The BCE-system, prestressed hybrids of AAC and HPC

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Abstract: The BCE technology opens up a new method to reinforce Autoclaved Aerated Concrete (AAC) by separating the production into two stages, primarily a block production from a traditional plant and secondarily the combining of blocks with affusion of High Performing Concrete (HPC) into prestressed hybrid panels, vertical or horizontal. The first stage means a continuous output of a limited number of standardized format blocks, up to 300 mm height, the second stage a non-standard production of individual panels following a CAD/CAM managed procedure, giving full freedom to architectural variation. The second stage can be added to existing block producing facilities, even in cases when a genuine PFA recipe is applied. The BCE horizontal panels have a higher capacity of up to 9 m span, carrying a live load of up to 6 kN/m², than those with unstressed reinforcement from a traditional AAC plant. Industrial wall panels as well as roof panels may be 12 m, the latter under limited snow loads as is the case on most locations. Vertical wall panels can be designed with reduced depth but with retained capacity. In comparison with existing concrete alternatives the BCE panels are carrying a lower embodied energy, generating lower emissions in production of carbon dioxide.


Preface

The BCE technology is based on experience with a complete building system of AAC (Autoclaved Aerated Concrete), the Siporex alternative, with which I was involved in the 1970s. The Swedish market for this material was declining at the time due to a rather formal attitude from the national authorities on heat losses through walls and structures towards sky and ground. It was only later that a full understanding of sources of loss, such as through ventilation and windows, was recognized. So, the value of AAC was underestimated at the time which is now compensated for by growing imports of the material from the continent. But there was something objectionable also about the reinforced products – they were rather complicated to produce, limited to 3M-standard, and stored awaiting customer’s order. Only from Japan did we learn that no such storage was allowed, due to the cost of land. This seemed futuristic but was in fact a proof of sound economy. Was there a way to approach this attitude by reforming the production layout? The BCE technology was first tried in the 1980s, then published at the 3rd Int. Conference on AAC (Zürich, 1992) and again at the 5th Conference (Bydgoszcz, 2011). Throughout this long period I have received a most stimulating support from my colleague and dear friend Bo R Schmidt, once marketing manager at Siporex. A helping hand over the last period belongs to Hamid Bagheri, whose licentiate thesis on BCE from 2006, available on Internet (Prestressed hybrids of AAC and HPC – The BCE building system), is attracting growing interest.

Bo G Hellers
1. General position of AAC

AAC (Autoclaved Aerated Concrete) is a man-made material of many virtues, insulating, structural, fireproof and moist reactive, now produced and spread around the globe. A recent estimate by van Boggelen (1) of the production capacity indicates 450 Mm$^3$/an (million cubic meters per year) of blocks, to which should be added another 100 Mm$^3$/an or so (personal estimate, based on less than 20%) of reinforced panels. More than half of the capacity is located in China’s 2,000+ production facilities, according to Kansagra (2). Assuming a two/third’s occupancy for attaining necessary profitability would imply a total production of some 350 Mm$^3$/an (the present decline in Chinese housing production may reduce this estimate to 250 Mm$^3$/an). The value is around USD 70/m$^3$ or 25 BUSD/an (billion US dollars per year), or in Swedish money up to 200 BSEK/an, an impressive figure. It is worth observing that Europe, which has a stable market at 20 Mm$^3$/an, not counting Russia, which has a market of similar size, Levchenko (3), now accounts for less than 10% of the global AAC production, a radical change from the 1990s, when Europe was still dominating. Major markets in Europe are Poland, Germany and neighbouring Russia, whereas Scandinavia is slumping with no remaining production but with growing imports from the continent. The focus has now shifted to Asia and the Arab world. The Americas are still behind with only three major producers but moving forward – it is anticipated that the US will experience a dramatic growth, when needed permits are in place and commercial forces are ready to move on the market opportunities. The critical properties of the material would be fire safety, on top of strength and earthquake resistance, even endurance (4) superior to heavier alternatives, beside good insulation capacity for heat and sound and reactivity for moisture, which may have a positive influence on the health of the occupants. So, in general, the global spread of AAC improves the indoor climate around the world to the benefit of mankind. No vapour barriers in enclosing structures are applied in buildings for normal use, allowing temporary moisture storage in the normal case when indoor air contains more moisture than the outdoor air, which may be released again to the indoor air by reversed diffusion in case the content goes down. The AAC material can well endure counterflows of heat and moisture. This is true also in cases when the envelope has an additional external heat insulation. Only in cases when the indoor environment contains a wet industrial process is it feasible to apply an inner vapour barrier. Also, the material must be protected from excessive carbon dioxide (cattle, chemical industries) which would hasten carbonizing and micro-cracking. Since AAC is very active in capillary suction, the material must never be in direct contact with open water.

AAC is easy to cut and shape into any suitable form using relevant tools.
(Another property of current interest is that AAC is > Class III Bullet Resistant (5), preventing ricochets in gunfights and warfare, which contributes to an overall safety in society. This property is outlined in (6) regretting that you cannot have a bullet-proof home in California, since AAC is not yet allowed in structures.)

So, the imminent success of AAC is due to the well-balanced properties of the mature material, but above all to the fact that AAC can now be produced from very simple raw materials – some 70% of the dry weight must be a source of silicium – traditionally this was silicous sand after grinding in a ball mill in which a larger area for reaction is attained. But more recently, the sand has in general been replaced with PFA (Class F Pulverized Fuel Ash), taken from the electrostatic filters after ground hard coal combustion, normally in utility plants. With PFA, taking out the useful pozzolanic fraction, milling is not needed. The total amount of ash is usually around 15% (up to 40%) of the coal weight so this source is plentiful on the many markets depending on coal for electricity generation. An estimate from EPRI (7) says that the ash production (including scrubber sludge) in the world is some 600 million tons per year. It would suffice in making at most 1,750 million m$^3$ of AAC annually, of which 350 million are now used, according to an earlier estimate, or 20%.

Taking the US as an example, the amount of PFA produced is more than 100 Mtons/an (other sources claim an even higher figure) of which more than 60% is placed in idle landfills. So there are at least 40 Mtons/an available for AAC production, which could amount to 120 Mm$^3$/an of AAC. In India, which is now a fast growing market for AAC with more than 60 production facilities currently in operation, it is compulsory for new plants to use PFA. Only three of the existing plants, one of them from 1972 (Siporex), when the author of these lines was responsible for development, produce reinforced material, using a required sand recipe.

The use of PFA for AAC is based on an American patent from 1931, according to Neufeld (8). This is remarkably soon after the start of AAC production at Yxhult, Sweden in 1929, using ground sandstone. A practical PFA recipe for making AAC was developed in the 1950’s, based on a new patent, by John Laing & Son (1948), by the English company Thermalite, starting production in 1951, sharing its skills eventually with Celcon Inc., now part of the Danish H+H group. PFA does not only serve as a source of silicium but it also renders a pozzolanic contribution to the binding of the mineral compound, partly replacing PC, Harris (9).

