the BV-girder that form a triangle together, only one leg can be taken into consideration.
On the surface of the shear joint surface the part that is taken by the spheres should be deducted.

8.2.6 Plates subjected to in-plane loading
This clause is not valid.

Explanatory notes
The clause concerned deals with the elementary calculation of plates subjected to in-plane loading by normal and shear forces. The fact that this clause is not applicable does not mean that BubbleDeck floors® cannot resist in-plane loading.

8.3 Punching shear
8.3.1 Verification criterion
In the formula for tu the contribution of ts should not be applied.
The value of t2 should to be multiplied by the factor 2/3.

Explanatory notes
There are no test results available to prove that the influence of punch reinforcement in BubbleDeck floors® could be determined in the same way as for solid floors. The contribution of punch reinforcement can therefore be left out of consideration. This is not a problem because punch reinforcement is impractical and anyway by leaving some spheres out the punch criterion can be satisfied. In this case the requirements for punch reinforcement according to NEN 6720 are valid for the area concerned.

8.3.2 Design value of the shear resistance

The values of the perimeter p from the formulaes should be multiplied by the factor 0.25.

Explanatory notes

The given factor is applicable for BubbleDeck floors® where there are spheres inside the periphery. It is generally recommended to exclude the spheres out of this periphery to achieve a locally solid floor where the requirements for punch according to NEN 6720 are valid.

8.3.3 Ultimate shear resistance

For the determination of the reinforcement percentages wox and woy the gross concrete cross section should be applied.

8.3.4 Shear stress to be resisted by the reinforcement

This clause is not valid.

8.6 Deflection

8.6.1 Verification criterion

Change the text by:

The calculation of the deflection should be carried out according to 8.6.2.

8.6.2 General verification

The given calculation method is applicable for BubbleDeck floors® under the condition that the values of the crack moments Mr and Mrt of the gross concrete cross section are multiplied by the factor 0.8. This reduction factor is not valid for the bends, so that these should be determined by the crack moments of the gross concrete cross section. In determining the bends it is allowed to consider the floor without spheres.

Explanatory notes

Figure 2 shows how to determine the M-_-diagrams.

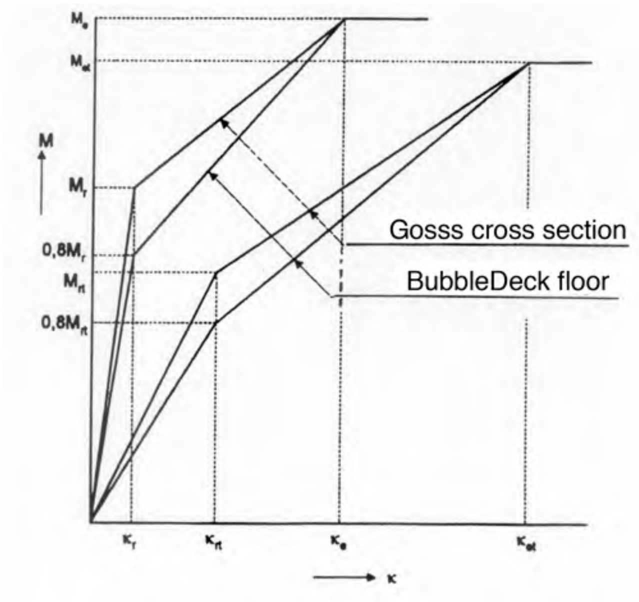


Figure 2 Determination of the M-_-diagrams of BubbleDeck floors®.

8.6.3. Flexural stiffness

This clause is not valid.

8.7 Cracking

For the determining of sb according to 8.7.1, ssr according to 8.7.3 and sb according to

8.7.4 of NEN 6720, the section factor should be equal to 75% of that of the gross concrete cross section.

9. Detailing

9.1 Minimum dimensions

Add to the clause:

9.1.7 BubbleDeck floors®

The minimum thickness of the concrete above, underneath and between the spheres should be at least 1/9 of the diameter of the spheres.

9.3 Fire resistance in relation to collapse

The requirements and the explanatory notes of 9.3 of NEN 6720 are not valid and have to be replaced by a complete new clause 9.3 with the same title: "Fire resistance in relation to collapse" and the following requirement text:
The fire resistance in relation to collapse of construction parts should be determined arithmetical according to chapter 5 of NEN 6071, or should be determined experimentally according to chapter 3 of NEN 6069.

Explanatory notes

Above adaptation on 9.3 will be included in the adaptation leaflet A2 of NEN 6720 to be published.

