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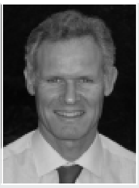

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Modular design for high-rise buildings

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Modular construction is widely used for residential buildings of four to eight storeys and there is pressure to extend this relatively new form of construction to 12 storeys or more. This paper reviews recent modular technologies, and also presents load tests and the analysis of light steel modular walls in compression. A design method for high-rise modular applications is presented taking account of second-order effects and installation tolerances. For the modular walls tested, it was found that the plasterboard and external sheathing boards effectively prevent minor axis buckling of the C sections, so that failure occurred either by major axis buckling or local crushing of the section. In all cases, the results of the tests on 75 mm and 100 mm deep × 1·6 mm thick C sections exceeded the design resistance to BS 5950-5 by 10 to 40%. However, an eccentricity of 20 mm in load application reduced the failure load by 18 to 36% owing to local crushing of the C section. Tension tests on typical connections between the modules and corridors gave a failure load of 40 kN, which is adequate to transfer wind forces to a braced core and also to provide tying action in the event of loss of support to one corner of a module. Corner posts provide enhanced compression resistance but their buckling resistance is dependent on the sway stiffness of the wall panel. It is also shown that the notional horizontal force approach for steel structures presented in BS 5950-1 should be increased for modular construction.

1. INTRODUCTION

Modular construction comprises prefabricated room-sized volumetric units that are normally fully fitted out in manufacture and are installed on site as load-bearing 'building blocks'. Their primary advantages are

(*a*) economy of scale in manufacturing of multiple repeated units
(*b*) speed of installation on site
(*c*) improved quality and accuracy in manufacture.

Potentially, modular buildings can also be dismantled and reused, thereby effectively maintaining their asset value. The current range of applications of modular construction is in cellular-type buildings, such as hotels, student residences, Ministry of Defence (MoD) accommodation and social housing, where the module size is compatible with manufacturing and transportation requirements. The current application of modular construction of all types is reviewed in a recent Steel Construction Institute Publication 348 (Lawson, 2007). A paper in *The Structural Engineer* (Lawson *et al*., 2005) describes the mixed use of modules, panels and steel frames to create more adaptable building forms.

There are two generic forms of modular construction, which affect their range of application: load-bearing modules in which loads are transferred through the side walls of the modules – see Figure 1; and corner-supported modules in which loads are transferred by way of edge beams to corner posts – see Figure 2.

In the first case, the compression resistance of the walls (comprising light steel C sections generally at 300 to 600 mm spacing) is crucial. Current heights of modular buildings for this type of construction are typically limited to four to eight storeys, depending on the particular modular system and the size and spacing of the C sections used.

In the second case, the compression resistance of the corner posts is the controlling factor and for this reason, square hollow sections (SHS) are often used owing to their high buckling resistance. Building heights are limited only by the size of the SHS that may be used for a given module size (150 × 150 × 12·5 SHS is the maximum sensible size of these posts).

Figure 1. Partially open-sided module with load-bearing walls (courtesy PCKO Architects)

Figure 2. Open-sided module with corner and intermediate posts supported by a structural frame (courtesy Yorkon and Joule Engineers)

Resistance to horizontal forces, such as wind loads and robustness to accidental actions, becomes increasingly important with the scale and height of the building. The strategies employed to ensure adequate stability of modular assemblies, as a function of the building height, are

(*a*) diaphragm action of boards or bracing within the walls of the modules – suitable for four to six-storey buildings
(*b*) separate braced structure using hot-rolled steel members located in the lifts and stair area or in the end gables – suitable for six to ten storeys
(*c*) reinforced concrete or steel-plated core – suitable for taller buildings.

Modules are tied at their corners so that structurally they act together to transfer wind loads and to provide for alternative load paths in the event of one module being severely damaged. This is the scenario presented in Approved document A of the Building Regulations (HMSO, 2006), which leads to minimum tying force requirements. A recent paper (Lawson *et al.*, 2008) reviews the robustness requirements for modular construction based on a 'localisation of damage' route. Modules or load-bearing elements are removed individually to assess the ability of the rest of the assembly to support the applied loads at the accidental limit state.

For taller buildings, questions of compression resistance and overall stability require a deeper understanding of the behaviour of the light steel C sections in load-bearing walls and of the robust performance of connections between the modules. A further issue in the design of modular construction is that of installation and manufacturing tolerances, which cause eccentricities in the compression load path and also lead to additional horizontal forces applied to the modules. This is considered later in the paper in the context of design to BS 5950-1 (BSI, 2000), which is the standard for structural steelwork in buildings.

2. HIGH-RISE BUILDING FORMS USING MODULAR CONSTRUCTION

Modular construction is conventionally used for cellular buildings up to eight storeys high where the walls are load-bearing and resist shear forces owing to wind. However, there is pressure to extend this technology to high-rise buildings by using additional concrete cores or structural frames to provide stability and robustness.

One technique is to cluster modules around a core without a separate structure in which the modules are designed to resist compression and the core provides overall stability. This concept has been used on a major project called Paragon in west London, shown in Figure 3, in which the modules were constructed with load-bearing corner posts (a paper on this project was presented in *The Structural Engineer* (Cartz and Crosby, 2007).

The building form may be elongated laterally provided that wind loads can be transferred to the core. This can be achieved by using in-plane trusses placed within the corridors, or by consideration of the structural interaction between the modules and their attachment to the core. Various alternative high-rise building forms in which modules are clustered around a core are presented in Figure 4.

An adaptation of this technology is to design a 'podium' or platform structure on which the modules are placed. In this way, more open space can be provided for retail or commercial use or below-ground car parking. Support beams should align with the walls of the modules and columns are typically arranged on a 6 to 8 m grid (7·2 m is optimum for car parking), as shown in Figure 5.

For the modular system covered by the tests reported in this

Figure 3. Modular building stabilised by a concrete core (courtesy Caledonian Building Systems)