Economically, the transition to PFA is most favourable – the cost of sand, which is currently rising dramatically, Beiser (10), is replaced at best with a bonus for accepting PFA, which must otherwise be submitted to low-grade use or even waste. Are there any losses of property? Yes, one – the strength of AAC, based on PFA, does not quite match the equivalent strength with sand in the higher density region. This is of minor importance, though, since the bulk of application belongs to the lower region. On the other hand, the heat conductivity with PFA is less over the whole range of density variation.

The making of AAC from PFA is a true refinement of the input, a conversion from waste to excellent material, an invention in the true sense that the result is surprising, almost a feat of alchemy, according to Chusid (11). In contrast, ordinary concrete needs good
raw materials to become good in itself – the quality is not really raised from the basic level. The transition to AAC is maintained in a plant of considerable complexity, which is sometimes taken as argument against the process. This objection is obviated by the fact that on-site pouring of the expanded grout, which has been tried repeatedly in the past, releases a high shrinkage during long-time curing, due to the required use of a large amount of binder, and in most cases leading to a destructive crack formation. The autoclaving of AAC blocks at about 200°C is a rapid (12-13 h) curing process, whereas reinforced material requires almost double (23-24 h) according to ACCOA (12), which takes care of the shrinkage – the consequence is that all AAC products on the market, blocks, panels and lintels, are prefabricated, and appear on the building market as industry products.

The longer curing time with reinforced material is due to the sensitive integration of steel and AAC into a structural composite, especially for anchorage towards the end sections. In practice, blocks and reinforced material are combined in autoclaving and the required time is then set by the reinforced material.

Disregarding its advantages, Ytong of Germany is not considering any change of recipe – they maintain ground sand as the only silicium source. The reason is cosmetic in the sense that the colour of the AAC is changed from white with sand to greyish with PFA. This is claimed to influence the public sentiment against the material. But it does not comply with the current trend to favour recycling.

The density of AAC varies between 275 (insulating quality) and 800 kg/m³ (structural quality). A superlight quality, 115 kg/m³ (YTONG Multipor), has recently been introduced on the market, probably produced through an excitation of air into the slurry on top of the natural expansion due to the forming of hydrogen gas in the reaction between aluminum powder and lime. (The detailed technology has not been revealed – obviously, the critical stage is to keep the slurry homogenized.) This density in blocks is then embraced on both sides by a relatively weak structural material of density 340 kg/m³ (the structural minimum of the basic process). This is indeed a complex AAC product of hybrid character – complicated to produce but easy and very handy to use in masonry. The load carrying capacity of such a masonry is influenced by creep, however, which increases sharply with decreasing density, as has been explained by Bagheri (13). This condition must be considered to its full effect with all AAC products, which has not really been the case previously. Putting density 400 on the market was made without considering the creep, for which I must take personal responsibility. Only after analysis of crack formation in masonry structures has the effect of creep become obvious. With unsymmetric sections, for instance, long-term deformations must be considered – at worst they can be critical.

More than 80% of the contemporary production is related to blocks, Dubral (14). Reinforced panels and lintels are complex components, which are produced in parallel with blocks and in standardized sizes for application and storage. The number of standardized products is up to 1,000 according to manuals (15), half of which are reinforced floor and roof panels. This is a restriction, both economical and space-demanding on industry, which must keep a considerable storage volume, but also on
architecture, which must comply or have special products made to order, at a much higher cost. The fact that reinforced material still plays such a minor role in the total output of AAC indicates that there is something fundamentally wrong, or lacking, with the present technology. Also, the span with the traditional panel has been restricted by Siporex to 8 m, in practice 7.2 m, to avoid excessive deflection or the use of uneconomical amounts of steel reinforcement, especially for shear, (16). Aircrete Europe suggests the same limit (8 m). Traditional prefab technology with ordinary concrete has supplied the building industry with spans of 12 m and more.

The present paper builds on a personal experience with AAC and its shortcomings. It suggests a different approach to reinforced products by combining AAC with HPC (High Performing Concrete) under prestressing conditions, which leads to more efficient products, both for walls and floors, including roofing. Combining with other materials reduces the need for density variation – throughout this paper, a density of 500 kg/m$^3$ is preferred.

2. Combination of Structural Components, Hybrids

The combination into one product of several densities of AAC, as with YTONG Multipor, is but half a step, although advanced, towards the world of hybrids. Another contribution to this concept is the steel reinforcement, in analogy with ordinary reinforced concrete, a strong metal for which it is only with great difficulty that an interaction with weak AAC can be established, especially for end anchorage. The relationship between moduli of elasticity is in the order of 100, which indicates a serious resistance problem. A further step can include full density concrete, a bridge between AAC and steel, which can readily interact with AAC, the materials being of the same nature, Bagheri & Hellers (17). Again, the interaction with steel is a fundamental property.

Moreover, the traditional AAC industry is based on one or at most two materials, AAC and steel reinforcement, forced to interact. This combination has generated a separate culture, in conflict and competing with the prefab concrete industry, which is a three component industry, using high performing concrete and two steel qualities, prestressing strands and ordinary ribbed steel. Is there a way to build better AAC products by combining more and well established materials into complex hybrids? Yes, this question was asked already in the 1940’s! As described by Collborg (18), window lintels were made from 1941 by a prestressed concrete core enfolded by AAC! This product was first made in the prestressing environment, later integrated in the AAC-production with AAC forming around the core. Whatever happened to this product attempt has not been possible to reveal. But for sure, quite a number of these lintels still exist and it would of great value to conduct a field survey to establish their condition.

Then, let us here try three concretes and three reinforcements!
– Basic materials: concrete, AAC, sprayed concrete
– Reinforcements: ribbed steel, prestressing strands, fibers (steel, glass)

Expanding upon the two materials of traditional reinforced production, it is possible to identify feasible products of three, four and five materials. The present paper exposes the three (AAC, concrete, unstressed steel) and four (AAC, concrete, unstressed and prestressed steel) material hybrids, which have been tested and suggested for application. The five material hybrid will be exposed in a coming paper. The combination of six materials remains to be tried. It is noticeable that advanced industry products may very well contain ten or more materials. A cell-phone contains more than one hundred!

**COMBINATION OF STRUCTURE MATERIALS**

**3 concretes, 3 reinforcements**

3 MATERIALS:
- AAC BLOCK
- CONCRETE GROUT
- PRE-STRESSED REINFORCEMENT

4 MATERIALS:
- AAC BLOCK
- CONCRETE GROUT
- REINFORCEMENT
- PRE-STRESSED REINFORCEMENT

5 MATERIALS:
- AAC BLOCK
- CONCRETE GROUT
- PRE-STRESSED REINFORCEMENT
- FILAMENT WINDING C-FIBERGLASS,
- LOW pH-BINDER
3. Raw materials (PFA) – influence on the AAC-production, Conclusions

With a sand recipe the thermal shrinkage of the AAC is less than for steel, which makes it possible, even viable, to include rebar steel in the process. The steel must be corrosion protected since it is embedded in a porous matrix of a relatively low pH-value. The steel will be slightly prestressed after cooling in the autoclave process, which suppresses crack formation, at best all the way up to the service load level, Koponen & Nieminen (19). This is in full compliance with Swedish experience, related by my precious colleague Gösta Dahl, who claims that this effect contributes to the sustainability of panels.

