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A novel steel–concrete composite system for modular nuclear reactors

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Small modular nuclear reactors offer an opportunity to develop innovative design and construction techniques with the aim of maximising factory prefabrication and transportability of prefabricated units, and of facilitating their site assembly. This can be achieved through designs that incorporate advanced modular construction techniques. This paper provides an overview of the work undertaken to develop an innovative form of composite steel–concrete modular construction. The basic building block of the system is a steel plate folded into a U-shape with a concrete infill. Individual building blocks – called 'steel bricks' – can be joined together to create larger units that make up the modules of a small modular reactor. The work addresses the application of Eurocode design rules to structural elements constructed of steel bricks, as well as all the key manufacturing operations including plate folding; joining of units together; lifting of units; stability during erection; alignment and manipulation; joints between panels; connection to reinforced concrete foundations; and quality assurance and inspection techniques. The concept is being demonstrated at full scale through the construction of a section of a diesel generator building.

Notation

A_a	critical cross-sectional area of steel (accounting for any holes)
A_c	cross-sectional area of concrete
b	width of the steel brick
E_{cm}	secant modulus of concrete
E_{effc}	effective modulus of elasticity of concrete
E_s	elastic modulus of the steel plate
$(EI)_{eff}$	characteristic value of the effective flexural stiffness of the composite member
f_{cd}	design yield strength of concrete, $f_{ck}/\gamma_c = f_{ck}/1{\cdot}5$
f_{ck}	characteristic compressive cylinder strength of concrete at 28 days
f_u	ultimate strength
f_{yd}	design yield strength of steel
h	height–depth of the steel brick
h_n	depth of the plastic neutral axis from the centre line of the cross-section
I_a	second moment of area of the steel section
I_c	second moment of area of the uncracked area of concrete
L	length
$M_{c.Rd}$	design resistance for bending about one principal axis of a cross-section
M_{Ed}	design bending moment
$M_{pl.Rd}$	plastic bending resistance
N_{cr}	elastic critical force
N_{Ed}	applied axial force
$N_{pl,Rd}$	plastic resistance to compression
s_t	minimum spacing distance between repeated openings
t	thickness of steel plate of the steel brick
$V_{a,Rd}$	design value of shear resistance of the steel section
V_{Ed}	design value of applied shear force
W_p	plastic section modulus for the steel section
W_{pc}	plastic section modulus of concrete (uncracked)
W_{pn}	plastic section modulus of the steel section within $2h_n$ from the middle line of the composite cross-section
W_{pcn}	plastic section modulus of concrete within $2h_n$ from the middle line of the composite cross-section
α	imperfection factor
δ	steel contribution ratio
$\bar{\lambda}$	relative slenderness
μ_d	moment resistance ratio obtained from the N–M interaction curve

1. Introduction

Most buildings and containment structures in nuclear power plants (NPPs) have been built from reinforced or post-tensioned concrete in the past. This has placed heavy reliance on cast in situ construction. It has led to lengthy site operations to accommodate the time spent in assembling the shuttering and reinforcing steel and stripping the shuttering after curing of the concrete. Attachments to the walls and floors, which are used to support electrical and mechanical equipment (e.g. piping, cable trays etc.), require the installation of either embedded steel plates or concrete expansion anchors into walls, floors and ceilings (there are around 100 000 plates in a typical NPP) (Meiswinkel *et al.*, 2013). Installing embedded steel plates is difficult because anchor rods and their end plates need to be inserted in the very dense reinforcement, requiring the displacement of the reinforcement bars. Installation of concrete expansion anchors entails the time-consuming process of locating reinforcing bars in the concrete to avoid cutting them. This is followed by drilling into the concrete and inserting anchors for bolting steel plates to the concrete surface to attach supports.

New-generation NPP manufacturers have been developing modular construction techniques that make greater use of prefabrication and factory assembly and reduce reliance on site activity. A modular build approach has already been deployed in some parts of large nuclear plants. However, due to their size, small reactors are particularly suited to modularisation with the potential for repeat volume standard components.

Composite steel–concrete (SC) modules are made by pouring concrete between two steel plates that serve the dual purpose of reinforcement and permanent formwork. Being located on the surface of the concrete, the steel plates provide more structurally efficient reinforcement to the concrete than embedded reinforcement. Studs welded on the inner surface of the steel plates provide composite action between steel and concrete. Previous research (AIJ, 2005; Anwar Hossain and Wright, 2004; McKinley and Boswell, 2002; Shanmungam *et al.*, 2002) has shown that the SC structures exhibit greater stiffness and strength as well as toughness when compared with that of conventional concrete structures. Another advantage is that the steel plates act as impermeable membranes in SC structures, eliminating any leakage problems, which is an important requirement in some areas of NPPs. For these reasons, a number of reactor manufacturers have made SC construction an integral part of their new designs (Masayoshi *et al.*, 2009; Schulz, 2006; Toyama *et al.*, 2009).

To support the surface plates during assembly, lifting, transportation, concrete pouring and curing, the two plates must be connected together to form a self-supporting unit. In this way, parts of modules may be fabricated off site and transported as units to be assembled on site and connected together. Many attempts have been made over the past 25 years to devise practical, economic and safe methods of connecting the plates. They generally use bars, ribs or trusses welded to both plates. When the concrete sets, these ribs, trusses or bars provide shear reinforcement to the concrete. Figure 1 shows an SC panel with shear studs and bar connectors.