Add to this clause:

9.3.1 Arithmetical determination of the fire resistance

In the arithmetical determination of the fire resistance according to chapter 5 of NEN 6071 a distinction should be made for the temperature of the reinforcement bars next to the spheres and bars situated in the solid part. For bars in the solid part apply figure 12 of NEN 6071. For bars next to the spheres the temperature should be determined with figure 3. The correction factors belonging to these temperatures for the tensile strength of the reinforcement steel should be determined according to 8.2.1 of NEN 6071. For the total of the reinforcement, the tensile strength should be equal to the calculated average dependent on the determined design values of the tensile strengths and the amount of steel of the bars next to the spheres and those situated in the solid part.

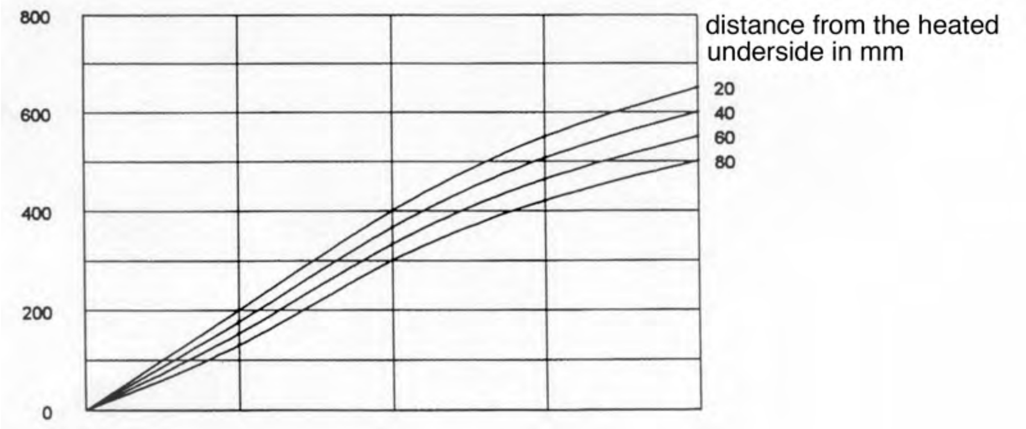


Figure 3 Temperature of the reinforcement next to the spheres in function of the fire time with different reinforcement distances
(Note: the reinforcement distance according to NEN 6720 is the distance from the surface of the concrete which is exposed to fire to the centre of the considered reinforcement bar).

Explanatory notes
Example: with a reinforcement distance of 40 mm, the temperature in the bars in the solid part after 90 minutes equals 440°C according to figure 12 of NEN 6071 and equals 500°C for the bars next to the spheres according to figure 3 of this CUR-Recommendation. The corresponding values for the tensile strength are respectively 344 N/mm2 and 278 N/mm2 according to figure 7 of NEN 6071. If 40% of the total reinforcement exists of bars next to the spheres than the calculated average equals 0.4 x 278 + 0.6 x 344 = 318 N/mm2.

9.3.2 Determination of the required reinforcement distance

If the reinforcement distances in table 1 are respected, then the arithmetical determination of the fire resistance is not necessary, except for the requirement in 9.3.3.

Table 1 Determination of the reinforcement distance in relation with the fire resistance.

Required fire resistance in minute	Reinforcement distance for bars in the solid part in mm	Reinforcement distance for bars next to spheres in mm
30	10	10
60	20	20
90	30	50
120	40	80

When determining the necessary reinforcement distance apply the calculated average of both reinforcement distances.

In seams between concrete filigran slabs, the slab thickness where the slabs lie directly against each other, should not be included in the determination of the reinforcement distance.

Explanatory notes
Example: for a required fire resistance of 120 minutes, the required reinforcement distance for bars in the solid part is 40 mm and for bars next to the spheres is 80 mm.

If 40% of the total reinforcement exists of bars next to the spheres, then the calculated average equals 0.4 x 80 + 0.6 x 40 = 56 mm. This is the required reinforcement distance of all bars if they were all situated on one level.

Application of large reinforcement distances can be difficult in practice. The reinforcement distances can be reduced by arithmetical determination according 9.3.1 of this Recommendation. In general this calculation will require in some areas of the floor more reinforcement than the reinforcement necessary at normal temperature. Research has proven that this 'overdimensioning' in general will be limited. In verification the method of redistribution of flexural moments can be used. In this case, it shows that 'overdimensioning' is even more limited or is not necessary at all.

In seams between concrete filigran slabs, it is assumed that the air can flow freely between the slabs, so that extra measurements are necessary to get acceptable reinforcement distances. The joining bars can be fixed on the required cover above the seam. Another possibility is the application of a triangle edge of the slab in which concrete is poured. Figure 4 shows both possibilities.

