NUMERICAL MODELLING OF TUBE AND FITTING ACCESS SCAFFOLD SYSTEMS

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ABSTRACT: This paper describes numerical modelling of tube and fitting access scaffold structures according to the new European standard EN12811. Firstly simplified one- and two-dimensional models are described. The results from these analyses are compared with the output from three dimensional finite element buckling and non-linear elastic analyses with good agreement. Buckling analyses show that effective lengths of tube and fitting scaffolds are strongly influenced by the pattern of ties to the building façade and in many cases much greater than those currently assumed in design being approximately equal to vertical tying increments and not the lift height. As a result of the analyses the authors recommend that the maximum vertical tying increments for large tube and fitting scaffolds should not be greater than every two levels. Wind loads parallel to façade were shown to generate uplift on some windward legs of façade braces which could cause premature failure of the scaffold. Current UK design practice which largely ignores the effects of wind loads is unconservative for large scaffolds. Recommendations for improved practice are made.

Keywords: semi-rigid connections, buckling analysis, scaffold structures, finite element analysis.

1. INTRODUCTION

The authors have made studies of the behaviour of proprietary scaffolding systems and of the behaviour of semi-rigid connections of the type used in scaffolds (Beale and Godley [1], Godley and Beale [2] and [3]). Previous work on the behaviour of tube and fitting systems was undertaken by Lightfoot et al [4] where the emphasis was on the importance of the behaviour of the connection.

The commonest type of scaffold within the UK is the tube and fitting scaffold which has over 80% of the scaffold market, proprietary and pre-formed scaffolds being more commonly used in specialised areas. Tube and fitting scaffolds are particularly popular in the erection of scaffolds around existing irregularly shaped buildings and for small domestic buildings. In the UK the design of tube and fitting scaffolds is regulated by BS 5973 [5]. This code uses linear analyses combined with the buckling lengths of the standards to take account of non-linear buckling effects. BS 5973 [5] implicitly suggests that the buckling length should be taken to be the same as the storey or lift height, irrespective of the position and frequency of any ties to the facade. The position and frequency of ties is determined by custom and practice and by the presence of wind loading due to sheeting. The arrival of a new European standard, EN12811-1 [6] means that current practice has had to be reviewed and that changes became necessary to design practice. The UK National Access and Scaffolding Confederation (NASC) commissioned the authors to develop a new design practice guide for tube and fitting scaffolds which conforms to the European standard [7]. One and two-dimensional models which were developed to produce the design guide and their verification by non-linear three-dimensional finite element analyses are reported in this paper.

2 SCAFFOLD COMPONENTS

2.1 Introduction

Tube and fitting scaffolding structures are erected against the façade of a building in order to provide access for the purposes of maintenance and repair. The shape and form of buildings commonly varies quite radically away from the regular and hence an efficient scaffolding system is one which, amongst its other attributes, can be arranged to accommodate a very large variety of configurations.

In the UK the traditional way of doing this is to construct the scaffold from tubes and connectors, called fittings. The tube is normally 48.3 mm diameter with a wall thickness of 4mm, manufactured from steel with a yield stress of 235N/mm^2.

The fittings come in a variety of styles but the most common types in use are the right angle coupler which connects two tubes at 90^o, the swivel coupler which allows the angle between the two connected members to vary, and the putlog coupler which connects tubes at right angles, but which has limited strength. Examples of these fittings are shown in Figure 1. A full description of these components is given in Godley and Beale [8].

2.2 Scaffold types

There are two primary types of access scaffold built from tubes and fittings. The first but less common type is the Putlog scaffold. This is shown in Figure 2 and comprises a line of columns or standards connected together at each level using right angle couplers by horizontal members called ledgers. This assembly is tied to the building façade at intervals by tie members. In addition putlog members which are tubes with flattened ends are fitted between the scaffold and the façade to provide a support for the timber scaffold planks which carry the load. The putlogs are connected to the ledgers with putlog couplers which leave the top surface of the putlog members flush to receive the timber planks. In-plane stiffness parallel to the façade is provided by façade bracing in the full height of the scaffold. For its stiffness normal to the façade the scaffold is dependent on the presence of the ties and their frequency.

The more common type of tube and fitting access scaffold is the independent tied scaffold shown in Figure 3. This scaffold is independent of the façade to the extent that it carries all the vertical loads and does not transfer any to the façade. There are two planes of standards and ledgers connected together with right angle couplers. They are linked together with transom members using putlog connectors and these transom members carry the scaffold boards. Diagonal bracing is installed in the plane parallel to the facade, but only in the front face of the scaffold. The rear face has no such bracing. Alternate pairs of standards are fitted with ledger bracing which is diagonal bracing in the plane normal to the façade. Independent tied scaffolds are tied to the façade at regular intervals, using a tie member connected to both standards. The tie member spans horizontally between the facade and the front braced face of the scaffold and thereby affords horizontal restraint to the rear face of the scaffold.

2.3 Scaffold Loading

The most recent European standard EN12811-1[6] defines the imposed loads for a number of scaffold classes. Unusually in a steel structure, self weight can be significant, particularly when the scaffold is boarded out at every level, and must be accounted for in the design. In the case of the

Putlog scaffold half of the imposed load is applied to the façade of the building.