Welding of ribs or bars is manual and raises issues of economy, quality and health and safety due to the confined space between the plates. One method in which automatic welding of the bars to the two plates was achieved is 'bi-steel' (Corus UK, 2003), where both ends of each bar are simultaneously friction-welded to both plates in an automated process. The need for shear studs is eliminated in bi-steel because the bars are of such a density as to provide shear reinforcement to the panel and shear connection between the plates and the concrete. However, manufacturing constraints of bi-steel dictate that the panel thickness is limited, fabrication requires specialist equipment and the friction-welding process places limitations on the plate thickness to avoid 'burn through' of the plates and on bar diameter to avoid distortion of the bars during welding.

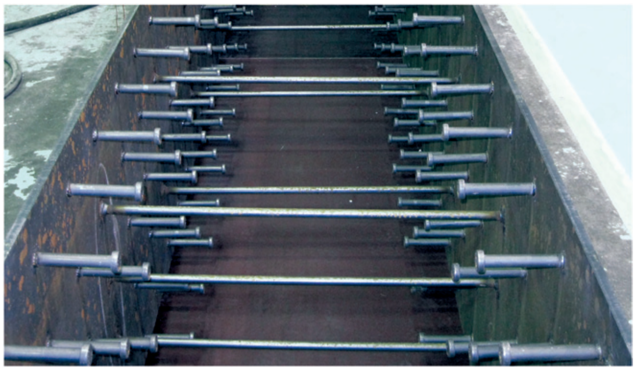

Figure 1. Typical SC panel with bar connectors and shear studs

In this paper, the authors discuss a new innovative form of manufacture of SC modules (referred to as 'steel bricks') suited for the construction of small modular reactors (SMRs) and which, importantly, overcomes the difficulties identified above. The authors consider how Eurocode design methodologies can be applied to steel bricks and discuss the manufacturing and construction issues. Performance under seismic, impact and other hazardous actions is outside the scope of this paper, but has been considered by several authors who studied the behaviour of SC structures (Kim *et al.*, 2009, 2015; Lee and Kim, 2015). Such studies have demonstrated good performance when compared with reinforced concrete and highlighted the benefits of confinement of the concrete between the steel plates, a particular advantage under impact loading. Corrosion of the exposed steel plates also needs to be considered to ensure that appropriate corrosion protection is provided in line with the design life of the plant. This too is outside the scope of this paper.

2. Steel brick concept

Steel brick is a new form of SC modular construction that overcomes the manufacturing problems of earlier generation SC systems. (Modular Walling Systems Ltd (MWS) hold a United States (US) Patent (other territories pending) for steel bricks. Caunton Engineering Limited (CEL) and The Steel Construction Institute (SCI) are collaborating in a project to develop steel bricks, considering analysis, design, physical testing, realistic scale fabrication and construction trials. The target market is the civil nuclear power plant (NPP) sector, most notably the emerging SMR market.) A steel brick is made by folding two plates into two 'L'-shaped sections (which are then welded to form a 'U' shape) as shown in Figure 2. This enables steel bricks with different plate materials on the two sides of a wall (e.g. stainless steel and carbon steel) to be manufactured as is required for some parts of an NPP to provide adequate corrosion resistance. Such construction should not give rise to bimetallic corrosion issues as the carbon steel/stainless steel joint is embedded in concrete (HA, 2002). Alternatively, a single plate can be double folded into a 'U' shape. Steel bricks are connected end to end to form sub-modules. Holes cut into the base of the 'U' allow the concrete to flow between bricks during concreting. Corner bricks are fabricated with holes in the vertical side of the 'U' to allow concrete to flow round the corner.

Steel bricks can be welded together, both end to end and vertically to create large sub-modules (Figure 3(a)). Using this concept, no tie bars or internally welded ribs are required, as these are replaced by the base of the 'U' which is integral to the steel brick and which also provides shear reinforcement to the concrete. The same concept can also easily be used to manufacture module floors in which the unit brick has holes in both the base of the 'U' and in one of the vertical sides to allow filling and flow of concrete in the plane of the floor (Figure 3(b)). This also enables the floor to have a concrete finish similar to conventional reinforced concrete floors by allowing the concrete to rise above the plate by a small amount. Attachments for electrical and mechanical equipment support can be easily added as shown in Figure 3(c).

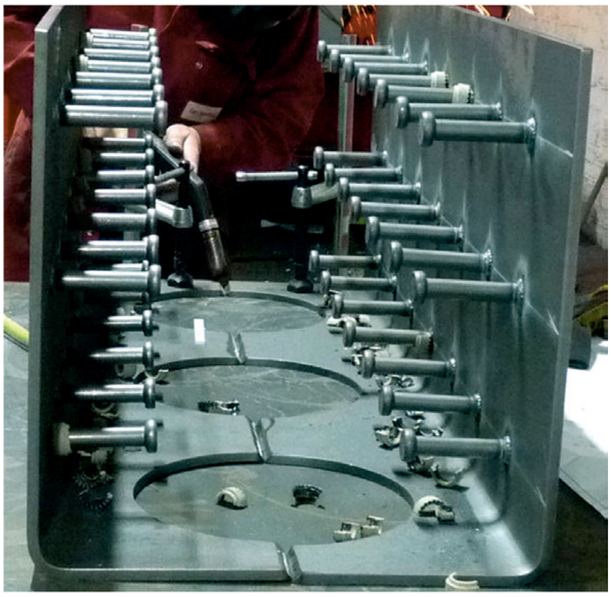

Figure 2. Steel brick concept

3. Materials

The design, manufacturing and test trials carried out to date have been on the following material specifications.