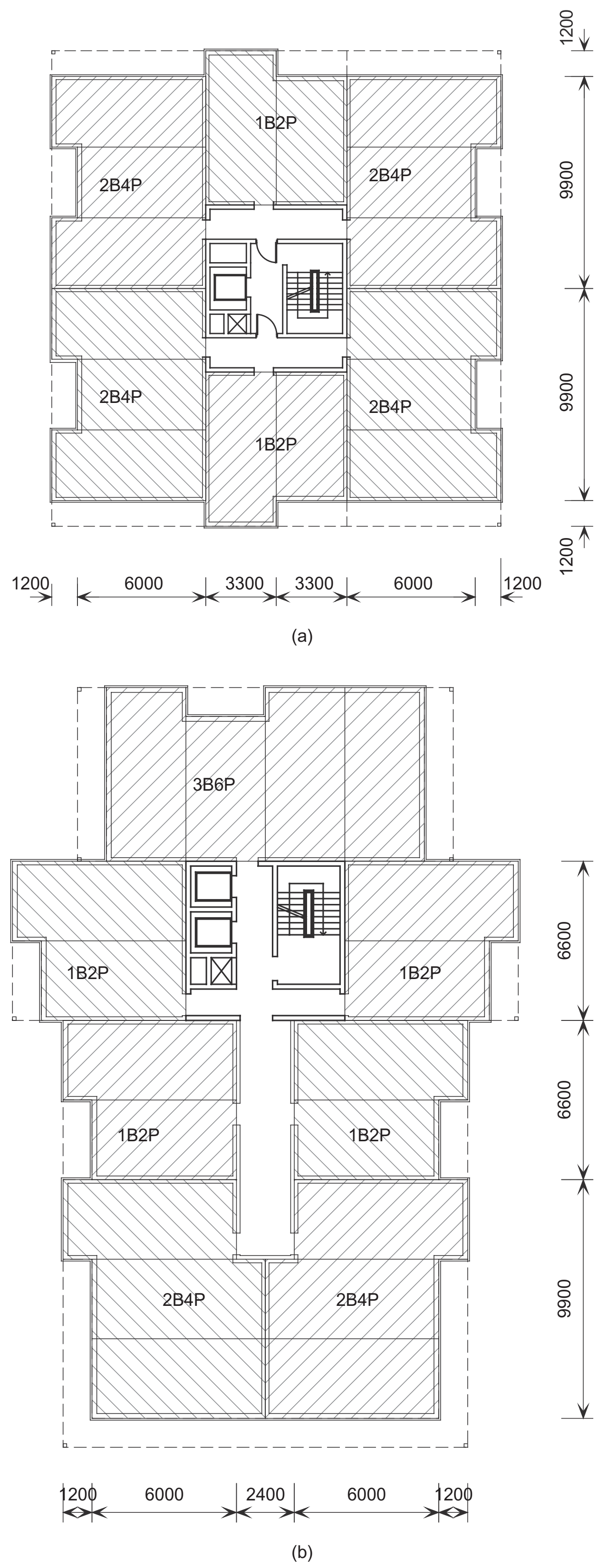

Figure 4. Typical high-rise building forms using modules and concrete cores (courtesy HTA Architects) (2B4P means a two-bedroom, four-person apartment for example): (a) option 1A; (b) option 2B

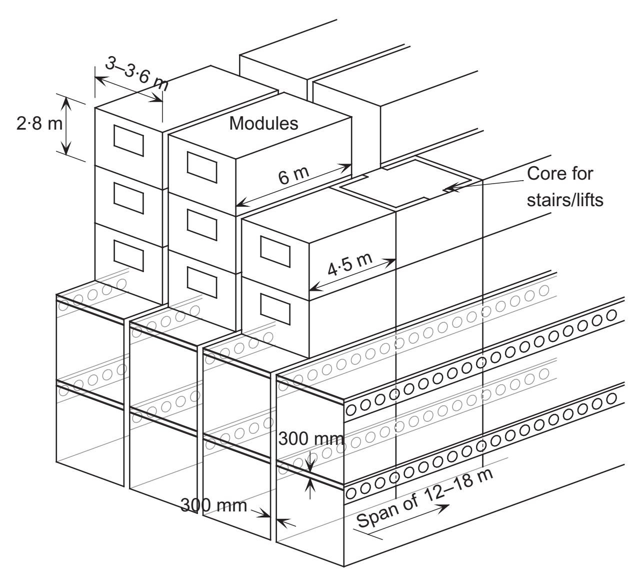

Figure 5. Modules supported by cellular beams acting as a podium

paper, three building projects have been completed to date based on the enhanced modular technology. Bond Street, Bristol is a 12-storey student residence and commercial building in which six to ten storeys of modules sit on a two-storey steel-framed podium (see Figure 6). The 400 bedroom modules are 2·7 m external width, but approximately 100 modules are combined in pairs to form 'premium' studios consisting of two rooms. The kitchen modules are 3·6 m external width. Stability is provided by four braced steel cores, into which some modules are placed (Figure 7).

A second building, Pitwines in Poole, is an eight-storey student residence comprising approximately 300 modules. Both buildings use lightweight cladding attached to the walls of the modules and comprise terracotta tiles or insulated render cladding. The nature of the student residential sector is that the construction period is often less than 12 months, and the installation of modules is generally carried out in the January to March period for a September completion. A further project using this technology has been completed in east London and another is under way in north London. This last project is shown under construction in Figure 8.

Another modular manufacturer has developed a system using

Figure 6. Twelve-storey modular student residence at Bond Street, Bristol (courtesy Unite Modular Solutions)

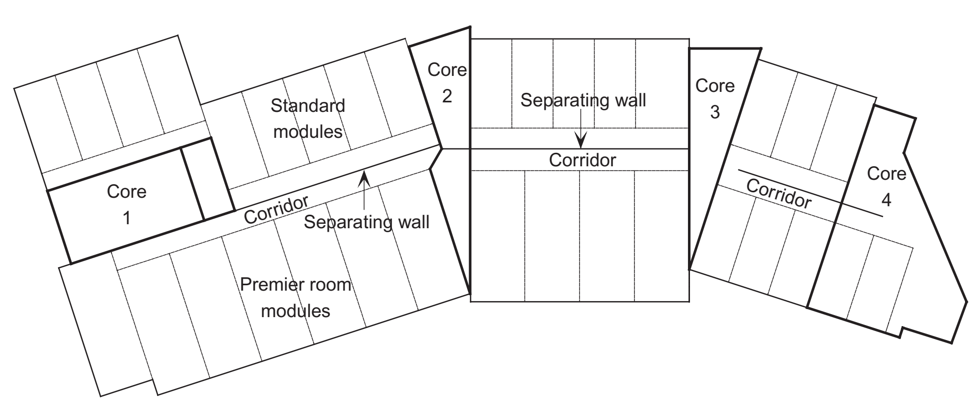

Figure 7. Plan of modular building at Bond Street, Bristol showing the core positions

Figure 8. Eleven-storey modular student residence in north London under construction (courtesy Unite Modular Solutions)

SHS corner posts and a concrete floor with perimeter parallel flange channel (PFC) steel sections. This has been used in eight to eleven residential buildings, such as the one shown in Figure 9, and construction of taller buildings is in progress. In this form of heavier modular construction, the effect of construction tolerances on the forces acting on the corner posts is much more important – see section 6.4.

Figure 9. Modular residential building, Wolverhampton (courtesy Vision Modular Structures)

3. DESIGN OF MODULAR WALLS TO BS 5950-5

Light steel walls and floors in modular construction are currently designed to BS 5950-5 (BSI, 1998), but interpretation of this standard is required to take account of the practical aspects of the constructional system. In modular systems with load-bearing walls, the light steel C sections in the walls are subject to potentially complex loading and restraint conditions. In most cases, these conditions are as outlined below.

(*a*) Axial load is transferred by way of direct wall–wall bearing, taking into account eccentricities in manufacture and installation of the modules, which causes additional build-up of moments and accentuates the local bearing stresses at the base of the wall.
(*b*) Loading from the floors and ceilings is taken as applied at the face of the wall (at an eccentricity of half the wall width), which causes additional local moments.
(*c*) Restraint is provided at the floor and ceiling positions so that the effective height of the wall may be taken as its clear internal height.
(*d*) Two layers of plasterboard or similar boards are attached to the internal face of the wall by screws at not more than 300 mm spacing and provide up to 90 min fire resistance.
(*e*) Cement particle board (CPB) or oriented strand board (OSB) are often attached to the exterior of the wall for weather-tightness of the module and to provide some diaphragm action. In production, boards may be fixed air-driven pins enhanced by glued joints.
(*f*) In taller modular buildings, second-order ($\boldsymbol{P}$–Δ) effects may occur owing to sway and other eccentricities that are often neglected in the design of low-rise buildings

The effects of axial loading and eccentricity can be taken into account in the design of compression members to BS 5950-5 (BSI, 1998), but the stabilising effect of the boards on local and overall buckling is largely unquantified. It is reasonable to assume that boards fixed on both sides provide restraint in the minor axis direction of the C section, but the stiffening effect of the boards in the major axis (out-of-plane) direction is not known, nor is the stabilising effect of boards attached only on one side. This is the subject of the test programme described below.