With a PFA recipe, the opposite is true – the thermal shrinkage of the AAC material is larger than for steel, which then acts resistant to the contraction under cooling and causes cracking. So, reinforced components with a full PFA recipe is not possible, Beier (20), who refers to unsuccessful experiments by Hebel. A balance point lies close to 70/30 (PFA/sand) (A Santesson), which is unpractical in the sense that a ball mill is still needed for the sand grinding. And a mixture of different silicium sources is certainly more complicated than with just one material. The situation now is controversial and prevents the full application of AAC, based on PFA, in the building industry. It holds back the application of steel to AAC made of PFA. This is the problem once put forward in order to find a solution – how combine a PFA-based AAC with steel to form a full building system? The effect of including dead-burnt MgO to the ash-recipe in order to reduce the thermal shrinkage has not been scientifically secured (tested by Bob Talling, Turku/Finland). Is it practical to include ordinary concrete in the product? And assuming that there is a solution to this problem – along what lines will it open up the use of AAC in the building process?

4. Alternative reinforcement technology, BCE, concreting

The BCE technology, Bagheri (13), which is outlined in more detail in the present paper, offers a novel solution to the problem of reinforcing AAC of a PFA recipe by dividing the manufacturing process into two stages – the first contains only blocks in a limited number of formats, from a simple block producing unit. The second stage is
radically new in that it contains a dry assembly (no mortar) of blocks, which are then milled, prestressed and grouted with HPC (high performing concrete, ≥ K40) to unity of any structural component.

Fig 4.1
Figures 4.5 and 4.6 from Bagheri (13) showing the principle of prestressed hybrid of AAC and HPC.
One condition is the very limited tolerance of modern double wire cutting (< 1.0 mm) according to the technology by Aircrete Europe. The BCE technology offers the possibility to subdivide the marketing of AAC into blocks, for immediate delivery from storage, whereas reinforced components of any formats are made in a second stage to order. No reinforced components are normally stored and their sizes need not be standardized, which will open the use of AAC in new directions giving freedom to architectural variation. Reinforced panels of limited width carry the total load in the principal direction. Side panels help in carrying concentrated loads, or unevenly distributed loads, by tongue and groove connection. For this purpose, it helps to keep the panel width at a moderate 0.6 m. Besides, the limited strength of the AAC material is reinforced by on-site grouting of HPC in open joints between panels.

The time needed to produce a complex product must be limited, at best to the level of a corresponding autoclaving process, including loading and unloading, that is less than one day or within 24 hrs.

Dividing the production into two stages saves time in the autoclaving process, which is halved in practice and more sufficient from an energy point of view.

Prestressing needs a stable counterforce, which in this case is available by the AAC material itself – basically, no prestressing bed is required, only the metal end supports which transfer the prestressing forces onto the section of AAC, lying on a flat bed. The forces may be of the magnitude, referring to longer panels that a restraining tie is needed to prevent upward buckling. Releasing the prestressing force leads to tensile forces in the upper part of the section which needs reinforcing steel for compensation, anchored in concrete. Normally, ordinary steel is applied – only in severe environments is a stainless quality required. The dead weight counteracts this requirement but its effect should be considered in combination with other design requirements for handling of the finished panel.

It should be emphasized that the present outline of the BCE-technology is based entirely on conventional and established materials, which ensures the properties of the hybrid.

The relation between span and depth for traditional AAC panels lies in the interval 20 - 30. With BCE this is expanded to 25 - 40. This may be subdivided into 25 - 30 for floors and 30 - 40 for roof panels and industry wall panels. Vertical members have a height to width relation less than 27 (Eurocode 6). Old rules said 26 and for load-bearing walls 20 (practical value), the capacity decreasing from 12, reaching nil at 26. The depth of a hybrid wall, carrying a concrete layer, can be reduced by almost 40% without losing on load capacity.

The relation "n" between the short term moduli of elasticity for concrete and the equivalent value for AAC, going from K40 to K60 for concrete and density 400 to 600 for AAC, lies in the span 12 < n < 36 with an average of 24. For practical reasons, it is advisable to use K50 (concrete) with density 500 (AAC) as an average, which amounts to a relation between moduli of n = 20. With equivalent creep factors (\(\varphi_\infty = 1\)), this relation remains constant over time, which keeps the stress distribution stable, see Bagheri (13).
**PRODUCER’S BENEFIT**

The combined production is split up into two stages

A. Basic production of AAC blocks (6 formats), rational, mechanized, energy efficient (PFA recipe), low labour intensity (production oriented management)

B. Component production, (any combinations produced to order, within the capacity of the hybrid), level of mechanization can be fixed ad hoc (market oriented management)

C. Storage problem reduced, rational handling of blocks – readymix concrete must be added

D. Reduction of steel volumes (stressed and unstressed steels), rational steel management

E. Extending existing block plants to full or selective component production (lintels), by adding (B) in separate facility, raising the market appearance

F. Quality management throughout (especially important with site production)

G. Over all cost reduction – less sensiveness to market variation

H. BCE production requires precise performance. But the technology is available and relatively simple – tools and instruments are at hand, which is especially favourable in developing countries

**CUSTOMER’S BENEFIT**

The customer can always have an AAC solution (if more than three storeys, combined with other materials, such as steel or reinforced concrete). No combination with organic materials, omitting risk of mould or degradation. Excellent durability and combination with finishing materials on walls and floors.

A. A free choice of product, within the capacity of the hybrid

B. The capacity of the product is extended beyond previous limits

C. Length/Depth relation can go to 40, the span of roof elements to 12 m

D. Length/Depth relation at 30 can take any practical load (floor to 9 m)

E. A wall retains its capacity at reduced depth by close to 40% - the Height/Depth relation can go from 20 to 27.

F. Fairly low depth

G. Fair acoustic insulation, easily improved with floor combination

H. Excellent fire protection thanks to AAC cover >50 mm

I. Excellent possibility to enhance compatibility with reinforced grouting (seismic)

J. Lintels and beams with prestressing can have any practical capacity within a building system

K. Deformations within the service load limit are small and adjustable
5. Influence of anchorage on deformation of a panel

The prestressing force approaches towards zero at the end. The decrease is taken as linear, based on research findings, over a length of $a = 55 \varnothing$.

![Fig 5.1. Linear decline of prestressing force towards the beam ends.](image)

The shear capacity is influenced positively by the compressive stress at a given section. The ambition behind BCE is to avoid shear reinforcement altogether, partly thanks to this effect. But as the transversal force normally grows towards the end of a panel, the shear capacity diminishes. The critical point where the transversal force must be resisted lies at a distance of half the height, or depth, of the panel.