- *Steel plates*: S355, subgrade J2 in accordance to PD 6695-1-10 (BSI, 2009). $E_s = 210$ GPa. Steel plate thickness 8–15 mm.
- *Headed shear connectors*: 19 mm diameter, $f_u = 450$ N/mm^2, with as-welded height not <75 mm. Shear connectors should be capable of interfacial slip of at least 5 mm.
- *Concrete*: normal weight, grade C40/50, $E_{cm} = 35$ GPa.

4. Application of EN1993-1-1 to the design of steel brick components (construction stage)

The unfilled steel brick units are subjected to axial and transverse actions during transportation, handling and lifting operations, stacking of panels to form walls, concrete placement and so on. Therefore, it is necessary to ensure that the panels are capable of resisting the stresses caused by these actions and that deformations remain within acceptable limits.

4.1 Classification of the non-composite steel brick section

Calculation of the resistance of steel brick bare steel members requires the determination of section properties at the critical cross-section. A typical steel brick member has been chosen with a cell diameter equal to 60% of the element width. This allows good flow of concrete between adjacent steel bricks but may be further optimised from a structural point of view.

Steel brick elements used in a horizontal orientation (such as floor elements) will typically have a depth ranging from 200 to 400 mm with a structural height of 300–500 mm (allowing for 100 mm of concrete topping). For a typical floor element with h and b up to 360 mm and a plate thickness of 10 mm, the cross-section classification is Class 1 or Class 2 in accordance with EN 1993-1-1 (BSI, 2005a). Steel bricks with h and b of up to 390 mm are typically Class 3.

Wall elements with an overall thickness of up to 340 mm with a plate thickness of 10 mm are of Class 2 and elements of up to 370 mm with a plate thickness of 10 mm are of Class 3.

4.2 Bending

For bending resistance calculation, the section is treated as an asymmetric I-section as shown in Figure 4. The bending