4. COMPRESSION TESTS ON MODULAR WALLS

The following tests were carried out to verify the structural action of the load-bearing walls in a typical modular system. Two series of tests were carried out: one series on 75 mm deep × 45 mm wide × 1·6 mm thick C sections at the Building Research Establishment (BRE) and one series on 100 mm deep × 42 mm wide × 1·6 mm thick C sections at the University of Surrey.

The tests were intended to establish the compression resistance of the C sections, nominally placed at 300 mm spacing, taking account of the restraining and stiffening effects of various types of board. The sensitivity to eccentricities up to 20 mm was also investigated, as this exceeds the maximum that may be envisaged with good control on installation of modules in practice.

The panels were loaded using a spreader beam and lateral restraints in the form of PFC sections, and the test arrangement is illustrated in Figure 10. The eccentricity in load application was introduced by a 6 mm thick steel plate inserted at the base of the wall.

The main variables were the type of boards that are attached on one or both sides and the eccentricity in axial load. Additional tests were included on taller walls to examine the influence of slenderness. The boards were fixed using 2 mm diameter air-driven nails at 200 mm centres, as used in production of the wall panels. The boards were attached 2 mm short of the web of the top and bottom track so that the boards were not loaded directly.

OSB was attached externally and, in some tests, CPB was included to assess the difference in restraint provided by the two types of board. Two layers of 15 mm fire-resistant plasterboard were used internally, as required for 90 min fire resistance. In two of the tests, this plasterboard was omitted.

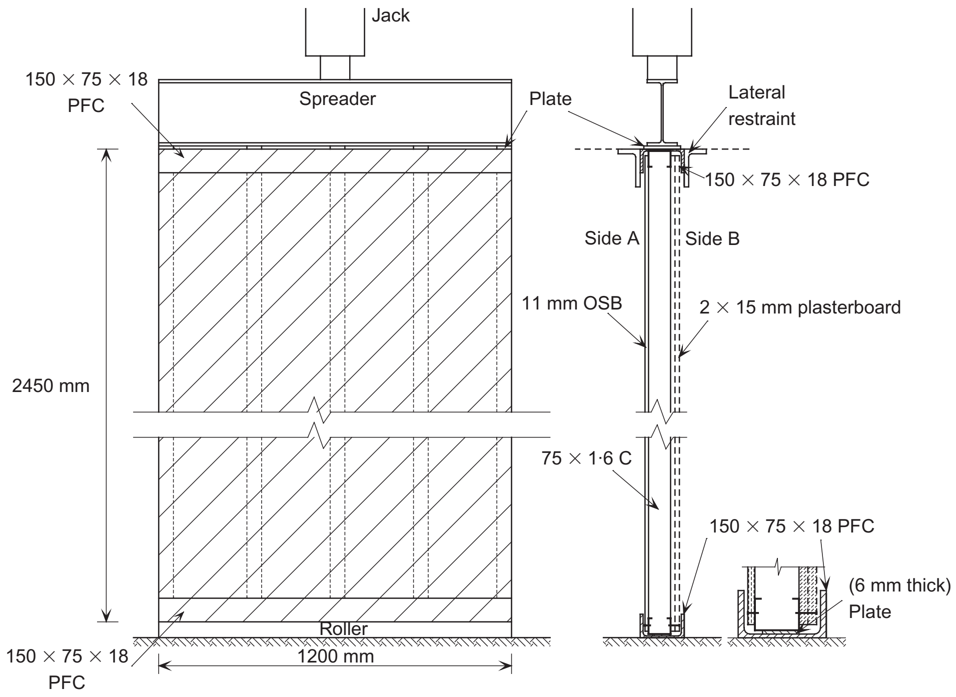

Figure 10. Test arrangement for BRE wall compression tests

The walls were first loaded up to around 100 kN to represent serviceability loading before loading incrementally to failure. Deflections were recorded at the top of the wall (vertically and horizontally) and at mid-height (horizontally). The test failure loads are presented in Table 1. The failure load generally occurred at a relatively small vertical displacement of less than 5 mm.

A further series of tests was carried out on 2300 mm high × 600 mm wide wall panels, comprising three 100 mm deep C sections with a mid-height noggin built into the wall panel to provide lateral restraint in the minor axis.

Wall test details	Wall height: m	Eccentricity of loading: mm	Failure load per C section: kN	Design resistance to BS 5950-5: kN	Model factor
75 × 45 × 1·6C: OSB boards on one side only	2·45	0	64	48	1·33
75 × 45 × 1·6C: Plasterboard on one side,	2·45	0	97	76 (inc. effect of boards)	1·27
OSB on the other	2·77	0	90	56 (inc. effect of boards)	1·61
	2·45	10	79	56 (inc. effect of boards)	1·41
	2·45	20	62	47 (crushing)	1·31
75 × 45 × 1·6C: Plasterboard on one side, CPB	2·45	0	96	76 (inc. effect of boards)	1·26
on the other	2·45	20	52	47 (crushing)	1·10
100 × 42 × 1·6C: Plasterboard on one side only	2·30	0	57	51	1·13
100 × 42 × 1·6C: CPB on one side only	2·30	0	70	61	1·14

Model factor = Failure load/design resistance

Table 1. Failure loads of C section wall studs and comparison with BS 5950-5

Two additional bending tests were carried out on wall panels using 75 × 1·6 C sections subject to a line load at mid-span. The purpose was to calculate the effective stiffness of the wall panels in order to calculate the modified slenderness of the C section for the compression resistance to major axis buckling. The cases considered were

(*a*) OSB board on one side and two layers of plasterboard on the other (OSB in compression)
(*b*) OSB board on one side with no plasterboard on the other (OSB in compression).

The measured values of I_{eff} taking into account the stiffening effects of composite action with the boards were 432×10^3 mm^4 and 270×10^3 mm^4 per C section respectively. The calculated second moment of area of the bare C section was 265×10^3 mm^4. It follows that the effective inertia is increased by 62% for boards fixed on both sides but by only 2% for OSB board on one side.

5. ANALYSIS OF WALL TESTS TO BS 5950-5

The light steel walls were analysed in accordance with BS 5950-5 using measured section dimensions and steel strengths. Composite action occurred owing to the additional stiffness of the boards attached to both sides, which increase the buckling resistance of the wall. The section properties of the C sections were calculated for the case where the edge lips are considered to be fully effective.

The strip steel was S350 grade supplied to BS EN 10327 (BSI, 2004b) and measured strengths were in the range 380–405 N/mm^2. Calculated compression resistances to BS 5950-5 are presented in Table 1. The model factor is the ratio of the test failure load to the compression resistance to BS 5950-5, based on measured material strengths and geometry.

The attachment of boards to both sides of the wall effectively prevents minor axis buckling, even for the narrow wall panels tested and so failure may occur in one of three modes

(*a*) crushing of the cross-section locally in compression, as in Figure 11
(*b*) buckling of the wall in the major axis direction, as in Figure 12
(*c*) delamination of the boards from the wall studs, leading to loss of composite action.

The stiffening effect of the boards leads to a reduction in slenderness and increase in buckling resistance. Using the measured 62% increase in bending stiffness of the wall panel, the effective slenderness of the bare C section is reduced by 22% owing to attachment of the OSB and plasterboards. For a 2·45 m wall panel, the slenderness in the major axis direction was 79, and so the effective slenderness becomes $0{\cdot}78 \times 79 = 62$. This leads to a buckling strength of $p_c = 263$ N/mm^2 according to Table 10 of BS 5950-5 when using a Q factor (effective area/gross area) of 0·88.