From a practical point of view the deflection of a horizontal panel under service load should be positive, that is going down (sometimes the condition is expressed slightly different, the deflection being nil as the panel is carrying half the service load including dead weight) and the angular rotation at the end being negative at the same load level. These conditions can be theoretically combined in the inequality

$$\frac{4}{48} ql^2 \leq Pe \leq \frac{5}{48} ql^2$$
It is obvious that the variation span between the limits is somewhat more than 20%. The influence of the decreasing prestress towards the ends should be included in this analysis, subtracting the end-effect from the fundamental solution of a panel under constant prestress.

\[
\frac{4}{48} q l^2 (1 - a/l)^{-1} \leq Pe \leq \frac{5}{48} q l^2 [1 - \frac{4}{3} (a/l)^2]^{-1}, \quad a = 55 \, \varnothing
\]

The deflection at the middle section is increased by considering the end effect but in practice only slightly, by around 1%, whereas the decrease of the angle rotation at the ends is approaching 10%. In practice, the conditions can be expressed in approximate terms as:

\[Pe = 0.10 \, q l^2,\]

with an allowable variation span of 0.01 \( q l^2 \) (20%). In practice, this is an upper average value between the limits of the revised formula.

\( P \) is the prestressing force and \( e \) the eccentricity under elastic conditions.

\( q \) is the sum of dead, permanent and service loads. The combination must be related to the duration of the individual load and its creep factor. The formula is used for indication – with full service load the effect surpasses its value, not necessarily indicating that the midspan deformation is positive. In an unloaded case, considering only prestressing and dead weight, the instantaneous midspan deformation is slightly negative (upwards) and this is increased with time due to creep.

Now, it is clear that the revised formula indicates that there is a solution to an equation of equality between the limit terms. The solution is \( a/l = 0.289 \), which marks an upper limit, which can also be expressed as \( \varnothing < l/190 \), indicating in practice an unrealistic value for \( \varnothing \), or the span being limited to less than 3 \( m \)1! (The BCE technology expands the practical span limit to 12 \( m \).)

At this limit, \( Pe = 0.117 \, q l^2 \), which marks an absolute maximum. This value is far above any practical level in connection with the present study.
6. **Structural design of panels (floor, roof, wall) - conditions**

In design, four kinds of loads must be considered, dead load, permanent load, live load and climate load. Two levels of limit states are important in the analysis, the failure load limit, which marks the ultimate load capacity and is decisive in choosing the prestressing force, and the service load limit, normally less than 70% of the failure load, which marks the load situation with an occurrence of perhaps a few times during the life-span of the panel. The requirement on the service load limit is that the section of a member is fully compressed – it is a proof of economical design if the service load effect level coincides with a zero stress at the bottom of the section. In practice this means that there must be a distance between the service load effect level and the load at which the first crack appears. This ensures a long life of the member, since no crack will in practice let in corrosive gases or carbon dioxide to hasten the deterioration of embedded steel and the AAC material. Cracking is of course objectionable also from an aesthetic point of view. The bending stiffness of the member remains intact as long as there is no cracking, which reduces deformation and keeps the member stiff and fit for function above the service load effect all the way up to the cracking level.

The load effect at failure is estimated from the general formula

\[ S_d = S (\gamma_n G + Y_q Q + \gamma_w W) \]

containing partial coefficients, which are nationally decided. (G is the sum of dead weight and permanent load, Q is the live load and W the wind or climate load.)

At the service load, \( S_s \), the live load is reduced by a factor \( \Psi \). Also, in connection with AAC and hybrids in which AAC is a dominant component, \( \gamma_n \) for dead weight may be changed due to the content of moisture, 1.1 at failure and 1.05 at service level.

**Horizontal panels**

The elastic opening gap at equilibrium between stones before failure is some 1.5 mm, based on the fact that the prestressing strands can reach an elastic 1% elongation. This corresponds to an angle rotation of the cracked course amounting to some 0.4°. But non-linear deformation adds considerably to this value, according to observations in connection with testing, see Section 9, where a gap of 3 mm is recorded long before failure. Again, it is clear that the technology is based on a high precision of cutting and leaving the intermediate sections between blocks dry, i.e. mortar free.

The load effects on floors at failure and service levels are estimated for comparison according to cultivated Swedish rules for AAC (15), based on (21), in accordance with EN 1991-1-1 and (22), with reference to load rates, 3.0, 3.5, 4.5, 5.5 and 7.5 kN/m², the sum of live and permanent loads. FC means first crack. The permanent load is taken as...
a constant 1.5 \( kN/m^2 \), which may be altered in a practical case under other conditions. The relation between service and rated loads is less than 0.75. For details, see the table (1).

<table>
<thead>
<tr>
<th>Load rate, kN/m(^2)</th>
<th>Service load, kN/m(^2)</th>
<th>Permanent load, kN/m(^2)</th>
<th>Live load, kN/m(^2)</th>
<th>( \Psi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td>2.0 &lt; ( Q_{FC} )</td>
<td>1.5</td>
<td>1.5</td>
<td>0.33</td>
</tr>
<tr>
<td>3.5</td>
<td>2.5 &lt; ( Q_{FC} )</td>
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<td>2.0</td>
<td>0.50</td>
</tr>
<tr>
<td>4.5</td>
<td>3.0 &lt; ( Q_{FC} )</td>
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<td>3.0</td>
<td>0.50</td>
</tr>
<tr>
<td>5.5</td>
<td>3.5 &lt; ( Q_{FC} )</td>
<td>1.5</td>
<td>4.0</td>
<td>0.50</td>
</tr>
<tr>
<td>7.5</td>
<td>4.5 &lt; ( Q_{FC} )</td>
<td>1.5</td>
<td>6.0</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Table 1. Load matrix for floor panels

Example: A. Load rate 4.5 kN/m\(^2\); \((b = 0.6 \, m)\)

\[
S_d = 1.1 \, G_{dead} + 1.0 \times 1.5 + 1.4 \times 3.0 = 1.1 \, G_{dead} + 5.7 \, kN/m^2 = 7.9 \, kN/m^2 = 4.74 \, kN/m
\]

\[
S_s = 1.05 \, G_{dead} + 3.0 \, kN/m^2 = 5.1 \, kN/m^2 = 3.06 \, kN/m (= 0.65 \, S_d)
\]

assuming that \( G_{dead} = 2.0 \, kN/m^2 \) for a reasonable span (7.5 m).

Note that \( Q/G_{dead} = 3.0/2.0 = 1.5 \), i.e. the live load capacity is 50% higher than the dead load. Also note that \( Y_q \) is taken as 1.4, a reasonable compromise between the Swedish value 1.3 and the value 1.5, given in (23).

A calculation according to Swedish rules would give

\[
S_d = 1.0 \, G_{dead} + 1.3 \, (3.0 + 1.5) = 2.0 + 5.85 = 7.85 \, kN/m^2 (= 4.71 \, kN/m) \text{ which is a little less (< 1 %) than with the suggested method. This conclusion is valid for all sorts of relations between loads and spans.}
\]

The calculation of the prestressing force, \( P \), is outlined by Hamid (13):

With reference to Fig 6.1:

\[
\sigma_c = \frac{P}{(L - 2a)} \leq f_{cc} = f_{cck}/(1.5 \, Y_n);
\]

\[
P = \frac{M_d}{\rho} < 2 \, P_{prestress};
\]
What might be added is that the force is reduced by elastic deformation after release with 8 – 10%. So, the initial level of prestressing may be chosen at 75% of the characteristic steel limit, which ensures sufficient safety, considering the requirement that the stress be less than 65% in service, Lorentsen (24). The level is lower, however, than what the French use, 80% in prestressing, with full responsibility.