Scaffolds are also exposed to wind loading. Wind loads can act parallel to the façade and normal to the façade. The intensity of the loading depends upon the wind speed and upon whether the scaffold is sheeted or unsheeted. In the direction normal to the facade, wind effects are not significant because they are usually reduced by the shielding effect of the façade, unless the façade is very permeable. When the façade is permeable, the wind loading induces bending in the members of the scaffold and may induce significant forces in the ties. In the unsheeted condition, wind loads parallel to the façade act on all the members of the scaffold. No shielding is allowed. These loads produce an overturning effect which induces an additional compressive force into the leeward members of the façade bracing panels. This is illustrated in Figure 4.

When the scaffold is sheeted, the area subjected to wind loads is much increased. When the wind blows parallel to the façade there is again a horizontal force which induces overturning effects and results in increased compression in the leeward leg of the bracing panels. This overturning effect is amplified by the effects of frictional drag on the sheeted face of the scaffold.

An additional effect of the wind blowing parallel to the façade is wind suction normal to the facade. This action puts the standards and ledgers into bending and induces significant loads into the ties to the facade. When the wind blows normal to the façade a sheeted scaffold on the windward face of the building will be subjected to considerable wind pressure. However, in the design of these structures, the assumption is that the rear face components such as the transoms and putlogs are in such close proximity to the facade that they will deflect into bearing with the facade. This action creates many points of support for the scaffold and thus reduces the bending moments induced into the members of the structure. When a scaffold is constructed on the leeward face of the building the wind forces are smaller but act in suction on the surface of the sheeting, inducing bending into the scaffold members and tension into the ties.

2.4 Structural characteristics of the couplers

In order to carry out this investigation the structural characteristics of the couplers had to be defined. A conservative approach would have been to assume that all connections were pinned and that no bending resistance was available in any of the joints. However, two connections offered the possibility of a significant cruciform bending stiffness and resistance, the right angle coupler and the putlog coupler. No experimental data was available to the authors on putlog connections. To obtain an estimate of their moment-rotation characteristics some preliminary cantilever tests were made at Oxford Brookes University on a typical right-angled coupler and on a putlog coupler. The results of these tests are shown in Figures 5 and 6. The solid lines in each figure are the results of individual experimental results and the dashed lines are the moment-rotation curves used in the analyses. As both putlog and right-angled connectors are manufactured by many different companies using forgings and pressings and hence have a range of stiffnesses and maximum moment capacities and as single values were required conservative values were used in the analyses. A research project is currently underway at Oxford Brookes University to obtain comprehensive design data.

In both cases the curves show the relationship between moment and rotation in cruciform bending of one side of the joint compared with the other about the Y axis in Figure 1(a). For both couplers, a stiffness and a design moment have been arbitrarily chosen below the experimental curves. The chosen characteristics are selected to allow for the statistical effects of determining the characteristic values that would have to be taken into account in a more formal assessment. In the case of the right angle coupler, this was a Type A coupler according to the designation of EN-74 [9].

The values chosen for the Type A coupler are 10kNm/radian and a characteristic moment of resistance of 0.4kNm. For the putlog coupler there are no figures available in British or European standards; the figures chosen here are 2kNm/radian and 0.06kNm for the characteristic moment of resistance. Apart from these values all other connections are assumed to be pinned.

2.5 Tie positions

The positions of ties which link the scaffold to the facade are crucial in any study of the buckling behaviour of an access scaffold. There are a very large number of possible tie patterns that can be used. In practice only regular patterns are of interest and those which represent the worst case. The critical parameters affecting structural behaviour are the vertical and horizontal intervals between ties. In this paper it is assumed that alternate pairs of ledger braced standards (or alternate pairs of single standards in the case of Putlog scaffolds) are tied and that the ties are in vertical rows and not staggered. In Figures 2 and 3 such a tie pattern is shown with ties at every third lift. Staggered tie patterns are more effective at restraining the standards against buckling, but the effect is not great. The effects of different tie patterns have been discussed by the authors in a previous paper (see Godley and Beale [3]).

3 THE BUCKLING BEHAVIOUR OF ACCESS SCAFFOLDS

3.1 Introduction

The authors have carried out an investigation into the buckling behaviour of scaffolds in an attempt to confirm apparent assumptions made in current British Standards governing scaffold design and to establish a simple design approach that avoids the use of sophisticated software.

The current UK design approach [5] is to carry out a linear analysis of the scaffold and to deal with the non-linear effects of buckling by using the effective length concept. An alternative to this approach is to use the amplification method to deal with the non-linearity. In both cases knowledge of the buckling modes and the associated buckling loads is required. Neither approach is as accurate as a full second order analysis. In this paper the buckling behaviour is described in terms of effective lengths in order to simplify comparisons with current practice.

In order to determine the effective lengths of scaffolds under different tie patterns unit loads were applied to the top of each standard and the critical load P_{cr} determined. The effective length L_E was then calculated using the formula

$$L_e = \sqrt{\frac{\pi^2 EI}{P_{cr}}} \tag{1}$$

where EI is the effective rigidity of the standard.

3.2 Two dimensional models

In the case of the Putlog scaffold, buckling in the plane of the façade is restricted by the presence of the facade bracing so that the buckling length is equal to the storey height. All analyses were undertaken with the storey or lift height taken to be 2.0m except that the first lift could be either 2.0m or 2.7m above the ground. Buckling normal to the facade is much influenced by the position of the ties. The standard which is not directly tied to the façade is the least restrained. Its horizontal restraints are provided at each level by the ledgers. The ledgers are normally put into bending as the buckling mode develops and act as horizontal linear springs. Because of the repetitive nature of the structure, for a putlog scaffold, the bucking may be represented by the two column model shown in Figure 7.