The calculated compression resistance was 67 kN, which is approximately 70% of the test result of 97 kN. This suggests that the buckling curve for cold-formed sections used in BS 5950-5 is conservative. In addition, local buckling of the

Figure 11. Local crushing of C section in compression tests

Figure 12. Wall failure by overall buckling in pure compression

flanges of C section may be reduced by the attachment of the boards, which increases the effectiveness of the cross-section.

The eccentricity of load application using a plate below the wall accentuates local crushing, as well as overall buckling. The crushing resistance of the C section without consideration of buckling is calculated from $A_{\mathrm{eff}}\, \boldsymbol{p}_{\mathrm{y}}$. The reduced crushing resistance owing to eccentric loading may be taken into account by considering a reduced compression area, A_{eff}. Because the buckling resistance is approximately 70% of the crushing resistance, it follows that buckling will occur first unless the crushing resistance is reduced by over 30%.

A 10 mm eccentricity caused a 19% reduction in load capacity and a 20 mm eccentricity caused a 36% reduction in capacity. However, in the tests, a 10 mm eccentricity did not reduce the failure load below the theoretical buckling capacity.

The second series of tests on walls used 100 × 1·6C sections with boards on one side only. These tests showed that minor axis buckling is prevented by fixing to plasterboard for 1·6 mm thick steel, but the increase in compression resistance relative to BS 5950-5 was less than for the 75 mm deep sections. This is attributable to the lower transverse bending stiffness of the web of the deeper C section, which means that the unsupported flange is only partially restrained.

6. STRUCTURAL ACTION OF GROUPS OF MODULES

The structural behaviour of an assembly of modules is complex because of the influence of the tolerances that are implicit in the installation procedure, the multiple interconnections between the modules and the way in which forces are transferred to the stabilising elements such as vertical bracing or core walls. The key factors to be taken into account in the design of high-rise modular buildings are

(*a*) the influence of initial eccentricities and construction tolerances on the additional forces and moments in the walls of the modules
(*b*) application of the design standard for steelwork, BS 5950-1 to modular technology, using the notional horizontal load approach
(*c*) second-order effects due to sway stability of the group of modules, especially in the design of the corner columns
(*d*) mechanism of force transfer of horizontal loads to the stabilising system, for example concrete cores
(*e*) robustness to accidental actions (also known as structural integrity) for modular systems.

These aspects are now discussed in turn.

6.1. Influence of constructional tolerances

The National Structural Steelwork Specification for Building Construction (NSSS) (BCSA, 2007) presents the permitted tolerances of steel frames, in which the maximum out-of-verticality of a single column is $\delta_{\mathrm{H}} \leqslant$ height/600, but $\leqslant$ 5 mm per storey. Furthermore, for steel-framed buildings of more than ten storeys high, the maximum out of verticality over the total building height is limited to 50 mm in the NSSS.

BS EN 1090-2 (BSI, 2008) concerns the execution of structures and in it, the essential tolerances define the maximum deviations that are permitted so as not to impair the overall performance of a structure or member. BS EN 1090-2 Table D.1.12, referring to multi-storey frames, differs from the NSSS in that the cumulative error over n floors each of height h is given by $h\sqrt{n}/300$. It follows that the permitted cumulative deviation over n storeys is $10\sqrt{n}$ mm (for $h = 3$ m) to BS EN 1090-2.

These permitted deviations for steel frames may not, however, reflect the practicalities involved in modular construction because of the difficulties in precisely positioning one module on another and in making suitable connections. For a single module placed on another module, it is proposed that the maximum out of alignment during installation may be taken as 12 mm in orthogonal plan directions relative to the top of the module below. This alignment requires careful control on site, especially in windy conditions.

For a vertical stack of modules, the cumulative positional error, e, owing to installation can be partially corrected over the building height. Using the same logic as in BS EN 1090-2, the cumulative positional tolerance (in millimetres) may be taken statistically as $e \leqslant 12\sqrt{n}$, where n is the number of modules in a vertical group. Typically, for $n = 10$, the total cumulative positional tolerance that is permitted becomes approximately 40 mm, but this neglects the geometric tolerances in the module manufacture.

An alternative simplified procedure that is easier to control on site is to limit the cumulative positional tolerance to 5 mm per module in orthogonal directions with a maximum of 50 mm (for $n = 10$), which is similar to the NSSS. However, it is considered that the maximum positional error of one module on another may be taken as 12 mm (except at ground level where a maximum of 5 mm should be achievable). This means that at the first floor, the cumulative tolerance of 10 mm will control, even if the first-floor module is 12 mm out of position relative to the base module and the base module is positioned at $\leqslant -2$ mm from datum.

Added to this positional error is the possibility of a systematic manufacturing error in the geometry of the modules. For a single module, the maximum permitted tolerance in geometry may be taken as illustrated in Figure 13. However, over a large number of modules, the average error in manufacture may be taken as half of the permitted tolerance for a single module. Therefore, the out of verticality of the corner posts may be taken as $h/1000$, where h is the module height (typically 3 m).

To take account of manufacturing tolerances, the cumulative out of verticality over the building height may be taken as $nh/1000$, or approximately $3n$ mm. The total permitted out-of-verticality δ_{H} over the building height, consisting of a stack of n modules vertically, is therefore a combination of positional and geometric tolerances, approximately as follows

$$\delta_{\mathrm{H}} \leqslant e + nh/1000 = 5n + 3n = 8n \quad \mathrm{mm} \tag{1}$$

Using this simplified formula, it follows that $\delta_{\mathrm{H}} = 80$ mm for

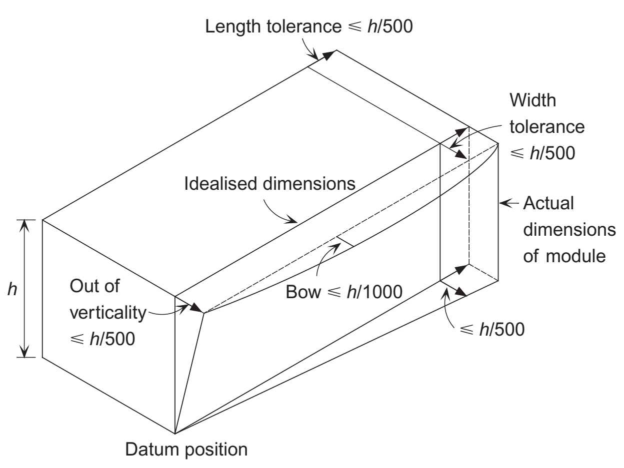

Figure 13. Permitted maximum geometric errors in manufacture of modules

$n = 10$ storeys, which is equivalent to approximately $h/350$ per floor. This is 60% higher than the tolerance permitted for structural steelwork and reflects the different installation and connection methods between structural frames and a group of modules.

It is recommended that the absolute out of verticality in modular construction is limited to a maximum of 80 mm relative to a ground datum, which will control for buildings of ten or more storeys. This is achievable with good control on installation. Adjustments in module position should be made gradually rather than at a few positions, which would otherwise add to local eccentricities. These adjustments can be made by varying the cavity spacing between the modules. In detailing, the cavity width should be at least equal to half of the expected maximum tolerance, or as a simple rule, taken as a minimum of 40 mm.

6.2. Application of notional horizontal forces in modular construction

A way of assessing the sway stability of a group of modules is by using the notional horizontal force approach given in clause 2.4.2.3 of BS 5950-1. For steel frames, this horizontal force corresponds to 0·5% of the factored vertical load acting per floor, and is used in the absence of wind loading. It represents the minimum horizontal force that is used to assess the sway stability of a frame. A further limit for the combination of wind and vertical load is that the wind load should not be less than 1% of the factored dead load acting horizontally. This may control where the self-weight exceeds the imposed loading on a floor.