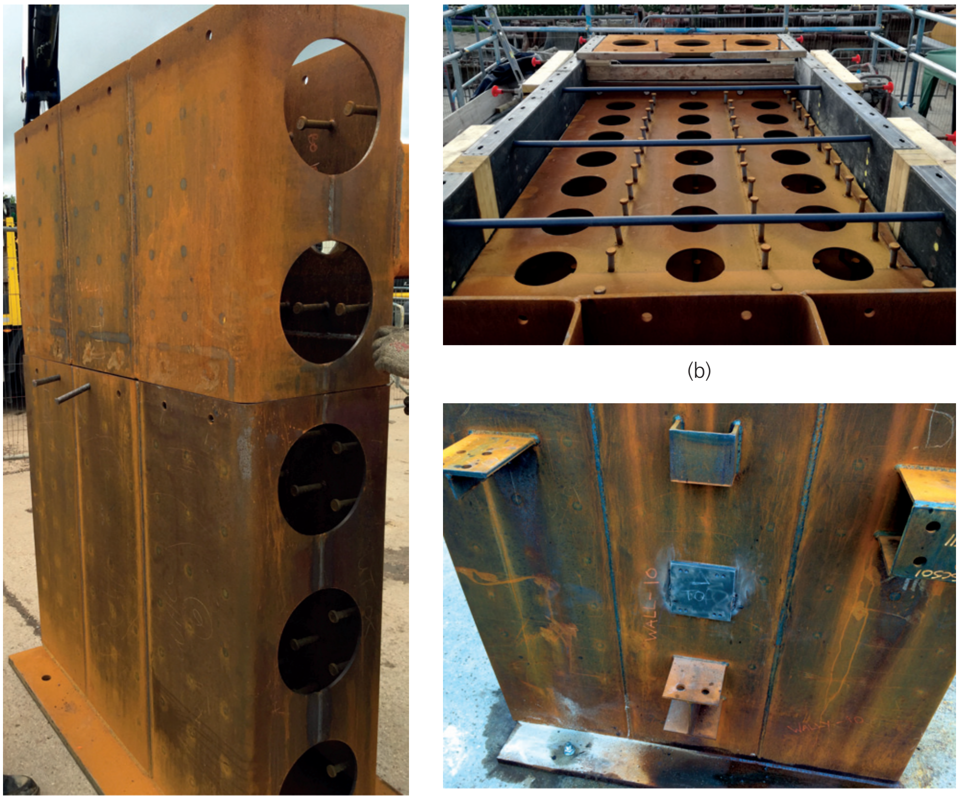

Figure 3. Steel brick (a) walls, (b) floors and (c) attachments

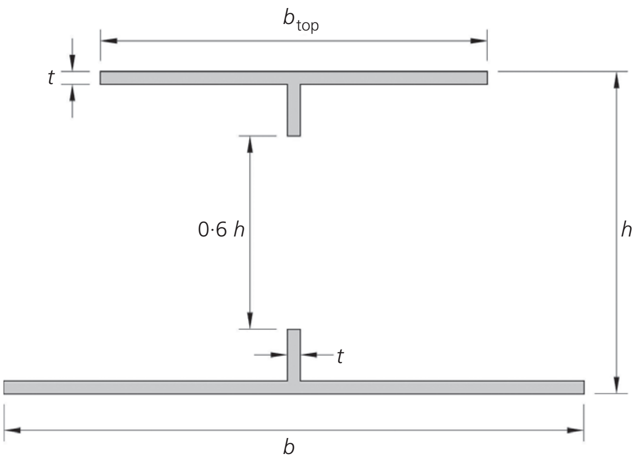

Figure 4. Horizontal steel brick member modelled as an asymmetric I-section

resistance per steel brick can be calculated using clause 6.2.5 of EN 1993-1-1. For Class 1 or Class 2 section, the bending resistance is given by

1. $$M_{c,Rd} = W_p f_y$$

4.3 Axial compression – global buckling

Member resistance to global buckling may be evaluated using buckling curves as given in EN 1993-1-1 (BSI, 2005a). The buckling resistance of a steel brick wall is determined as a function of its slenderness, $\bar{\lambda}$, expressed as a proportion of plastic resistance of the cross-section in compression, $N_{pl,Rd}$.

A steel brick vertical compression member acting as a wall only requires consideration of buckling about the z–z axis. It is recommended to use buckling curve 'b' where the imperfection parameter, α, is equal to 0·34.

4.4 Effect of fresh concrete pressure during construction

The effect of concrete pressure during construction is generally the sizing criterion for the thickness of the steel plate in vertical steel brick members. The concrete flow rate, temperature, type and density of concrete will affect the pressure. For a typical steel brick vertical member of square dimensions of around 450–500 mm, the maximum serviceability limit state (SLS) pressure due to fresh concrete should be limited to 140 kN/m^2. The maximum plate deflection should be limited to b/360 under SLS actions.

5. Application of EN1994-1-1 to the design of composite steel brick components

5.1 Classification of the composite steel brick section

EN 1994-1-1 (BSI, 2004) clause 6.6.5.5(2) allows compression flanges which are otherwise classified as Class 3 or Class 4 in the non-composite condition to be assumed as at least Class 2 in the composite condition if shear connectors are present. This assumes that the spacing between shear connectors in the load direction does not exceed $22t\sqrt{(235/f_y)}$ and the clear distance from the edge of a compression flange to the nearest line of shear connectors does not exceed $9t\sqrt{(235/f_y)}$.

5.2 Composite steel brick compression members

Two design methods are given in EN 1994-1-1 for members under axial compression: the general method and the simplified method. Here the authors consider the application of the simplified method (clause 6.7.3 of EN 1994-1-1) to the design of composite steel brick compression members. In this configuration, steel brick members are assumed to act as multiple inter-connected concrete-filled rectangular tubes with doubly symmetrical uniform cross-sections. The aspect ratio of depth (h) to the width (d) of the composite cross-section is within the limits of between 0·2 and 5 since typical members are of approximately square form. The application of the calculation steps of the simplified method to a steel brick compression member is set out in the following sections.

5.2.1 Relative slenderness

The relative slenderness $\bar{\lambda}$ should be $\leq 2{\cdot}0$ in accordance with EN 1994-1-1 clause 6.7.3.1(1) and calculated using expression 6.39 of EN 1994-1-1 (BSI, 2004). For typical steel brick configurations of square form of dimensions from 300 to 500 mm with steel plate thickness >8 mm and a critical length <8 m, the relative slenderness is <1.

The elastic critical buckling force, N_{cr}, is given by

$$N_{cr} = \frac{\pi^2 (EI)_{eff}}{L^2} \tag{2}$$

The characteristic value of the effective flexural stiffness of the composite member $(EI)_{eff}$ is obtained by combining the flexural stiffness of steel and uncracked concrete components of the cross-section as per EN 1994-1-1 clause 6.7.3.3(3) (6.40), where the correction factor for concrete is given as 0·6.

$$(EI)_{eff} = E_s I_a + 0{\cdot}6 E_{cm} I_c \tag{3}$$

Under long-term loading, creep and shrinkage of concrete may cause a reduction in the effective flexural stiffness of the composite columns, thereby reducing its buckling resistance. In such cases, E_{cm} should be reduced to the value $E_{c,eff}$ in accordance with equation 6.41 of EN 1994-1-1. As a simple rule, the long-term effects should be considered in a composite column if its buckling length-to-depth ratio exceeds 15.

5.2.2 Steel contribution ratio, δ

The steel contribution ratio, δ, as defined in EN 1994-1-1 clause 6.7.3.3(1) for a typical 500 mm square form of steel brick with 10 mm plate and C40/50 concrete is about 0·5 per steel brick, well within the limits of $0{\cdot}2 \leq \delta \leq 0{\cdot}9$.