This model has one tied column and one free column. The tied column is restrained, for the example in Figure 7, at every third level and the free column is linked to it by linear springs with a stiffness *k*. If the springs are stiff enough, the free column and the tied column buckle together at a load corresponding to an effective length equal to the tie interval. In the case of the example in Figure 7, with stiff springs the effective length is 6m and the ledgers do not flex but remain straight. If the springs are not stiff enough for this mode of buckling, the free column will buckle independently of the tied column at a load corresponding to an effective length greater than the tie interval.

Figure 8 shows the relationship between the effective length produced by the two-dimensional model of Figure 7 and the stiffness of the restraining springs. It was produced using a linear eigenvalue program. From the graph it can be seen that for a tie interval of 6m, for example, log10(spring stiffness) must exceed 0.34 if the two columns are to buckle together.

The ledger is a continuous beam along the face of the scaffold at each lift. The lowest stiffness it can afford to the free standard occurs when alternate free columns buckle inwards and outwards and the ledger behaves as a simply supported beam as shown in Figure 9. From this figure the deflection, Δ is given by:

$$\Delta = \frac{P(2L_b)^3}{48EI} \tag{2}$$

in which EI is the flexural rigidity of the ledger, P the applied load and L_b is the bay width, the distance between the columns. Hence the required stiffness, k, can be found by:

$$P = \frac{48EI}{(2L_b)^3}\Delta = k\Delta \tag{3}$$

The value of the stiffness k is therefore

$$k = \frac{48EI}{(2L_b)^3} \tag{4}$$

In the case of normal Putlog scaffolds, the stiffness afforded by the restraining ledgers is always sufficient for the columns to buckle together, except when the tie interval is 2m. This is illustrated in Table 1, where the effective lengths for three different bay lengths are tabulated for simply supported ledgers.

The highest value of stiffness that can be generated by the ledger is when it behaves as a fixed ended beam as shown in Figure 10. In this case

$$\Delta = \frac{P(2L_b)^3}{192EI} \tag{5}$$

which implies that

$$P = \frac{192EI}{(2L_b)^3}\Delta = k\Delta \tag{6}$$

Hence in this case the stiffness is given by

$$k = \frac{192EI}{(2L_b)^3} \tag{7}$$

Something close to this situation occurs if one free column buckles in isolation from the remainder of the columns, perhaps because it is carrying an additional axial load as part of a bracing system such as that shown in Figure 4. The curve in Figure 8 is still valid, and in Table 1 the effective lengths are shown for the fixed end mode for three different bay lengths.

The results in Table 1 show that in nearly all cases the effective length can be taken as equal to the tie interval. This is because for most of the cases considered, the stiffness provided by the ledger is sufficient to ensure that the free standard buckles with the tied standard. In those cases where there are ties at every level, bucking occurs in a mode where each free standard buckles alternately towards and away from the façade.

In the case of independent tied scaffolds buckling normal to the façade takes place in a mode where alternate non-tied standards buckle in alternate directions. Consequently for this case the scaffold model in Figure 7 can be used, because the ledger bracing simulates ties normal to the façade at every lift, provided that there is at least one tie per ledger braced frame. In the direction parallel to the façade buckling of the front face is governed by the façade bracing and the buckled length is equal to the lift interval. For the rear plane of standards, the buckling load is strongly influenced by the tie positions, and conservatively it could be assumed to be equal to the tie interval. However, the semi rigid nature of the connection between the standard and the ledger reduces the buckling length. Figure 11, drawn for a scaffold tied at every other lift, presents a suitable single column model of the buckling of a rear standard. This model consists of a column with horizontal restraints at the tie levels and rotational restraints at all the intersections with the ledger. The rotational stiffness, k_φ, of this connection is calculated for ledgers in double curvature between standards as shown in Figure 12.

The rotation, θ, of the ledger is

$$\theta = \frac{M}{2}\frac{L_b}{2}\frac{1}{3EI} = \frac{ML_b}{12EI} \tag{8}$$

where EI is the flexural rigidity of the ledger, L_b is the bay width and M the restraining moment applied to the standard from the ledgers. The rotation, ϕ, of the right angle coupler connecting the ledger and the standard of cruciform stiffness, k_c, between ledger and standard is given by

$$\phi = \frac{M}{k_c} \tag{9}$$

Hence the total rotation between ledger and standard is

$$\theta + \phi = \frac{ML_b}{12EI} + \frac{M}{k_c} = M\left(\frac{L_b}{12EI} + \frac{1}{k_c}\right) \tag{10}$$

The equivalent rotational stiffness, k_φ, at the connection is therefore

$$k_\varphi = \frac{M}{\theta + \phi} = \frac{1}{\dfrac{L_b}{12EI} + \dfrac{1}{k_c}} = \frac{k_c}{1 + \dfrac{k_c L_b}{12EI}} \tag{11}$$

An analysis of this model shows that the buckling lengths are a little smaller than the vertical interval between ties.