BS EN 1993-1-1 Eurocode 3 clause 5.3.2 (BSI, 2004a) permits an out-of-verticality of $L/200$ for a single column, but this is reduced by a factor of 2/3 when considering the average over a number of storeys (i.e. $\delta_{\mathrm{H}} \leq L/300$). The permitted out of verticality of a whole structure is obtained by multiplying this value for a single column by a factor of $\{[0{\cdot}5\ [1 + (1/m)]\}^{0{\cdot}5}$ for m columns in a group horizontally. The result tends to $\delta_{\mathrm{H}} \leq L/420$, which is higher than in the NSSS, but further requirement in the approach of Eurocode 3 is that this out of verticality is considered in combination with wind loading rather than as an alternative load case, as in BS 5950-1.

The combined eccentricity on a vertical assembly of modules takes into account the effects of eccentricities of one module placed on another, and the reducing compression forces on the walls acting at the increased eccentricity with height. This effect is illustrated in Figure 14. The walls of the module are unable to resist high moments owing to these effects and so the equivalent horizontal forces required for equilibrium are transferred as shear forces in the ceiling, floors and end walls of the modules. The total additional moment acting on the base module is therefore given by an effective eccentricity Δ_{eff} multiplied by the compression force in the base module, as follows

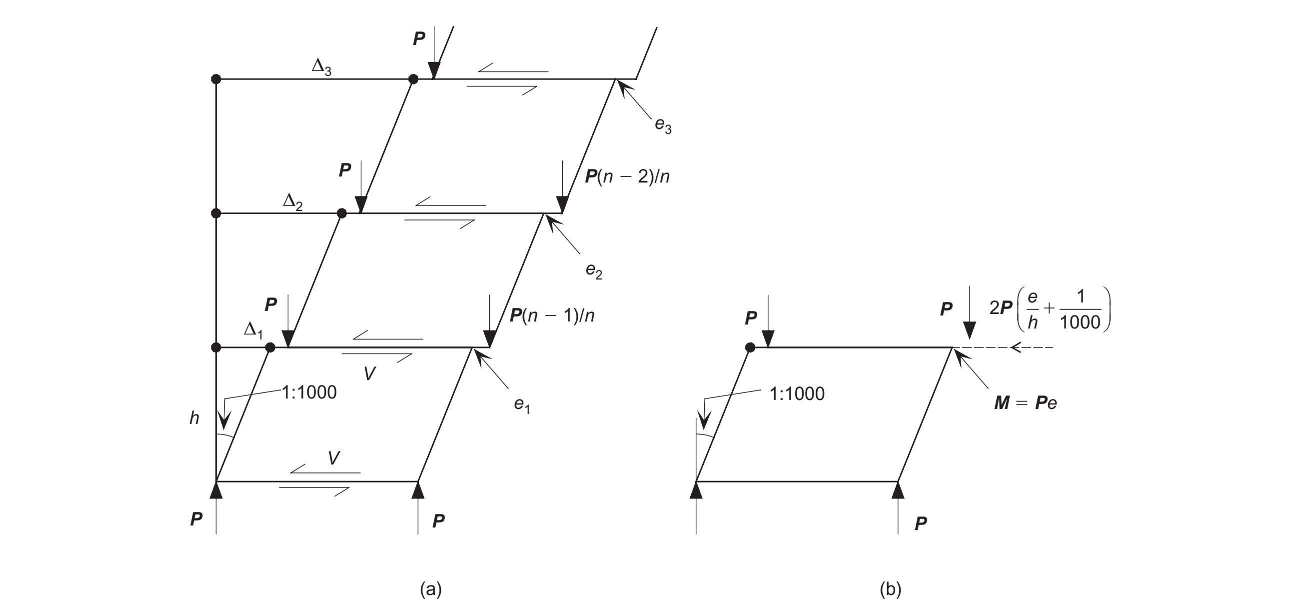

Figure 14. Combined eccentricities acting on the ground-floor modules: (a) shear in end walls due to eccentric loading for a four-sided module; (b) transfer of eccentric loading to stabilising system for corner-supported modules

2	$$\begin{aligned} M_{\text{add}} &= P_{\text{wall}}\Delta_{\text{eff}} \\ &= P_{\text{wall}} \left[\frac{(n-1)}{n} + 2\frac{(n-2)}{n} + 3\frac{(n-3)}{n} \dots + \frac{1}{n}\right] \\ &\quad \times (e + h/1000) \end{aligned}$$

where P_{wall} is the compression force at the base of the ground-floor module $= nW_u$, n is the number of modules in a vertical assembly, e is the average positional eccentricity per module, h is the module height (in mm), and W_u is the factored load acting on each module.

As a good approximation, it is found that the following formula holds for the effective eccentricity of the vertical stack of modules as a function of n:

3	$$\Delta_{\text{eff}} = \left[\frac{n-1}{6}\right] 8n$$

The effective base eccentricities are presented in Table 2 for $n = 6$ to 12 storeys and for a module height, $h = 3$ m. This eccentricity may be converted to a notional horizontal force applied at each floor level, which is expressed as a percentage of the vertical load acting at each floor level, and is defined as the force which causes the same equivalent moment in the base module as the effective eccentricity in Equation 2. This moment is given by

4	$$kW_u n^2 h/2 = P_{\text{wall}}\Delta_{\text{eff}} = nW_u\Delta_{\text{eff}}$$

where k is the proportion of the factored load acting on each floor, and so

5	$$k = \left[\frac{8(n-1)}{3h}\right] \quad \text{or} \quad \left[\frac{n-1}{3n}\right]\left[\frac{80}{h}\right] \quad \text{for } n > 10$$

From Table 2, and using the tolerances defined above, it is calculated that the notional horizontal force varies from 0·5% to 0·9%, when expressed as a percentage of the vertical load applied to the module. It should be noted that $k = 0{\cdot}5\%$, when the maximum tolerance is 50 mm, which agrees with BS 5950-1.

For modular construction, it is therefore recommended that the notional horizontal force is taken as a minimum of 1% of the factored vertical load acting on each module, which reflects the higher tolerances that are permitted in modular construction. It is used as the minimum horizontal load in assessing overall sway stability of the structure, and it is proposed that it is used in combination with wind forces.

As an example, for a module of 25 m^2 floor area subject to factored loading of 7 kN/m^2, the notional horizontal force acting in orthogonal directions is approximately 2 kN. For a vertical stack of ten modules, the base shear is therefore 20 kN. This shear force may be shared between the two walls of the module in the direction under consideration. The notional force may be compared with a wind load of up to 10 times this magnitude acting as a shear on the longitudinal side façade of the building, and so is still relatively small. The notional horizontal force may, however, control when there are less than 10 modules in a horizontal group.

If the modules are unable to resist the horizontal force required for overall stability, the forces must be combined for a number of modules on plan at each level and transferred to the stabilising system. This may be the case for partially open-sided modules.

6.3. Forces at module interconnections

The structural interactions within a group of modules are complex. Horizontal forces may be transferred by tension and compression forces in the ties at the corners of the modules by utilising the diaphragm action of the base and ceiling of the modules. Shear forces may be transferred through the continuous corridor members rather than the corner connections because of the potential articulation through the bolts and connecting plates between the modules. These actions are illustrated in Figure 15.

Where the corridor floor is used to transfer shear forces, the connection of the modules to the corridor may be made by a detail of the form of Figure 16. The extended plate is screw fixed on site to the corridor members and is bolted to the re-entrant corners between the modules so that it also acts as a tie plate. This detail is not used to provide vertical support to the corridor floor, which is supported on continuous angles attached to corridor wall of the modules.