5.2.3 Plastic compressive resistance

The plastic compressive resistance $N_{pl,Rd}$ per unit width of a composite cross-section should be calculated by adding the plastic resistances of its components as per EN 1994-1-1 clause 6.7.3.2(1) (6.30) assuming the coefficient 0·85 is replaced by 1 for steel bricks (treating steel bricks as concrete-filled sections).

$$N_{pl,Rd} = A_a f_{yd} + 1 \times A_c f_{cd} \tag{4}$$

5.2.4 Plastic bending resistance

The plastic bending resistance of a composite steel brick member per unit width $M_{pl,Rd}$ can be calculated as

$$M_{pl,Rd} = f_{yd}(W_p - W_{pn}) + 0{\cdot}5 f_{cd}(W_{pc} - W_{pcn}) \tag{5}$$

The above bending resistance equation follows the guidance in section 23.3.2 (23.7) of the *Steel Designer's Manual*, 7th edn (Chung and Lawson, 2012).

5.2.5 Shear resistance

The applied shear force V_{Ed} may be conservatively assumed to be resisted entirely by the steel section. No reduction to the resistance of the cross-section in compression or bending is made when V_{Ed} is smaller than half of the shear resistance of the steel section – that is $0{\cdot}5V_{a,Rd}$ (clause 6.2.2.2 of

EN 1994-1-1 and clause 6.2.6 of EN 1993-1-1). When this limit is exceeded, the influence of the vertical shear on the resistance to bending should be taken into account by a reduced design yield strength in the shear area of the steel section in accordance with clause 6.2.2.4(2). If the applied shear force is greater than the shear resistance of the steel, contribution from concrete should be included in accordance with EN 1994-1-1 clause 6.7.3.2(4).

5.2.6 Interaction curve for combined compression and bending

The resistance of composite cross-sections to combined compression, N, and uniaxial bending, M, and the corresponding non-linear N–M interaction curve (as shown in Figure 5) are evaluated according to the rectangular stress blocks in the elements of the cross-sections.

The non-linear N–M interaction curve for a composite cross-section may be readily simplified into a multi-linear interaction curve with three to five key points, as shown in Figure 5. These points of the multi-linear interaction curve are determined from internal forces and moments, based on the rectangular stress blocks of the various elements of the composite cross-section with different positions of the neutral axis, h_n, of the cross-section. Tensile strength of concrete in the derivation of moments is conservatively ignored for simplicity and because its contribution is relatively insignificant.

For a steel brick vertical member of square form, the following points may be assumed to form the N–M poly-linear interaction curve. At point A, the plastic compression resistance

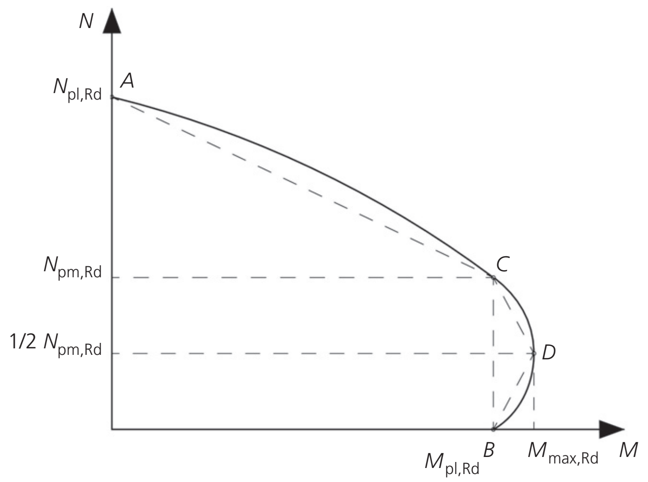

Figure 5. Interaction curve for combined compression and bending with points A to D

$N_{pl,Rd}$ is assumed and at point B the plastic bending resistance $M_{pl,Rd}$. Point C corresponds to the plastic compression resistance of concrete $A_c f_{cd}$ and the plastic bending resistance, $M_{pl,Rd}$. Point D corresponds to a combination where the plastic neutral axis coincides with the centroid of the cross-section; the axial capacity is half of that at point C ($0{\cdot}5A_c f_{cd}$) and the moment resistance is at its maximum.

5.2.7 Resistance of members to combined compression and uniaxial bending

The following condition based on the interaction curve in accordance to EN 1994-1-1 clause 6.7.3.6(1) (6·45) for steel bricks with steel grade S355 must be satisfied

$$\frac{M_{Ed}}{\mu_d M_{pl,Rd}} \leq 0{\cdot}9 \tag{6}$$

where μ_d is the moment resistance ratio obtained from the N–M interaction curve and may be determined using the expressions below which have been obtained using geometry considerations.

$$\mu_d = 1 - \frac{\chi_d - \chi_{pm}}{1 - \chi_{pm}} \quad \text{when } \chi_d > \chi_{pm}$$

$$\mu_d = 1 \quad \text{when } \chi_d \leq \chi_{pm}$$

where χ_{pm} is the axial resistance ratio due to concrete, which is given by $A_{c}f_{cd}/N_{pl,Rd}$. χ_d is the design axial resistance ratio, which is given by $N_{Ed}/N_{pl,Rd}$.

5.3 Composite horizontal steel brick members

Steel brick horizontal members are treated as multiple interconnected beams (Figure 6) with the web and top flange reduced in area to account for the cellular openings which are generally at regular spacing, $s_t = 0{\cdot}3h$.

Composite action is assumed to be provided by shear connectors welded to the top flange and an effective bearing resistance contribution from concrete bearing on the perimeter of the cells in the web provided the plastic neutral axis (PNA) is within or below the cell.