The tensile stress of an uncracked end section should be checked towards a design value. The relation and inequality formula is

$$\sigma_t = 1.05 R_d \left( A_{HPC}^0 + \frac{1}{n} A_{AAC} \right)^{-1} \leq \frac{f_{ctk}}{1.5\gamma_n} = f_{ct}$$

The definition of $A_{HPC}^0$ is excluding the main part of the concrete flange. The rest of the section is active in supporting the tensile stresses, c.f. the illustration, Fig 6.2. The factor 1.05 is an estimate related to the limited depth of the double web.
Fig 6.2
Shear resisting parts of hybrid section.

The condition that the tensile stress in the concrete must be less than the design value (uncracked section) is normally not critical but should be checked in particular cases. Such cases include concentrated loads or highly uneven distribution of loads.

The design must also include shear towards the supports, normally at the ends of the panel. As the prestressing force is decreasing linearly in the zone of anchorage, it is recommended to assume a cracked section in analogy with the conditions of reinforced concrete. The critical inequality then is

\[
\tau = 1.17 R_d (A_{HPC} + \frac{1}{n} A_{AAC})^{-1} \leq f_v + 0.3 \sigma_{cm} \leq 2f_v (\rho = 0);
\]

under the constraint \(d \geq 2\Phi\).

\(f_v = \xi (1 + 50\rho) 0.30 f_{ct}\);  
\(\xi = 1.4\) when \(H \leq 0.2\ m\); \(= 1.6 - H\), when \(0.2\ m < H \leq 0.5\ m\);  
\(\rho\) is the sum of steel for prestressing and for resisting tension in the top flange.

\[
\sigma_{cm} = P_{eff} (x = h/2) (1.2 \gamma_n A)^{-1} = \alpha P_{prestress} (1.2 \gamma_n A)^{-1};
\]

\(\alpha = \frac{H/2 + 30 - 5\Phi}{55\Phi}\); \(A = A_{HPC} + \frac{1}{n} A_{AAC}\);

Continuing the previous example (A):

\(l = 7.5\ m\)

\(H = 0.3\ m; \ l/H = 25\)

\(M_d = S_d l^2 / 8 = 33.33\ kNm\); \(R_d = 17.78\ kN\)

\(P_{prestress} = 71.5\ kN\)
Take $\varnothing 9.3$ ($F_m = 96.7$ kN); $\alpha = 0.171$

The reinforcement of the top flange is $2 \varnothing 5$, #200 (NPS 500) -

the stress level under prestressing must be checked, c.f. the next section.

$t = 15$ mm ($f_{cc} = 21.5$ MPa)

d = 20 mm (> $2 \varnothing$, the decisive condition)

$\sigma_t = 1.07 < 1.36$ MPa

$\tau = 1.17 R_d/A = 0.864$ MPa

$f_v = 0.690$ MPa; $0.3 \sigma_{cm} = 0.231$ MPa; $0.864 < \Sigma = 0.921 < 2 f_v (\rho = 0) = 1.064$ MPa

Concluding, the design is OK, and the contribution to the total shear resistance of prestressing is roughly 25%.

The dead weight $G_{dead}$ is confirmed at $< 2.0$ kN/m\(^2\) (1.92).

Taking a second example B (worst case):

$l = 9.0$ m; Load rate 7.5 kN/m\(^2\)

$H = 0.3$ m; $l/H = 30$ (b = 0.6 m)

Assuming $G_{dead} = 2.5$ kN/m\(^2\);

$S_d = 1.1 G_{dead} + 1.0 \times 1.5 + 1.4 \times 6.0 = 12.65$ kN/m\(^2\) (= 7.59 kN/m)

$S_s = 1.05 G_{dead} + 4.5 = 7.13$ kN/m\(^2\) (= 4.28 kN/m) (= 0.56 $S_d$);

$M_d = 7.59 \times 9^2/8 = 76.85$ kNm; $R_d = 7.59 \times 9/2 = 34.16$ kN;

$P_{prestress} = 178.1$ kN; Take $\varnothing 15.3$ ($F_m = 260$ kN);

t = 38 mm; $d = 39$ mm; $\sigma_t = 1.35 < 1.36$ MPa;

$\tau = 0.992$ MPa; $\alpha = 0.123$; Try $3 \varnothing 6$ #150 (stress conditions must be checked);

$f_v = 0.773$ MPa; $\sigma_{cm} = 0.820$ MPa; $0.3 \sigma_{cm} = 0.246$ MPa; $\Sigma = 1.019$ MPa;

$0.992 < 1.019 < 2 f_v (\rho = 0) = 1.064$ MPa; So, the design is OK. Again, the contribution of the prestressing to the shear capacity is close to 25%.

A check of the dead load shows $2.52 \approx 2.5$ kN/m\(^2\), so this is no problem.

The relation $Q/G_{dead} = 6/2.5 = 2.4$, a figure apparently growing with the live load, in practice from 1.0 to 2.5, which is a remarkable hybrid characteristic, and certainly unlike the case with a homogeneous concrete structure, especially with unstressed reinforcement. This conclusion gives evidence to the statement that a concrete structure with unstressed reinforcement can carry half its weight, whereas a hybrid structure, like BCE, can carry double its weight and more.
The relation between geometrical parameters and load rate is shown in Fig 6.3, limiting the span at 9.0 m. The flange depth $t$ grows linearly, whereas the width of the vertical tracks, $d$, shows a more complex growth.

![Graph showing the relation between load rate and geometrical parameters $t$ and $d$.]

**Fig 6.3**
Geometrical parameters $t$ and $d$ versus load rate for 9 m span.

**VERTICAL MASONRY AND PANELS (WALLS)**
A traditional way to build a wall is by laying a masonry, using AAC blocks put together by thin bed mortar in tied connections wherever possible, requiring overlapping blocks course by course. Normally this means that the overlapping is half the block length. The relation between height and width is limited to 27 (Eurocode 6), or 26 by traditional rules. In practice with load bearing masonry, considering the attainable quality from today's masons, the limit is around 20. With a 150 mm width this means a height limit of 3 m. The capacity of such a wall is about 50% of the design value of AAC itself. In order to use a full 100% of the capacity, the relation must be limited to 12. With a height of 3 m, this means a width of 250 mm, which is a practical value for AAC blocks. Considering the imposed deformation from floor structures, the stress distribution in the wall becomes uneven, which reduces the load capacity. It goes beyond the scope of the present paper to deal with the exact calculation of stress distribution and stability conditions for walls. Experiments show that eccentricities diminish as the load goes up towards failure (25) – what is important is the fact that if the masonry wall is vertically prestressed, its capacity is increased. S Aroni (26) claims that a 30% engagement of the compressive strength for prestressing marks an optimum and makes the wall less sensitive to deflection at floor joints. An illustration, Fig 6.4, shows how prestressing rods could be penetrating each block by a 300 mm spacing, the 50 mm holes being filled finally with grout to anchor the forces.
A different approach is to manufacture hybrid wall panels, very similar to floor panels, which are prestressed at the neutral axis, again to a 30% level. Such panels have been illustrated by Bagheri (13), 2:4 and can be made very thin (120 mm) to accommodate an exterior super insulating polystyrene panel.
Fig 6.5
Section. Exterior bearing wall member with insulation
Such combination of materials renders an ideal situation for the wall, strength and yet insulating, AAC facing the in-door climate, while HPC forms a layer of high moisture penetration resistance facing the insulating panel. If the neutral axis is taken at the intersection between AAC and HPC, the moment of inertia is unchanged from an original AAC wall with a much larger depth – the 120 mm must then be 175 mm - but the load capacity is unchanged. The geometrical relations are outlined in Fig 6.6