The forces in the tie connection in Figure 16 may be calculated on the basis of the wind forces acting on the module. The highest force occurs for an external pressure coefficient of +0·85 and a negative internal pressure of −0·3. The wind force on one module is divided between two module-to-corridor connections. For a wind pressure of 1·2 kN/m^2, the force in this connection is approximately 8 kN at working loads.

The shear attachment to the core is made both through the corridor and also at the module adjacent to the core. This

Number of modules	Approx. building height: m	Cumulative out-of-verticality at top of building: mm	Effective eccentricity on base module – Simplified formula in Equation 3: mm	Notional horizontal force Equation 5: %
$n = 6$	16	48	$5/6 \times 48 = 40$	0·5
$n = 8$	22	64	$7/6 \times 64 = 75$	0·7
$n = 10$	27	80	$9/6 \times 80 = 120$	0·9
$n = 12$	33	80	$11/6 \times 80 = 147$	0·9

Table 2. Summary of effective eccentricities and notional horizontal forces in modular construction as a function of building height

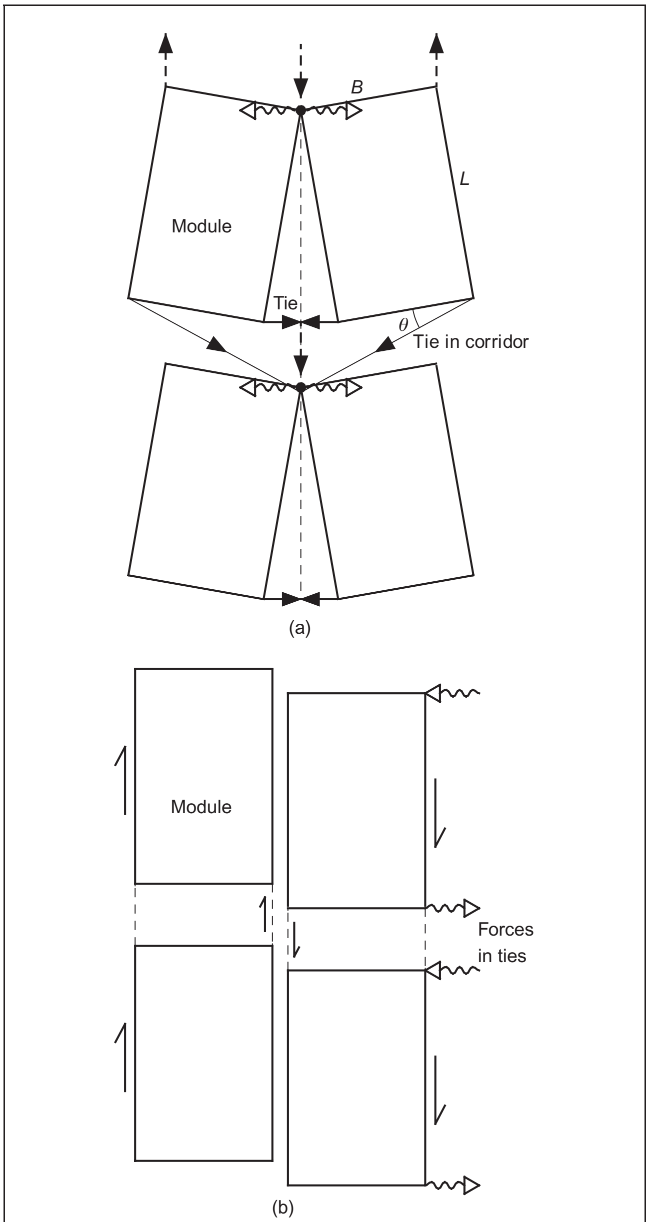

Figure 15. Force transfer between modules: (a) force transfer at corridor – bending action; (b) force transfer at corridor – pure shear

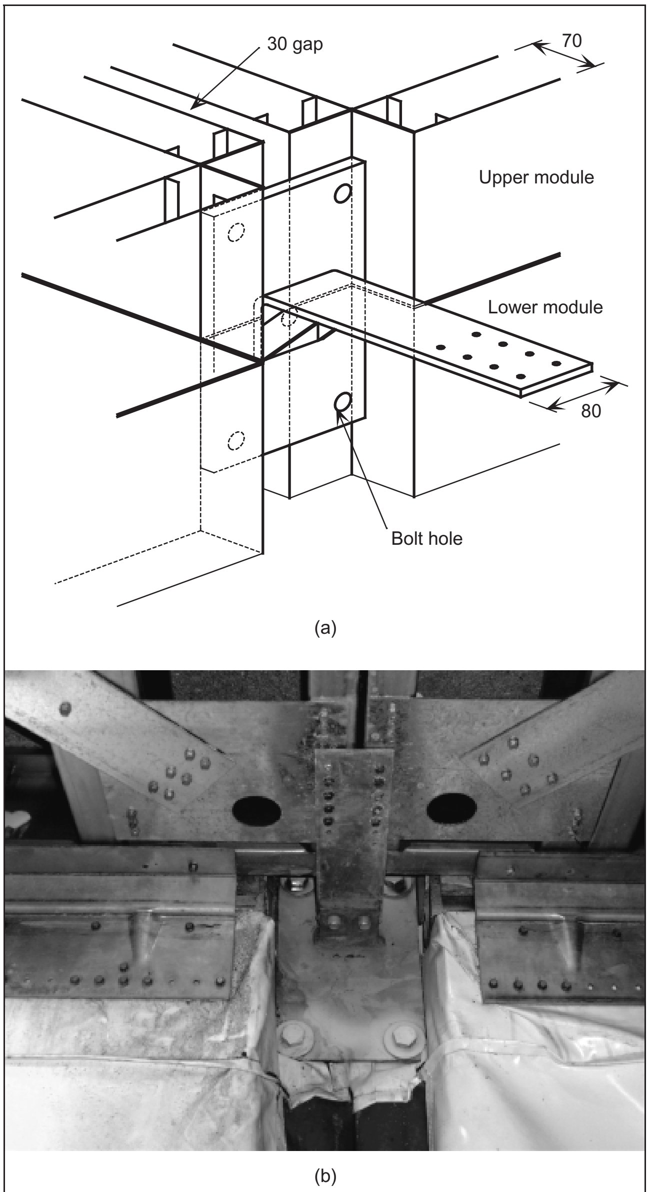

Figure 16. Connection detail between the corridor cassette and modules: (a) sketch detail; (b) actual detail

connection force at the core is designed for the aggregate of the module–corridor connection forces, which for a group of three to four modules is 24 to 32 kN (factored loading) or 18 to 24 kN as a working load.

6.4. Stability of corner posts in modular construction

Corner posts add to the compressive resistance of a wall and, if they are included in the module, it is normal practice to assume that all the applied vertical loads acting on the module are resisted by the corner posts. These posts are usually in the form of steel angle sections for low-rise applications, or SHSs for taller buildings. The posts are effectively restrained from buckling by the in-plane stiffness of the walls of the modules to which they are connected, but this assumption may not be valid for partially open-sided modules or for highly perforated walls.

Consider the stability of the corner posts of a module when restrained only by the in-plane stiffness of the walls of the module, as illustrated in Figure 17. The posts are discontinuous at the module–module connections and do not contribute to the sway stiffness of the structure, but are restrained against buckling in their height between the connection points.