3.3 Three-dimensional finite element analyses

To validate the column models three-dimensional finite element models of the scaffolds were created using the LUSAS program [10]. Examples of the models are shown in Figure 13. The models were based on the following assumptions:

- The standards and ledgers were initially modelled with 4 Kirchoff elements per lift. These elements are non-conforming, curved beam elements which have three rotational and three translational degrees of freedom at each end and in addition at the midside node degrees of freedom corresponding to incremental changes in translation and rotation. For the non-linear analyses described later the Kirchoff elements were replaced by co-rotational elements with three translational and rotational degrees of freedom at each end as these had improved non-linear capability. The second moment of area of the ledgers was taken to be 138000mm^4.

- Joint elements were used to connect the standards and ledgers. These elements have three translational and three rotational degrees of freedom. Milojkovic et al [11] have shown that the 50mm offset between ledger and standard in tube-and-fitting scaffolds can be ignored for scaffold assemblies and hence in this analysis the standards and ledgers were co-planar. Constraint equations were used to enforce the condition of zero horizontal and vertical translations between ledger and standard.

- The putlog connections from the standard to the façade or between front and rear faces in the transoms of the tied scaffolds were modelled using bar elements with translational degrees of freedom at each end for the buckling analyses. For non-linear analyses the transom connections were modelled by co-rotational beam elements with rotational releases at the ends. This change was introduced to allow for the possibility of transom buckling. For both types of scaffold the connection at the façade was restrained horizontally parallel to the façade and vertically. The connection was free to move in a direction normal to the façade. In practice there are small frictional resistances normal to the façade but these are not possible to quantify and hence were ignored.

- For the Putlog scaffold the ties were modelled as bar elements and pinned at each end. In the case of the independent scaffold the ties were considered to be beams connecting the two faces parallel to the façade and going into the façade. At the façade the tie was pinned. Connections between each tie and the front and rear standards were modelled in the buckling analyses by using joint elements with constraint equations to remove translational degrees of freedom. For the non-linear analyses translational degrees of freedom were removed by giving the joints large axial stiffnesses.

- Façade and ledger bracing elements were modelled using bar elements. To allow for the reduction in axial stiffness due to bending in the diagonal elements an effective area of 16mm^2 instead of the full area of 557mm^2 of the standards, putlogs and ledgers was used. An explanation of the derivation of this reduction in strength is given in Godley and Beale [3].

- Plan bracing was inserted into the model in every fourth lift, fourth bay as a horizontal diagonal bar element with an effective area of 16mm^2.

- The supports to the ground were pinned.

- The loading from the scaffold boards, live loads, and dead load was applied at nodal points. Wind loading was applied as a combination of point and distributed loads to all standards. The live loads are applied to the top two levels only as vertical loads in agreement with the European standard EN 12811-1 [6].

Once a linear model was proved to be correct a linear eigenvalue buckling analysis was carried out. Convergence difficulties were encountered in obtaining buckling modes for load cases combining wind and live and dead loads. These were due to the range of different stiffnesses in the structure - very low joint stiffnesses in combination with relatively high beam stiffnesses. However, for the buckling analyses reported in this paper good converged results were obtained for loads applied at the top of the scaffold. Figure 13 gives examples of the buckling modes determined.

The tie arrangements are shown in Table 2. Tables 3 and 4 give the results of the analyses for different bay widths. Note that the tie pattern for load case 5 defined in Table 2 has ties at 2m intervals and the effective lengths correspond to those in the first line of Table 2. Load cases 6, 7 and 8 have ties at 4, 6 and 8m intervals and the results are close to the figures in Table 1. They are a little higher in the finite element model because the height of the scaffold in this model was not always a multiple of the tie interval (The top lift was always tied in the finite element model). In addition for the independent tied scaffold effective lengths were determined for cases where the ledger brace was omitted (Table 5). The results are given for buckling both parallel to and normal to the facade for the independent tied scaffold. For the putlog scaffold buckling always occurred normal to the facade. Buckling parallel to the facade is constrained by the facade bracing and therefore has an effective length of either the lift height or the bottom storey height, whichever is greater.

3.4 Discussion of buckling results

The first result to be noted from the results concerning different tie heights is that the effective length of the scaffold standards is governed by the distance between tied levels, and not the distance between lift heights - the assumption made in the current British Standard. For example the effective length of an independent tied scaffold with a bay width of 2.1m, an initial lift height of 2.0m, lifts at 2.0m intervals and tied every 6.0m (case 7) is 4.81m and not 2.0m.

For the Putlog scaffold, buckling parallel to the facade is restricted by the storey height because of the presence of the facade bracing. Buckling normal to the facade is influenced by the position of ties. The standard not directly tied to the facade is the least restrained. Its restraints normal to the facade are provided at each level by the ledgers. Three modes of buckling can occur. In the first, all standards buckle in the same direction. In the second, alternate untied standards buckle inwards and outwards, the tied standard being unbuckled. The third mode occurs when an isolated standard buckles. The first mode normally occurs for Putlog scaffolds and is shown in Figure 13(c). However, when there is significant side load applied to the scaffold, the leeward standard of the bracing panel resisting the overturning moment due to this side load is subject to increased compression and may buckle in isolation. This is the third mode of failure and is shown in Figure 13(a) for a scaffold tied at every lift. The reduction in capacity due to the effects of side load on this bracing panel was reported by Godley and Beale in 1997 (Godley and Beale, [2]). The second mode occurs when the Putlog scaffold is tied at every level, as in cases 1 and 5.