\[ x = \frac{1}{2} \left( \frac{k^2 - n}{k+n} \right) t; \quad x=0 \text{ implies that } k=\sqrt{n} = 4.5; \]

k= 5 implies that x= t/10;

The design relation between height and width is unchanged at 2 700/175 = 15.4, which indicates that the wall roughly keeps a 80% capacity of the basic design value. The wall structure and especially the section, carrying a 9 m panel, is shown in Fig 6.7. A calculation of appearing stresses at the design level shows that even at a load rate of 5.5 kN/ m² is it possible to carry two full floors with such a wall. At an even higher load rate, 7.5 kN/m², you would have to confine the structure to one floor plus roof. Note that there must be a reinforcement along the wall to ensure coherence, which is placed on-site before grouting.
Fig. 6.6
Hybrid wall section
NA$H$ = Neutral axis (AAC/PCC)
NA$_0$ = Neutral axis (AAC)

(ex. k= 5)
(x= t/10)
Fig 6.7
Joint between vertical and horizontal hybrid members.
7. DEFORMATION CHARACTERISTICS UNDER LOAD UP TO FAILURE

The BCE-technology intends to keep all structural elements under elastic conditions, at least up to the service load level. The position of the neutral axis is marked by $x$.

Fig 7.1
Hybrid horizontal member. NA= Neutral axis.

$$x = \frac{1}{2} \left( \frac{1}{h^2} (b-2d)h^2 + 2d(h-c)^2 - (b-2a)t^2 \right) \frac{1}{h^2} (b-2d)h + (b-2a)t + 2d(h-c)$$;

Fig 7.2
Distance $X$ versus load rate for 9 m span.
$I_x = x^2[(b-2a)t + 2d(h-c) + 1/n(b-2d)h] + x [(b-2a)t^2 - 2d(h-c)h - 1/n(b-2d)h^2] +$

$(b-2a)t^3/4 + d/(b+h)(4h^2 - 2hc + c^2) + 1/n(b-2d)h^3/3 ;$

**Fig 7.3**
Surface momentum of inertia versus load rate for 9 m span.

Analysing the bending stiffness and the deflection under load requires a discussion on the proper value of the modulus of elasticity, $E$. The value $E_0$ refers to the short term load, normally the live and climate loads, while $E_\infty$ marks the value for long term load, such as the dead load, the permanent and the prestressing loads.

$E_\infty = \frac{E_0}{1 + \varphi_\infty}$; \hspace{1em} $\varphi_\infty = 1$ for both materials, HPC and AAC, which means that

$E_\infty = \frac{1}{2} E_0$; \hspace{1em} $E_0 = 34.0$ GPa, and $E_\infty = 17.0$ GPa for HPC.

For AAC the equivalent values are $E_0 = 1.7$ GPa, and $E_\infty = 0.85$ GPa.

The condition of zero stress at the section bottom under bending moment above service effect level, $M_{FC}$ can be expressed as:

$M_{FC} = 2P_{prestress} \left[ e + \frac{I_x}{Ay} \right]$;

Now, apply this formula on the previous examples!

**A.** $l = 7.5$ m; Load rate 4.5 kN/m$^2$; ($b = 0.6$ m)

$I_x = 207.2 \times 10^6$ mm$^4$; $e = 134.3$ mm; $A = 24080$ mm$^2$; $y = h - x = 193.6$ mm;

$e + \frac{I_x}{Ay} = 134.3 + 44.5 = 178.8$ mm; $M_{FC} = 143.0 \times 178.8 \times 10^{-3} = 25.6$ kNm;

$M_{FC} = S_{FC} \times l^2/8$ implies that $S_{FC} = 3.64$ kN/m, which is 77% of $S_d$;
So, in practice: 3.06 kN/m < $S_s$ < 3.64 kN/m, the upper value bordering the first crack.

Checking with the formula \(0.1 \ q l^2\) leads to 20.5 kNm, which is less than 25.6 kNm, indicating a positive deformation. Assuming a short term modulus of elasticity

\[
\delta_{\text{prestress}} = - \frac{2 \ P_{\text{prestress}} \ e \ l^2}{8 \ E_0 \ I_x} = -19.2 \text{ mm};
\]

\[
\delta_s = \frac{5 \ S_s \ l^4}{384 \ E_0 \ I_x} = 21.3 \text{ mm}; \text{ The total deformation is } \delta_s + \delta_{\text{prestress}} = 2.1 \text{ mm};
\]

This small deformation under full service load effect shows the remarkable stiffness of the hybrid. Considering the creep on parts of the load effect, on the dead weight, including the permanent load, and on the prestressing leads to a total deformation of minus ten (-10,0) mm, that is ten millimeters upwards. Increasing the load towards failure pushes the deformation downwards, passing zero with cracks opening.

A check of the stress distribution on the middle section subject only to prestressing and dead weight (a situation lying on the bed of manufacture) shows that the uppermost fibre is still under compression (-0.24 MPa), so there is no real need for reinforcement.

The fact that there is some steel (2 ∅ 5) in the flange serves the purpose of shear capacity and safety, for instance in connection with lifting. (Lifting requires a particular analysis, beyond the present study.) A check on the stress at the bottom fibre shows that this is −0.82 MPa (compression on AAC), while the design value is −1.27 MPa. The compression on the bottom fibre of HPC, 50 mm above the section bottom, is -13.6 MPa, which is substantially less than the design value, -21.5 MPa.

### B. \(l = 9.0 \text{ m}; \text{ Load rate } 7.5 \text{ kN/m}^2; \ (b = 0.6 \text{ m})\)

\[
l_x = 327.8 \times 10^6 \text{ mm}^4; \ e = 137.9 \text{ mm}; \ A = 40735 \text{ mm}^2; \ y = h-x = 203.4 \text{ mm};
\]

\[
e + \frac{l_x}{A_y} = 137.9 + 39.6 = 177.5 \text{ mm}; \ M_{FC} = 63.2 \text{ kNm};
\]

\[
M_{FC} = S_{FC} \times l^2/8 \text{ implies that } S_{FC} = 6.24 \text{ kN/m}, \text{ which is 82\% of } S_d;
\]

So, in practice: 4.28 kN/m < $S_s$ < 6.24 kN/m, the upper value bordering the first crack.