The initial out of verticality of the corner post increases under an axial load, $\boldsymbol{P}$, in each post, which may be approximated by strut buckling theory, according to

$$\delta = \frac{\delta_o}{1 - (2\boldsymbol{P}/\boldsymbol{P}_{crit})} \qquad (6)$$

where $\boldsymbol{P}$ is the axial compression acting on one post; δ_o is the initial out of verticality and eccentricity of the corner post; $\boldsymbol{P}_{crit}$ is the critical buckling resistance for sway stability of the module.

From this simple shear failure mechanism, the work done in

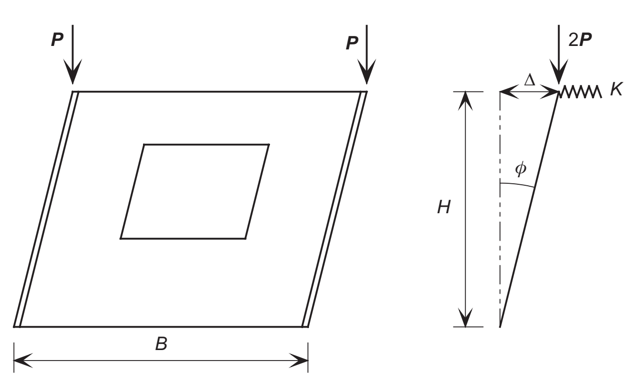

Figure 17. Sway stability of the wall of a module for corner posts in compression

shear and compression may be equated, in order to determine the critical buckling load, $\boldsymbol{P}_{\text{crit}}$, as follows

$$\frac{k\Delta^2}{2} = \frac{(2\boldsymbol{P}_{\text{crit}})\Delta^2}{2h} \quad \text{or } \boldsymbol{P}_{\text{crit}} = 0{\cdot}5kh \tag{7}$$

where k is the shear stiffness of the wall panel.

As $\boldsymbol{P}$ approaches $\boldsymbol{P}_{\text{crit}}$, so the shear deflection of the wall panel increases rapidly. Therefore, it is necessary to keep $\boldsymbol{P}$ well below $\boldsymbol{P}_{\text{crit}}$ to avoid instability effects. The eccentricity of load causes both bending in the post and shear in the wall panel.

The shear stiffness of the wall can be estimated from shear diaphragm tests and corresponds to the horizontal load at a serviceability deflection of $h/500$, where h is the module height in millimetres. This is achieved for a shear force of typically

(*a*) 10 kN for a 2·4 m wide wall panel with a window, or approximately 4 kN/m width
(*b*) 20 kN for a 2·4 m wide unperforated panel, or approximately 8 kN/m width.

In the case of a module with $h = 3$ m and width of $b = 3{\cdot}6$ m, it follows that the typical shear stiffness of an end wall panel with a window becomes

$$k = \frac{4 \times 3{\cdot}6 \times 500}{3{\cdot}0} = 2400 \text{ kN/m}$$

Inserting this value of k in Equation 7 leads to a critical buckling load owing to shear in the end wall of a module of

$$\boldsymbol{P}_{\text{crit}} = 0{\cdot}5 \times 2400 \times 3 = 3600 \text{ kN}$$

To check the stability of the corner post, it is recommended that the eccentricity in load application is taken as the maximum positional eccentricity of 12 mm when one module is placed on another plus the maximum out of verticality in manufacture of a single module (or $h/500$, as shown in Figure 13). For a 3 m high module, $\delta_o = 12 + 6 = 18$ mm. These eccentricities are illustrated in Figure 18.

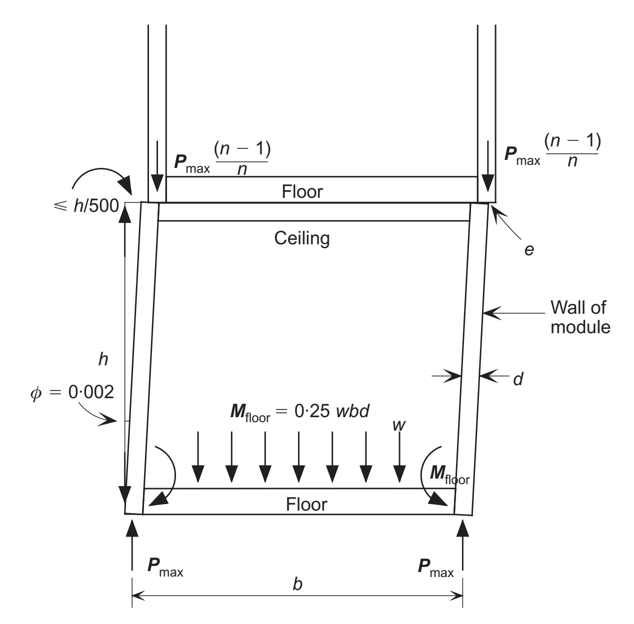

Figure 18. Illustration of eccentricity of forces applied to the walls or corner posts of a module

In addition, a local moment is transferred from the floor or edge beam, which may act in the same sense as the positional eccentricity. For a floor–wall junction, this shear load may be assumed to act at the face of the wall studs (or a minimum of 50 mm). For a corner post, the shear load acts at the centre of the bolt group, and a minimum eccentricity of 75 mm from the centre of the post may be used. This local moment acts only on individual modules and is not cumulative.

The additional moment acting on a corner post is calculated from $\boldsymbol{M} = \boldsymbol{P}\delta$, where δ is given by Equation 6. For a wall, the effective eccentricity also includes the bow in the wall between the corners (or $h/1000$, as shown in Figure 14).

For a corner post, the effective eccentricity is therefore given by $\delta_o = 18 + 75/n$ mm. For a load-bearing wall, the effective eccentricity is given by $\delta_o = 21 + 50/n$ mm. For $n = 10$, the effective eccentricities become approximately 25 mm in both cases.

The stability of a corner post is then checked as

$$\boldsymbol{P}/\boldsymbol{P}_{\text{c}} + \boldsymbol{M}/\boldsymbol{M}_{\text{c}} \leq 1{\cdot}0 \tag{8}$$

where $\boldsymbol{P}$ is the load acting at the top of the base module and $\boldsymbol{P}_{\text{c}}$ is the compression resistance of the post.

When the post is restrained against buckling in its height by attachment to the adjacent walls, then the bending resistance may be taken as $\boldsymbol{M}_{\text{c}} = \boldsymbol{M}_{\text{el}}$, where $\boldsymbol{M}_{\text{el}}$ is the elastic bending resistance of the post. Elastic properties should be used in order to take account of uncertainties in this simple linear interaction method in Equation 8.

For an unsupported post (not restrained by the walls), the compression resistance is given by $\boldsymbol{P}_{\text{c}} = \boldsymbol{p}_{\text{c}}\text{A}$, where $\boldsymbol{p}_{\text{c}}$ is calculated from the minor axis slenderness of the post and $\boldsymbol{M}_{\text{c}}$

is the bending resistance for lateral torsional buckling. The interaction equation is also modified to take into account bending in two directions, as in BS 5950-1.

As an example for a 7·2 m long × 3·6 m wide module, with a factored floor load of 7 kN/m², the compression force acting at the top corner of the ground-floor module in a 12-storey building is approximately

$$P = 7 \times 7{\cdot}2 \times 3{\cdot}6 \times (12 - 1)/4 = 499 \text{ kN}$$

Check the compression resistance of the corner posts using 100 × 100 × 10 SHS (in S355 steel), which are stabilised by the walls of the modules: crushing resistance, $P_y = 1239$ kN, and bending resistance, $M_{el} = 32{\cdot}8$ kN m.