Buckling of an independent tied scaffold normal to the facade occurs in a mode where the untied standards buckle in alternate directions. This mode is shown in Figure 13(b). This mode occurs because, by virtue of the ledger bracing and the fact that all ties are assumed to be attached to the ledger braced standards, the braced standard is effectively tied at every level. Consequently the buckling lengths are very close to those of the putlog scaffold with ties at every level. This applies irrespective of the actual tie interval, provided that there is at least one tie at each ledger braced frame. Buckling parallel to the facade for the front face is governed by the facade bracing and the buckling length is approximately equal to the lift height. For the rear plane of standards, however, buckling is strongly influenced by the tie positions, and conservatively it could be assumed to be equal to the tie interval. However, the semi-rigid nature of the connection between ledger and standard reduces the buckling length. Figure 13(d) shows the buckled mode for the scaffold with ties at every alternate lift. Table 5 shows that the absence of ledger bracing makes little difference on the buckling parallel to the facade but reduces the buckling load normal to the facade.

Due to the large effective lengths of the standards which occur when scaffolds are tied at vertical intervals greater than every two lifts the authors recommend that all tube and fitting scaffolds should be tied at vertical intervals not greater than 4.0m. This recommendation was used in all the non-linear analyses. It is also to be noted that the buckling analyses have shown that the current UK standard BS5973 [5] which implicitly assumes an effective length equal to the maximum lift height yields unconservative results.

4 NON-LINEAR ANALYSES

4.1 Introduction

Non-linear elastic geometric analyses were performed with reasonable correspondence between the results from the three-dimensional analyses and the one- and two-dimensional analyses. Material non-linearity was considered by use of the interaction formulae given below.

In conformance with the European standard EN12811-1 [6] a load factor of 1.5 was applied to all loads. The following load combinations were considered for every scaffold analysed:

Load case 1: Self weight plus service imposed load plus frame imperfection load normal to the facade plus a horizontal load equal to 0.3kN in each bay normal to the facade at the working lift. The horizontal load is given in EN12811-1 [6] as a horizontal load to be applied when no wind load is applied.

Load case 2: Self weight plus service imposed load plus frame imperfection load normal to the facade plus service wind load normal to the facade.

Load case 3: Self weight plus out-of-service imposed load plus frame imperfection load normal to the facade plus out-of-service wind load normal to the facade.

Load case 4: Self weight plus service imposed load plus frame imperfection load parallel to the facade plus a horizontal load equal to 0.3kN in each bay parallel to the facade at the working lift.

Load case 5: Self weight plus service imposed load plus frame imperfection load parallel to the facade plus service wind load parallel to the facade.

Load case 6: Self weight plus out-of-service imposed load plus frame imperfection load parallel to the facade plus out-of-service wind load parallel to the facade.

Frame imperfections were ignored for the early buckling analyses which only considered point loads at the top of the scaffold. For the non-linear analyses these were represented by horizontal loads directly proportional to all vertical loads with the constant of proportionality, ϕ, given by the formula

$$\phi = \phi_0 \left(0.5 + \frac{1}{n_c}\right)^{0.5} \left(0.2 + \frac{1}{n_s}\right)^{0.5} \tag{12}$$

in which n_c = the number of fully loaded standards (= no of bays)
n_s = the number of lifts
ϕ_0 = 0.01 (corresponding to the erection tolerance set in BS5973 [5])

These loads were applied either normal or parallel to the facade depending upon the load case being considered.

The non-linear analyses started from 10% of the design load and increments were made until either the structure was unable to carry further load or three times the design load was achieved.

The following analyses were made to each scaffold considered:

(i) A linear analysis to check structural loads and structural geometry including restraints.

(ii) A buckling analysis.

(iii) A non-linear analysis including geometrical $P-\delta$ non-linearity. This started from 10% of the design load and increments were made until either the structure was not able to carry further load or three times the design load was achieved.

(iv) A Fortran program was written to process the output of each load increment in the non-linear analysis. The following checks were incorporated:

$$\left(\frac{N_{Sd}}{P_c} + \frac{M_{xSD} + M_{zSd}}{M_{Rd}}\right)1.1 = k \quad \text{(for compressive axial loads)} \tag{13}$$

$$\left(\frac{N_{Sd}}{Af_y} + \frac{M_{xSD} + M_{zSd}}{M_{Rd}}\right)1.1 = k \quad \text{(for tensile axial loads)} \tag{14}$$

$$\frac{M_{coupler} \cdot 1.35}{400} = m_{test} \tag{15}$$

where N_{Sd} is the design axial load, M_{xSd} is the design bending moment about the xx axis, M_{zSd} is the design bending moment about the zz axis, P_c = 48kN for 2m lift and M_{Rd} =1.85kNm (plastic moment of resistance of a 4mm thick scaffold tube with f_y = 235N/mm^2). k and m_{test} are limit state values with values less than 1.0 implying that the test is satisfied and values greater than 1.0 implying that the element of the structure has failed. $M_{coupler}$ is the characteristic moment for the coupler. Initial checks of coupler and joint slippage were made but as the forces in the connections were low these checks were not made for all analyses. The use of a partial safety factor of 1.35 for the coupler test is always conservative. During the process of conducting the analyses it was suggested that the partial safety factor for the coupler test should be reduced to 1.1. However, throughout the analyses, the original moment test was always satisfied and so no change was made. Failure was deemed to have occurred when the limit state values exceeded 1.0 for any element in the scaffold.