Checking with the formula \(0.1 \ q l^2\) leads to 34.0 kNm, which is much less than 63.2 kNm, indicating a considerable positive deformation, downwards. Assuming a short term modulus of elasticity:

\[
\delta_{\text{prestress}} = - 44.6 \text{ mm};
\]

\[
\delta_s = 47.8 \text{ mm}; \text{ So, the total deformation is } 3.2 \text{ mm, again indicating a remarkable stiffness of this hybrid. Considering the creep on parts of the load effect, leads to a total deformation of } -24.8 \text{ mm, that is upwards.}
\]

A check of the stress distribution on the middle section subject only to prestressing and dead weight shows that the uppermost fibre is under tension at 1.41 MPa (this is close to the design value of tensile capacity in concrete). It is equivalent to a tensile force of 4.16 kN to be carried by the reinforcement (3 ∅ 6), which means a steel stress of barely 50 MPa, far below the design value of more than 200 MPa. Again, the steel serves the
purpose of shear capacity and safety in handling. But checking on the stress distribution at the bottom of the section from the influence of prestressing and dead weight reveals serious concerns about this design. The bottom fibre is compressed at – 1.50 MPa, not considering the loss of prestress from elastic compression, which is more than the design value – 1.27 MPa. Moreover checking on the concrete stress at the bottom of the web shows – 24.8 MPa, more than the design value at – 21.5 MPa. The conclusion from these observations is that the influence of compression on the prestressing level must be considered. A detailed calculation shows that the elastic loss of prestressing is 13.6%, which brings down the compression stresses to -1.24 (AAC) and -20.5 (HPC) MPa, just below the design values, - 1.27 (AAC) and – 21.5 (HPC) MPa. An equivalent calculation for the load rate 5.5 kN/m² and 9 m span shows similar results, with a minute reduction of stresses.

The conclusion from this worst case is that the whole spectrum of load rates in Table 1 and spans up to 9 m can be applied with the BCE technology.

The full design of horizontal spanning 9 m - elements can be summarized in graphical form, in Figs 7.2 – 7.3. The deformation under load at different load rates are shown in Fig 7.4, where the effect of creep is clearly illustrated. The loci of deformations at service load and at first crack are marked for consideration of where the panels are in real life. It is obvious that the deformation at first crack is close to nil when the panel is young, while the deformation at mid-span becomes increasingly negative with time at first crack. The stresses are not influenced by time. The distance between the service load levels and the levels at first crack are growing with the load rate and is close to nil as the rate is low. The long-term distance is shown in the figure. The relation between load at first crack and the design load is fairly constant, at 0.82.
Fig 7.4
Mid-deformation of a horizontal 9 m member versus load rate at different ages. (Orientation changed for better visibility)
8. EMBODIED ENERGY OF MATERIALS AND BCE-STRUCTURES

A building requires energy in service during its lifetime, however long this will be, but also in construction, when embodied energy of the engaged materials is of primary interest. In studying the amount it is traditionally claimed that the energy in service is far higher than the embodied energy of the materials. This used to be true but with modern levels of heat insulation and other measures to reduce energy consumption, embodied energy is becoming of higher and relatively new interest.

Several producers of AAC have released data on energy consumption in AAC production. Figures are influenced by the specific choice of raw materials and different levels of steam recycling in the autoclaving process. For blocks an average (Mohit/NXTBLOC, Wehrhahn, Energy star, S Aroni, Hällabrottet) value is 265 kWh/m$^3$ with a minimum value at 250 kWh/m$^3$ (Wehrhahn). The minimum value indicates what is possible with the best technology at present. A full ash related recipe may further reduce the input of energy in production – ash is a waste material from a combustion process, so part of the energy has been paid for in the previous stage. Plants focusing on reinforced material (Ikaalinen, Dalby) use 400 kWh/m$^3$ or close to 50% more, apparently due to longer process time and the steel content. A mixed production (Briesemann/Frey) is declared at 345 kWh/m$^3$.

The emission of $CO_2$ is 210 g/kg for concrete, 265 g/kg for AAC and 1,000 g/kg for steel. For reinforced AAC the figure is 425 g/kg. Per volume, AAC emits one fourth of what concrete stands for. The weight relation is one fifth.

Comparative calculations of emissions from panels show that the BCE system emits less than competitive systems. It is apparent from Fig 8.1 that the live load level has little influence on the emissions of on-site concrete structures, whereas the influence is significant with a full AAC structure. The reason is that the dead load dominates with on-site concrete, while it is less important with AAC. The advantage of BCE can be summarized approximately and illustrated as in Fig 8.2, which shows that on-site concrete emits twice as much as BCE.

It may cause some disappointment that the differences are not larger. But we are considering the use of concretes, one way or the other, and energy saving is not the only parameter. The saving of aggregates and steel may be of a more significant value and in this respect the BCE method stands out as most rewarding.
Fig 8.1
Embodied energy emissions of CO$_2$ (carbon dioxide) at different load intensities.

8.2
Full bars indicate relative emissions of carbon dioxide. Hatched areas indicate proportion between concretes.
9. Tests on BCE

The first tests on BCE were conducted in 1983 under my supervision in collaboration with the company Tretum AB and its capable engineer Rolf Bergdahl. The configuration of test panels are obvious from Fig 9.1. The span of the panels was 4.2 m. It was made clear from these tests that the key to a reliable function under load is the adhesion between the AAC and the HPC grout. This must be secured by keeping the surfaces and adjacent AAC material wet according to Bagheri & Hellers (17), which may require extra wetting under dry conditions. The capillary suction pulls the moisture into the critical space. With poor adhesion the load capacity is soon lost by shear failure, while a good adhesion secures a surprisingly high capacity, limited only by deformation, Fig 9.2. These early tests confirmed the fact that BCE is actually a promising technology but sensitive to the process of production – conditions must be in good order to secure a good result. But this is true also for traditional AAC production, so there is nothing new about the approach. The answer in both cases is high quality control.

Fig. 9.1
Horizontal hybrid member. Test arrangement.
9.2
End part shows considerable deflection after member failure.

These early conclusions were checked at SP (National Institute for Materials Testing) that same year (1983) with a full-scale test on a 4.2 m panel, Fig 9.3. The deformation chart under load, Fig 9.4, is clearly non-linear from the start, which indicates a lack of adhesion between materials. The preparation of the panel before testing had obviously not been ideal.

9.3
Horizontal member before bending test.
Deformation at midsection versus load on horizontal member.

The year after (1984) an equivalent test was performed on an element with no-bond prestressing reinforcement, simulating an on-site production, Fig 9.5. The testing was performed with the element upside-down and the non-linear behaviour in deflection was no surprise. But the element showed considerable bending stiffness. The idea to use AAC blocks in this connection goes beyond the mere in-fill purpose – the blocks interact with the surrounding concrete in a hybrid structure, which is the basis of the BCE technology.
9.5
No-bond prestressed horizontal member at 15 kN load.