5.3.1 Bending resistance

Plastic bending resistance is determined at the centre line of a cell (critical cross-section) by equating the internal forces in the steel tee section above and below the cell and in the concrete. This force equilibrium results in different cases for the position

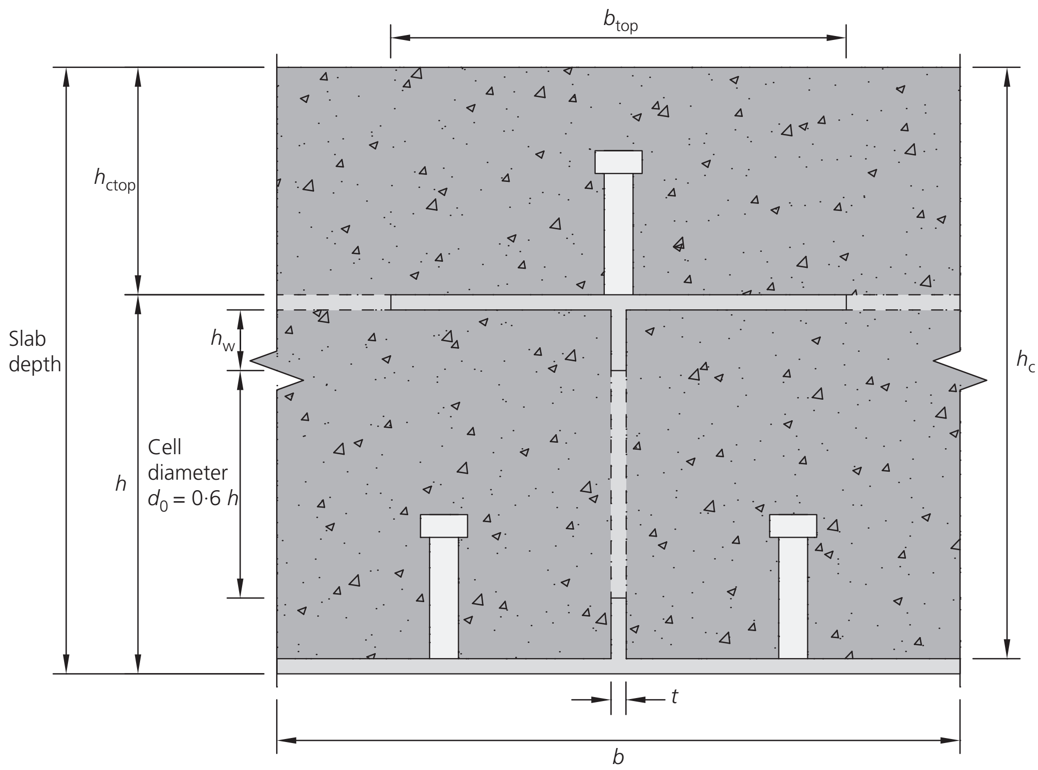

Figure 6. Composite horizontal steel brick member notation

of the PNA (in the concrete, above the steel brick or at a position within the depth of the steel brick).

The concrete compression resistance is taken as the lesser of either $0{\cdot}85f_{cd}$ multiplied by the area of concrete in compression or the resistance provided by the shear connectors between the support and the centre line of the cell under consideration (normally that at which M_{Ed} is maximum).

When the PNA is above the steel brick (within the concrete topping), any tension in the top tee is ignored (Lawson and Hicks, 2011) assuming tension is carried by the bottom tee exclusively since the bottom tee is assumed to yield earlier. When the PNA is within the steel brick, then stress blocks above and below PNA are taken into account. Tension in concrete is ignored. An example where PNA is in the top tee is shown in Figure 7, where x_{web} denotes the distance from the

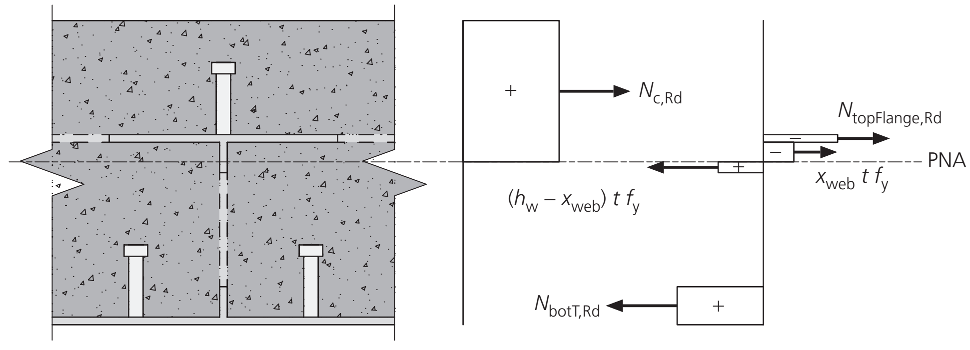

Figure 7. Stress blocks of a case where the PNA is within the top tee

top of the web of height h_w. By taking moments about the PNA for each stress block the bending resistance, for this case, of the member can be calculated as follows