The out-of-plane displacement and its associated moment, M, are obtained from Equation 6

$$\delta_o = 18 + 75/n = 25 \text{ mm}$$

$$\delta = \frac{25}{1 - (2 \times 499/3600)} = 34 \text{ mm}$$

$$M = 499 \times 0{\cdot}034 = 17{\cdot}0 \text{ kN m}$$

Using the linear combination of axial force and moment for member stability

$$P/P_c + M/M_{el} = 499/1239 + 17{\cdot}0/32{\cdot}8$$
$$= 0{\cdot}40 + 0{\cdot}52 = 0{\cdot}92 < 1{\cdot}0$$

It follows that the effect of eccentricity in installation and out of verticality in manufacture is to reduce the compressive resistance of a corner post by about 60%. It is also recommended that for simple design, the effective eccentricity of load acting on the corner post is taken as not less than 35 mm, which allows for a 40% magnification in sway from the initial eccentricity of 25 mm.

6.5. Robustness to accidental damage

The ability of an assembly of modules to resist applied loads in the event of serious damage to a module at a lower level is dependent on the development of tie forces at the corners of the modules. The loading at this so-called accidental limit state is taken as the self-weight plus one third of the imposed load all multiplied by a partial factor of safety of 1·05 to BS 5950-1.

To satisfy 'robustness' in the event of accidental damage to one of the modules, the tie forces between the adjacent modules may be established on the basis of a cantilever model, as presented in a recent paper (Lawson *et al.*, 2008). Assuming that the worst case corresponds to loss of support to one side of a corner module and that each module above is able to develop tying forces equally, the tension force in the ties is given as follows

$$T = \left[\frac{W_a b}{4h}\right] \quad (9)$$

where W_a is the load acting on the module at the accidental limit state, and b and h are the dimensions of narrow end of the module.

Figure 19 shows the results of a finite-element analysis of a module when one corner support is removed, which is a more likely case than complete removal of one side wall. The applied load is taken as 10 kN/m per wall for a heavyweight module using the partial factors noted above. Torsional stiffness of the module is developed by diaphragm action of the walls and floor/ceiling. From this analysis, the maximum horizontal tying force is equal to 26% of the total load applied to the module (rather than 48% in the cantilever formula) and the maximum vertical load is approximately 40% of the total load. It is concluded that the minimum values of the horizontal tying force, T, may be taken as 30 kN for lightweight modules (self-weight < 3·5 kN/m²) or 50 kN for heavyweight modules (self-weight < 6 kN/m²).

6.6. Module connection tests

As part of the development programme for the modular supplier, tests on complete modules were carried out at the BRE to assess the tensile resistance of the tie detail between the corridor cassette and the corner of the module. The tie connection is made at the re-entrant corner of the module.

The module was held in place at two corners and a tensile force was applied at the top opposite corner causing pull-out of the connecting bolt to the 4 mm thick corner angle manufactured as part of the module. Forces within the module are transferred by way of in-plane diaphragm action of the ceiling and walls. A rigid corner gusset plate was attached across the junction between the bottom track and the end wall stud, and the tension force reached of 40 kN at failure corresponding to a displacement of 10 mm. The gusset detail at a load level of 25 kN is shown in Figure 20. The load–deflection graph for this test is shown in Figure 21.

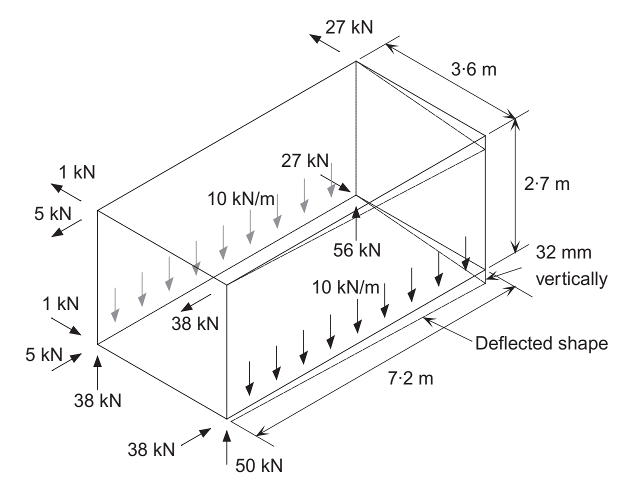

Figure 19. Illustration of tie forces when support to one corner of a module is removed

Figure 20. Tensile test on module with stiffening plate

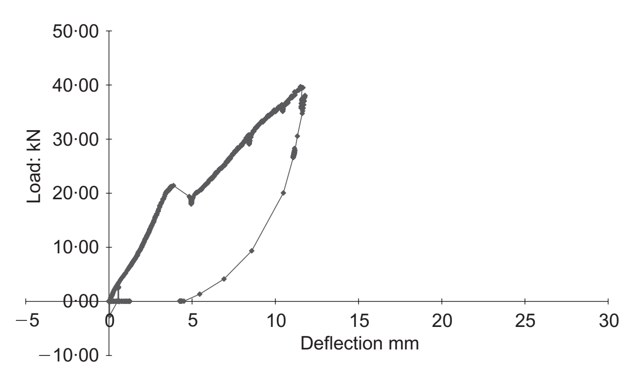

Figure 21. Load–displacement results for module test with stiffening plate. Unite module corner test 7

The test using a stiffening plate at the corner of the module showed that this arrangement offers the best solution for the module-to-corridor connection. The characteristic resistance of this connection is taken as 20% less than the failure load of a single test, or $0{\cdot}8 \times 40 = 36$ kN, which exceeds the calculated load of 24 kN for transfer of wind forces across three modules to an adjacent core.

7. CONCLUSIONS

This paper presents the results of tests on light steel walls in compression, which are used to demonstrate the extension of modular construction up to 12 storeys high. The tests showed that the stiffening effect of the fascia boards is very high and that the compression resistance of the C sections is increased in comparison to the bare steel section. These conclusions refer to internal wall heights of 2·3 to 2·8 m using 75 mm to 100 mm deep C sections.

(*a*) Minor axis buckling is effectively prevented by attachment of various types of boards on one side only, provided the steel thickness is not less than 1·6 mm.
(*b*) The test load capacities exceeded the design resistance to BS 5950-5 by 10 to 40% due to the stiffening effects of the attached boards.
(*c*) The effective bending stiffness of the bare steel sections is increased by up to 62% due to the attachment of OSB and CPB boards on both sides.
(*d*) The effect of 10 mm out-of-plane eccentricity in load application reduces the failure load by 19%, and the effect of 20 mm out-of-plane eccentricity accentuates local crushing and reduces the failure load by 18 to 36%.

The tests on the module-to-module connections showed that a tying force of 40 kN can be resisted. For robustness to accidental actions, the minimum tying force between modules should be taken as 30 kN for lightweight modules (self-weight $< 3{\cdot}5$ kN/m^2) and 50 kN for heavyweight modules.

The effect of installation and geometric inaccuracies must be taken into account in the design of modular buildings. It is proposed that the maximum positional error is 12 mm for one module placed on another. When combined with manufacturing tolerances, it is proposed that the maximum out of verticality should not exceed 8 mm per module in a vertical group (or an absolute maximum of 80 mm) relative to ground datum. Using these tolerances, the notional horizontal force used to evaluate stability of a group of modules should be taken as a minimum of 1% of the applied vertical load on the modules, which acts in combination with wind loading but at reduced load factors.

For modules designed with corner posts, it is shown that an additional effect owing to the shear flexibility of the end walls has to be taken into account when calculating the moments acting on the posts due to sway effects. The minimum eccentricity for design of the corner posts should not be less than 35 mm taking account of second-order effects, and the minimum eccentricity for design of load-bearing side walls should not be less than 25 mm.

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