Testing of up to 9 m panels is published by Flansbjer & Hellers (27). The tests were conducted again at the National Institute for Materials Testing, this time in Borås. The AAC material was taken from England, based on an ash recipe. By cautious preparation, wetting, the materials were acting well together under load – the first part of the deformation was truly linear. Fig 9.6. Failure occurred after separation due to shear. The tests showed very convincingly that the hybrid did work in the way anticipated and attained the full capacity according to calculation. The paper contains two examples of design.
9.6
The relation in tests between displacement(s) versus load. $l = 9.0\text{m}$. 
10. Production lay-out of panel

The scheme of production is outlined in Fig 10.1. The idea to produce panels for delivery within 24 hours, so all equipment meeting incoming blocks must be circulated within a day. If necessary this calls for using a curing chamber to speed up the ripening of the hybrid. Needless to say, the scheme is radically different from the traditional making of unstressed panels, which is a comment that has been received from traditional producers. Of special interest is the conditions of the AAC surfaces which will meet the concrete grout – they must be kept clean of dust and contain enough moisture to serve as a curing supply in the build-up of a bond between the concretes. Note that the work-force is limited to 10 per eight-hour shift (R Bergdahl). By mechanizing and computer management of the production line the number could possibly be halved. The BCE idea is to allow any reasonable format in the interest of architectural variation and this view should be integrated in the scheme. Only by practice can solutions appear.

Fig 10.1
Conceptual separated production layout for an alternative hybrid AAC production plant.
11. BUILDING SYSTEMS, EXAMPLES (INCL. SEISMIC CONSIDERATIONS)

With reference to Bagheri (13) some examples of joints between horizontal and vertical members are exhibited, one-sided and double-sided. Only traditional masonry is applied. The joints are either dry or reinforced in order to withstand possible seismic loads. All steel must be grouted to full compliance with surrounding structure. As demonstrated previously in connection with walls, there is a supplementary possibility to use hybrid members also for vertical load-bearing, the detailing left to the reader. The front block applied at one-sided joints may be taken from a higher density in order to accommodate to the concrete strip and strengthen the joint by attracting the load. This is presented in Fig 11.1 to Fig 11.6.

A modern wall of high insulation quality is demonstrated in Fig 11.7 and Fig 11.8.

Window and door openings require beams, which can be produced in hybrid technology. The unsymmetrical solutions demonstrated in the following examples must be stable in time, which requires equal creep properties, Fig 11.9 and Fig 11.10.
Fig 11.1
Section
Connection between exterior bearing wall, vertical wall members to foundation slab
Fig 11.2
Section
Connection between exterior bearing wall, vertical wall members to foundation slab.
(Extra reinforcement is recommended for seismic regions)
Connection between exterior bearing wall member to floor member

<table>
<thead>
<tr>
<th>Floor Member’s Span (m)</th>
<th>Wall Member’s Width (mm)</th>
<th>The Minimum Support Length of Floor Member Over Wall Member (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L &lt; 5</td>
<td>200</td>
<td>65</td>
</tr>
<tr>
<td>5.0 ≤ L ≤ 7.5</td>
<td>250</td>
<td>90</td>
</tr>
<tr>
<td>7.5 &lt; L ≤ 9.0</td>
<td>250 – 300</td>
<td>120</td>
</tr>
</tbody>
</table>
Fig 11.4
Section
Connection between exterior bearing wall member to floor member
(extra reinforcement is recommended for seismic regions)
Fig 11.5
Section
Connection between interior bearing wall, vertical wall members to horizontal members.

<table>
<thead>
<tr>
<th>FLOOR MEMBER’S SPAN (m)</th>
<th>WALL MEMBER’S WIDTH mm</th>
<th>THE MINIMUM SUPPORT LENGTH OF FLOOR MEMBER OVER WALL MEMBER mm</th>
</tr>
</thead>
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<td>250 – 300</td>
<td>120</td>
</tr>
</tbody>
</table>
Fig 11.6
Connection between interior bearing wall, vertical wall members to horizontal members. (extra reinforcement is recommended for seismic regions).
Fig 11.7
Section
Connection between exterior bearing wall member to floor member with insulation.
FLOOR MEMBER'S SPAN (m) | WALL MEMBER'S WIDTH mm | THE MINIMUM SUPPORT LENGTH OF FLOOR MEMBER OVER WALL MEMBER mm
--- | --- | ---
L < 5 | 200 | 65
5.0 ≤ L ≤ 7.5 | 250 | 90
7.5 ≤ L ≤ 9.0 | 250 – 300 | 120

Fig 11.8
Section
Connection between exterior bearing wall member to floor member with insulation.
Fig 11.9
Section
Connection between exterior bearing wall member to floor member and window lintel.

<table>
<thead>
<tr>
<th>WINDOW SPAN (mm)</th>
<th>THE MINIMUM SUPPORT LENGTH OF LINTEL OVER WALL MEMBER (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L \leq 1800 )</td>
<td>150</td>
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<tr>
<td>1800 &lt; ( L \leq 3000 )</td>
<td>300</td>
</tr>
</tbody>
</table>
Fig 11.10
Section
Connection between exterior bearing wall member to floor member and window lintel.
(Extra reinforcement is recommended for seismic regions).
12. CONCLUSIONS, ECONOMY

ENVIRONMENTAL BENEFIT
Surrounding society requires environmental consideration with respect to the use of raw materials and energy consumption in process.

A. Waste materials such as PFA or silicious dust can be used for making AAC
B. Waste steam from utility plants (electricity from coal) can be used in autoclaving
C. AAC production should be linked to utility plants to catch PFA and steam
D. Steam in the autoclaving process can be circulated (done already)
E. Cementitious binders are used (Siporex recipe) (done already)
F. Steel use radically limited, by 75% (versus homogeneous AAC)
G. Use of concrete radically limited, by 75%, compared with concrete (prefab)
H. Emission of CO\textsubscript{2} reduced by 30% compared with a genuine concrete HDF
I. Transportation of blocks very rational, stacked on truck to an on-site plant

CONCLUSIONS
The supply of structural materials is a critical issue in a world, ever more concerned about raw materials and energy

AAC is a material, which combines low requirements on raw materials with low energy use and release of carbon dioxide in process – the quality of these features may be improved radically by building an industry combinatory park (around a utility plant) in which PFA and steam are generated and used in successive steps

The inclusion of non-stressed reinforcement in the moulding process in order to build horizontal members has never been a success – the amount of steel is high due to low compressive and shear strengths of the AAC material and the need for special anchorage steel.

These draw-backs are all compensated for by forming a hybrid in which HPC under prestressing is combined with AAC. HPC has excellent compressive and good shear strengths. The prestressing enhances the shear strength by which any reinforcement for shear can be omitted altogether.

The importance of heat insulation is not influenced by forming a hybrid with less capacity – it only goes to show that AAC-structures must be completed on the exterior side with high insulating material in the temperate part of the world, such as foamed plastic material or mineral wool. The forming of wall hybrids gives room for such insulating materials.

The hydrothermal behaviour of AAC is balancing the moisture content of the indoor air, obviates the need for vapour barriers, and promotes well-being, even health and counteracts allergies.
13. References

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