$$
\begin{aligned}
M_{\mathrm{pl,Rd}} = {} & \left(x_{\mathrm{web}} + t + h_{\mathrm{ctop}} - \frac{z_{\mathrm{c}}}{2}\right) N_{\mathrm{c,Rd}} \\
& + \left(x_{\mathrm{web}} + \frac{t}{2}\right) N_{\mathrm{topFlange,Rd}} + \frac{x_{\mathrm{web}}}{2}\left(x_{\mathrm{web}} t f_{\mathrm{y}}\right) \\
& + \frac{h_{\mathrm{w}} - x_{\mathrm{web}}}{2}\left(h_{\mathrm{w}} - x_{\mathrm{web}}\right) t f_{\mathrm{y}} \\
& + \left(h - t - x_{\mathrm{web}} - h_{\mathrm{bot,cent}}\right) N_{\mathrm{botT,Rd}}
\end{aligned} \tag{7}
$$

where the depth of concrete, z_c, in compression is given by

$$z_{\mathrm{c}} = \frac{N_{\mathrm{c,Rd}}}{0{\cdot}85 f_{\mathrm{cd}} \cdot b} \leq \left(h_{\mathrm{ctop}} + t + x_{\mathrm{web}}\right) \tag{8}$$

and the concrete compression resistance is given by

$$N_{\mathrm{c,Rd}} = \min\left(0{\cdot}85 f_{\mathrm{cd}}\, b\, \left(h_{\mathrm{ctop}} + t + x_{\mathrm{web}}\right);\ nP_{\mathrm{Rd}}\right) \tag{9}$$

5.3.2 Shear resistance

The total shear resistance, V_{Rd}, is made up of contributions from the steel brick ($V_{\mathrm{a,Rd}}$) and the concrete (EN 1994-1-1 clause 6.2.2.2). The contribution in shear resistance from the concrete ($V_{\mathrm{c,Rd}}$) is calculated in accordance with EN 1992-1-1 clause 6.2.2.

6. Fabrication and construction issues

6.1 Plate bending

A series of plate bending tests were carried out on 8, 10, 12 and 15 mm plates in lengths up to 10 m. Clause 4.14 of BS EN 1993-1-8 (BSI, 2005b) prohibits welding within $5t$ of a cold-formed corner unless either the cold-formed zones are normalised after cold forming and before welding or the r/t ratio satisfies the relevant value obtained from table 4.2 of the code. The tests demonstrated that greater minimum internal bend radii can be achieved that exceed the minimum specified values.

6.2 Welding of individual steel brick units

Comparative welding trials were performed to assess whether, when welding the two halves of a steel brick together, edge preparation was required. Both metal inert gas (MIG) and submerged arc welding were investigated. The welded specimens were subjected to a series of non-destructive and destructive tests (visual, magnetic particle inspection (MPI), radiography, Vickers hardness survey, Charpy, tensile bend test and macro test). In the case of the MIG-welded specimens, all the non-prepped test pieces failed due to the lack of weld fusion, whereas all the prepped test pieces passed. The submerged arc welded specimens, on the other hand, all passed and this welding process has therefore been adopted.

Where steel bricks are welded together to form large panels, the weld is between the free edge of a side plate and a rolled corner. The shape of the rolled corner provides a natural edge preparation, and the free edge of the side plate needs no further preparation (Figure 8). This forms a ‘single prep butt weld’ as defined in detail 1.9.2, table 1, BS EN 9692-1:2013 (BSI, 2013) for MIG welding, and detail 1.4, table 1, BS EN ISO 9692-2:1998 (BSI, 1998) for submerged arc welding. All welds passed the test and the inspection criteria (visual, MPI, radiography, Vickers hardness survey, Charpy, tensile bend test and macro tests).

A frequent requirement on NPPs is the need for penetrations through walls to accommodate mechanical and electrical services. This was investigated and penetrations of different

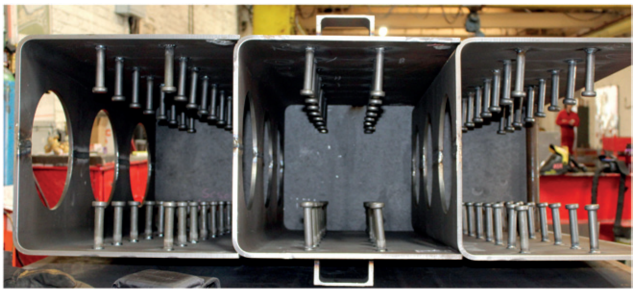

Figure 8. Steel brick-to-steel brick weld detail

Figure 9. Steel bricks with penetration details

shapes and sizes were incorporated into steel brick assemblies as shown in Figure 9. These trials demonstrated how penetrations can be built into SC structures fabricated using steel bricks.

6.3 Site assembly and welding trials

Structural components were designed and fabricated to examine the issues involved in the manufacture and assembly of multiple steel bricks into sub-modules. The initial trial structure comprised two walls and a floor as shown in Figure 10. Connection pieces were manufactured in a number of configurations to explore the fabrication and fit-up issues that would result. Different means of joining wall elements both horizontally and vertically as well as joining floor to wall elements were also investigated.

Manual metal arc (stick welding) and MIG welding were trialled on site for connecting sub-assemblies together. Two types of consumable wires were also trialled, the first requiring carbon dioxide shielding gas, and the second using a self-shielding wire. The latter produced less satisfactory results.