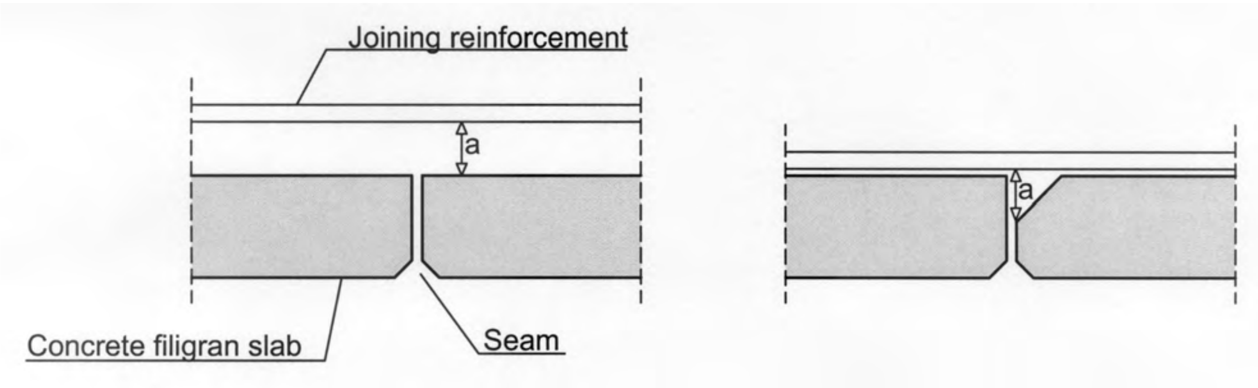


Figure 4 Reinforcement distances for joining bars.

9.3.3 Verification of the shear force resistance

If a fire resistance in relation to collapse of 60 minutes or more is required for the floor, a verification calculation should be executed on shear force resistance, under fire conditions according to NEN 6071. The contribution to the shear force resistance of the concrete should be applied to 50% of the value of t1 according 8.2.3.3 of NEN 6720.

Explanatory notes
In contrast to the verification of shear force resistance under normal circumstances, in case of fire possible collapse due to exceeding of the shear force tensile resistance has to be assumed. For BubbleDeck floors only 50% of this resistance should be taken into consideration.

9.6 Anchorage length of reinforcing bars

Add to this clause:

9.6.4 Anchorage of reinforcing bars in BubbleDeck floors®

The required anchorage length according to 9.6.1 up to and including 9.6.3 of NEN 6720 has to be increased by 10% for every sphere in contact with the bar in the anchoring area.

9.9 Minimum reinforcement

9.9.2.1 Minimum reinforcement

The section factor W should be determined for the net concrete cross section.

9.11 Detailing of the reinforcement

9.11.1 Plates

Add to this clause:

9.11.1.7 BV-girder

BV-girders that are applied as shear force reinforcement, have to meet the following requirements:

∑ • the centre-to-centre-distance of the BV-girder should be a maximum of twice the floor thickness;

∑ • the centre-to-centre-distance of two down running or up running diagonals should be a maximum of 2/3 of the floor thickness;

∑ • every pair of two diagonals, existing of one up running and one down running bar, should be welded with 2 welding points to both the lower and the upper longitudinal bar;

• the welding points with which the diagonals are attached to the lower and upper longitudinal bar should have a resistance per welding point of at least 25% of the flow strength of the diagonal;

∑ • the bend radius of bent bars should be minimal 2.5k according to 9.5 of NEN 6720;

∑ • the distance between the edge of the floor support and the connection of the first diagonal from the floor support with the upper longitudinal bar of the BV-girder should be maximal _ times the height of the BV-girder.

Titles of cited standards and CUR-Recommendations

NEN 5950: 1995
Regulations for concrete Technology (VBT 1995). Requirements, production and inspection, including amendment sheet A2: 1999

NEN 6069: 1991
Experimental determination of the fire resistance of building parts with correction sheet of March 1992

NEN 6071: 1991
Arithmetical determination of the fire resistance of building parts. Concrete constructions, including amendment sheet A1: 1997

NEN 6720: 1995
Technical principles for building constructions TGB 1990
Regulations for concrete: Structural requirements and calculation methods (VBC 1995), including amendment sheet A2: 2001 (in preparation)

NEN 6722: 1989
Regulations Concrete. Construction (VBU 1988), with correction sheet May 1989

CUR- Recommendation 37
High Strength Concrete. Additional requirements on NEN 6720 (VBC 1990), NEN 5950 (VBT 1986) and NEN 6722 (VBU 1988)

CUR- Recommendation 39
Concrete with rough light aggregates. Additional requitements on NEN 6720 (VBC 1990), NEN 5950 (VBT 1986) and NEN 6722 (VBU 1988).

Dutch standards are publications of the Dutch Standardization Institute (NEN), Vlinderweg 6, Postbus 5059, 2600 GB Delft (orders with NEN, sales and information department, Tel.nr.: 0031 15 269 03 91).

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Gouda, December 2001
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