4.2 Loading

All dead loads were applied as point loads at the appropriate lift position. Different load distributions were made for tied scaffolds with and without cantilevered boards at the rear. For the independent tied scaffold an allowance was made for the diagonal brace in the plane normal to the façade but not for the diagonal brace parallel to the façade. To simplify loading data the loss of weight when ledger bracing was removed was ignored. This produced slightly conservative results. The weights of individual components are given in Table 6. These weights are in conformance with the weights given for these components in BS5973:1993 [5]. The mass of a coupler in this code is said to range from 1.1 to 2.25kg. The value of 1.8kg was an assumed median value.

For the non-linear analyses the imposed loads were uniformly distributed loads for Classes 1, 2, 3 and 4. For the service condition this load was applied to the top lift with 50% of the intensity applied to the second lift. In the out-of-service condition corresponding to the maximum load that the scaffold could carry under storm conditions this load was applied to the top lift only, reduced by the factor given in column 3 of Table 7.

Service wind load intensity was taken to be 200 N/m^2 on every element. For the out-of-service or storm wind load the intensity, q, was given by

$$q = a\ln(h) + b \tag{16}$$

where the values of the coefficients a and b were derived by regression from Table 4 of BS6399 part 2 [12], for different town and country distances (denoted by the abbreviations T and C in Table 8) and wind velocity factors (S). h is the height of the scaffold above ground level. In accordance with EN12811-1 [6] rules for temporary structures the out-of-service wind pressures were reduced to 70% of the values calculated by the above formula.

The direction of the wind load normal to the facade was such as to put the ties into tension. The compressive direction was not considered as it was assumed that the scaffold would deflect into bearing with the facade, this improving its stability.

For unsheeted scaffolds the wind load on standards, ledgers and transoms was applied as distributed line loads. The loads on toe boards and guard-rails were applied as point loads on the standards at the appropriate levels. The guard-rail wind load on the top lift was exceptionally applied at the top lift due to the model not including standards above the top lift level.

For sheeted scaffolds the following was applied:

Winds normal to the facade: pressure coefficient 0.5 normal to the facade, no load on the end of the facade

Winds parallel to the facade: pressure coefficient 1.0 on the ends parallel to the facade, 0.5 (suction) normal and 0.01 (friction) parallel to all front elements along the front of the facade.

For debris netted scaffolds the following was applied:

Winds normal to the facade: pressure coefficient 0.25 normal to the facade, no load on the end of the facade

Winds parallel to the facade: pressure coefficient 0.5 on the ends parallel to the facade, 0.25

(suction) normal and 0.03 (friction) parallel to all front elements along the front of the facade.

To calculate the equivalent wind loads for both sheeted and debris netted scaffolds the load was applied to each bay and lift panel as shown in elevation in Figure 14. The areas of each subsection were calculated and the total load on the panel distributed to the ledgers and standards in proportion to the area of each subsection.

A small correction was made to the bottom lift where it was assumed that the wind load was only applied to the ledger above the panel and to the adjacent standards. The trapezoidal distribution was applied to the largest element of either the standard or the ledger. The effect of wind loads acting on the guard-rails was ignored as it would have meant two different loading distributions for boarded and unboarded scaffolds.

For the in-service wind condition the trapezoidal and triangular loads were modelled accurately as the program was able to apply distributed multi-linear constant loads. For the out-of-service load condition the assumption was made that the logarithmic wind distribution could be approximated to the rectangle by applying the appropriate height pressure at nodal points and using linear interpolation. For a 20m high scaffold the error in this assumption was shown to be less than 0.1%. In addition as the program could not handle trapezoidal and triangular loads with variable magnitudes these loads were modelled by uniformly distributed loads with the same total force on the sides of each panel. The resulting total load applied to the scaffold by LUSAS was within 0.5% of the total load calculated by integration using a symbolic algebra package.

4.3 Results and Discussion

Table 8 summarises the scaffolds analysed. For these analyses the scaffolds had all lifts including the first at 2.0m height and alternative pairs of standards tied at lifts at heights 4.0m, 8.0m, 12.0m, etc. The top lift was always tied. Note that partial ledger bracing cases required two sets of full analyses - one with the bottom two ledger braces removed and one with the top two ledger braces removed. The cantilevered cases included one board cantilevered behind the rear face, adjacent to the facade. Further details of the loads applied to each scaffold are given in Beale & Godley [8]. Three different face bracing patterns were analysed – one bay every five bays, two adjacent bays every five bays and full diagonal bracing, repeating every five bays. The scaffold dimensions were chosen to verify the two-dimensional model. They were the tallest scaffolds which the one and two-dimensional models calculated that would support the required Class of Load under the action of its corresponding wind load.

The results of the analyses are summarised in Table 9 with sample ultimate modes of failure given in Figure 15. The method of applying load in LUSAS is by use of a load parameter called T_λ. A value of $T_\lambda = 1$ is the design load. The value of the limit state *k* defined by the maximum value of the tests described in Eqs. (13) or (14) shows if the scaffold successfully passes the test. If *k* is less than 1 the scaffold passed the test. The coupler moment test (Eq. (15)) was never critical in any of the analyses and hence is not shown.