6.4 Concreting trials

The assembled structure (along with a number of other structures comprising a range of connection details) was used for conducting initial concreting trials (Figure 11). A concrete mix was designed to ensure good flowability of concrete. This included the use of well-graded aggregates (with a maximum aggregate size of 20 mm, 60% of aggregates with diameter <5 mm and 30% with diameter <0·5 mm), as well as the use of additives. The open ends were shuttered to prevent the loss of concrete and to enable examination of the concreting quality after concrete setting by removal of the shuttering. Externally mounted vibrator units were used to aid flow of concrete and reduce the risk of any void forming. The concreted structures were subsequently cut up using diamond wire cutting and this enabled the concrete quality to be inspected in several parts of these structures. The trials demonstrated that there were no unwanted voids in the concrete.

Figure 10. Trial structure during site assembly

Figure 11. Concreting trials

6.5 Future work

Following successful completion of medium-scale fabrication, erection and concreting trials, a part of a diesel generator building will be fabricated and constructed to demonstrate the technology at full scale, albeit on a part of a building (Figure 12). The design will explore a range of connection details between members with a range of moment capacities. Construction will also consider issues of full-scale fabrication, manufacturing of large components, lifting (and the need for appropriate lifting points in the sub-modules) handling and concreting, so as to satisfy UK Construction (Design and Management) Regulations (HMG, 2015). The trials will also explore methods of safe and cost-efficient deconstruction of the structure. This is likely to build on the remote diamond wire cutting technique used to dismantle the structures used in the concreting trials.

7. Conclusions

This paper has summarised ongoing work on the development of a new generation of SC modular structures that is suited to the construction of SMRs. The work has explored the application of existing Eurocode design rules at the construction and final stages and summarised practical trials that have addressed all aspects of construction using this technique.

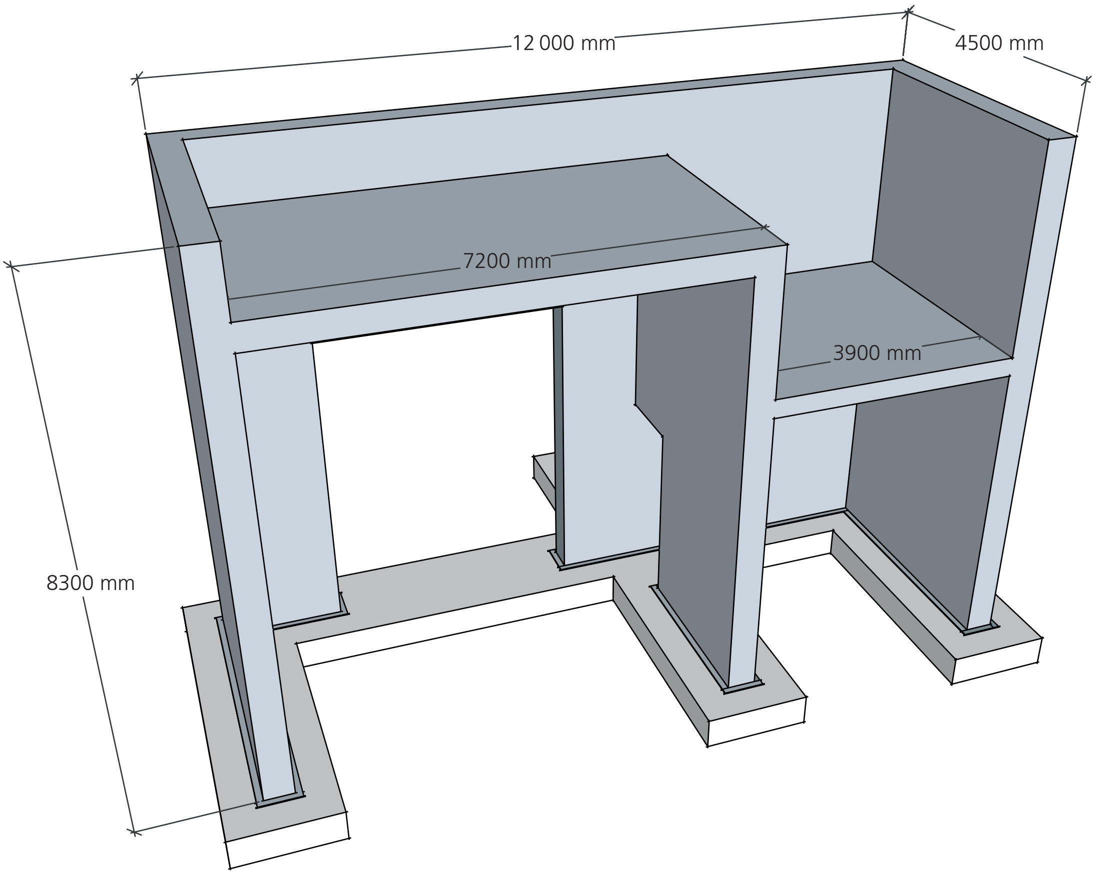

Figure 12. Part of a diesel generator building used to demonstrate construction at full scale

Steel bricks have been shown to overcome most of the manufacturing difficulties encountered with earlier generations of SC construction, while retaining the advantages. The work has demonstrated how existing design rules can be conservatively applied to this form of construction. The manufacturing trials have shown that individual steel bricks and their assembly into sub-modules can be achieved using existing and relatively simple technologies.

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