From Table 9 it can be seen that all the Putlog scaffolds were capable of carrying the design load, with the proviso that the six lift, class 1, unboarded, two bays in five braced scaffold (reference number 2) developed a 2kN uplift force and the nine lift, class 2, unboarded, five bays in five braced scaffold (reference number 6) developed a 5.4kN uplift force in the windward leg of the facade brace. In both cases this was under the out-of-service wind load combination parallel to the facade. Most of the tied scaffolds were capable of handling the design loads. Certain results must be commented upon, however. Firstly, the cantilevered, unsheeted, 8 lift scaffold with partial ledger bracing (reference number 7) had high moments in the tie connecting the third pair of standards to the facade at the fourth lift level under the condition of out-of-service wind parallel to the face. These moments, whilst being individually acceptable, caused high values in the interaction formula. For example at $T_{\lambda} = 1$, for this element M_z = 1.69kNm. Secondly, the Class 4, 17 lift tied scaffold (reference number 8) marginally failed to carry the design load due to the same cause under the same wind load. However, it is felt that this result demonstrates that the approximate two-dimensional analysis accurately predicts maximum failure loads. In addition it is unlikely that this marginal failure would ever be observed, due to the simplifications made in the analysis which have all been designed to be conservative. Thirdly, the 8 lift, debris netted, boarded scaffold (reference number 9) demonstrated significant bending in the rear ledgers under both out-of-service wind load conditions. In addition the model generated significant tensile forces in the rear ledger members. This is thought to be due to the boundary conditions applied to the ties which allowed high horizontal reactions to develop. Modifications to the computer model were made where horizontal springs were introduced at the end of each tie. These springs reduced the values of the horizontal reaction forces and hence reduced the axial forces in the rear ledgers. However, they made very little difference to the ultimate limit state which was governed by ledger failure.

5 CONCLUSIONS

One-, two- and three-dimensional numerical models of tube and fitting scaffolds have been developed which show agreement with respect to each other; the three–dimensional results confirming the approximate one- and two-dimensional results. The approximate models are adequate to predict scaffold behaviour and can easily be used in design.

Buckling analyses have shown that the effective lengths of tube and fitting scaffolds are approximately equal to the vertical tying intervals and not the lift height as assumed in the current UK standard. The authors recommend that the vertical tying increments for all access tube and fitting scaffolds should not be more than every two levels. Wind load effects parallel to the façade can cause uplift on the leeward legs of façade braces which could cause premature scaffold collapses and must therefore be considered when designing scaffolds. Current UK design practice based on BS5973 [5] which largely ignores wind loads can be unconservative, especially for large scaffolds.

6 REFERENCES

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FIGURE CAPTIONS

Figure 1. Three types of scaffolding coupler

Figure 1(a). (a) right-angle coupler
Figure 1(b). Swivel coupler
Figure 1(c). Putlog coupler

Figure 2. Typical Putlog Scaffold

Figure 3. Typical Independent Tied Scaffold

Figure 4. Bracing action under side loads

Figure 5. Moment-Rotation Characteristics for a Putlog Coupler

Figure 6. Moment–Rotation Characteristics for a Right Angle Coupler

Figure 7. Two column Putlog model for buckling normal to the facade

Figure 8. Variation of effective length with spring stiffness

Figure 9. Derivation of stiffness for ‘pinned’ ledgers

Figure 10. Derivation of stiffness for ‘fixed’ ledgers

Figure 11. Single column model for the rear face of a tied scaffold

Figure 12. Derivation of rotation stiffness for standard-ledger connection in double curvature

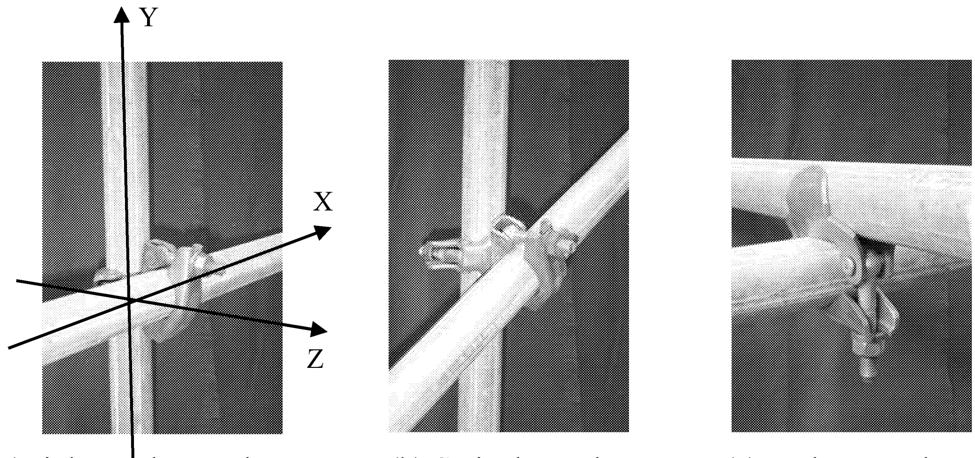
Figure 13. Examples of buckling modes

Figure 13(a). Buckling of leeward braced standard normal to the facade for a Putlog scaffold
Figure 13(b). Buckling of independent tied scaffold normal to the façade
Figure 13(c). Uniform buckling of Putlog scaffold normal to the facade
Figure 13(d). Buckling of tied scaffold parallel to the façade (only the rear face buckles)

Figure 14. Wind load distribution patterns

Figure 15. Examples of ultimate failure modes

Figure 15(a). Tied scaffold buckling normal to the facade
Figure 15(b). Putlog scaffold buckling normal to facade
Figure 15(c). Tied scaffold with rear face buckling parallel to facade
Figure 15(d). Tied scaffold buckling normal to facade with no ledger bracing in bottom two lifts



(a) right-angle coupler (b) Swivel coupler (c) Putlog coupler

Figure 1. Three types of scaffolding coupler

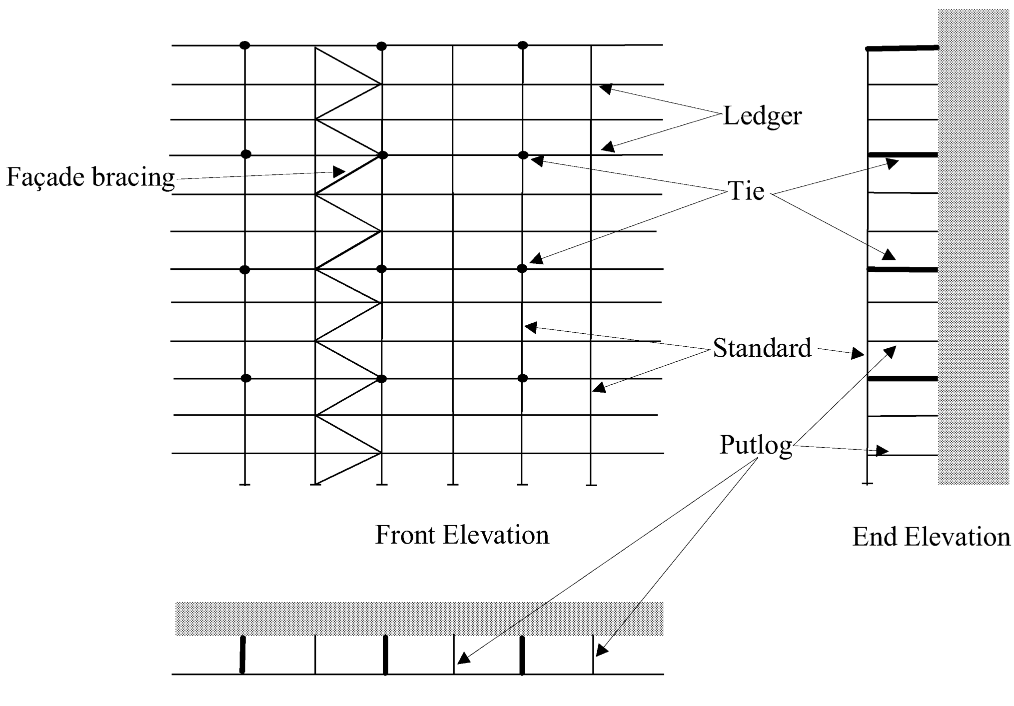


Figure 2. Typical Putlog Scaffold

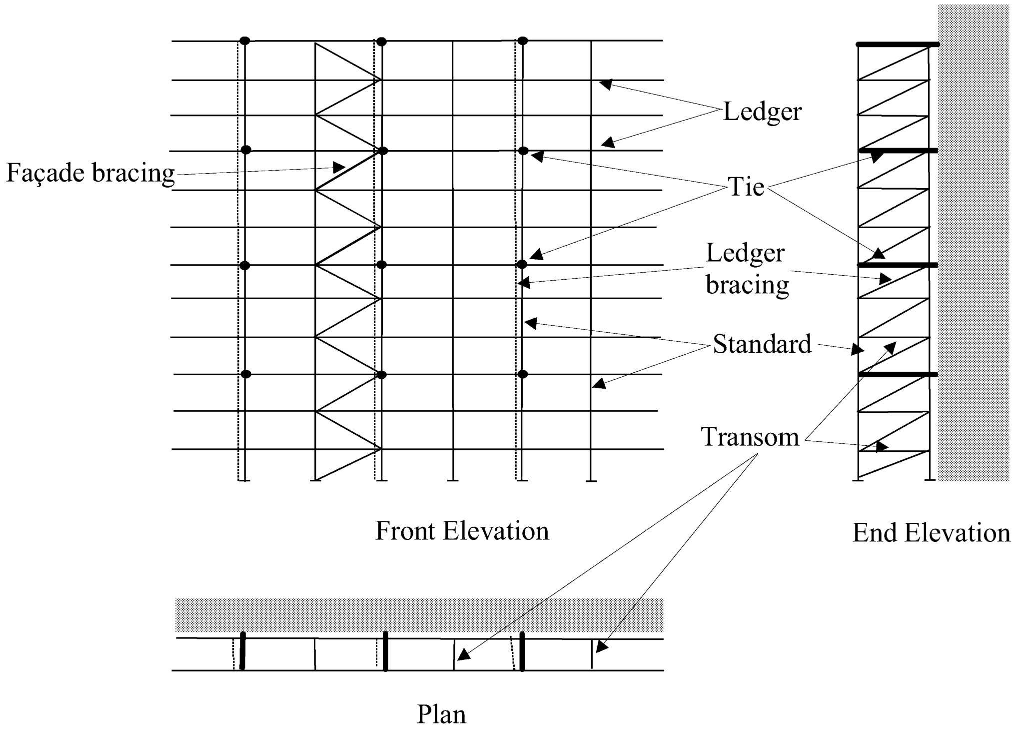


Figure 3. Typical Independent Tied Scaffold

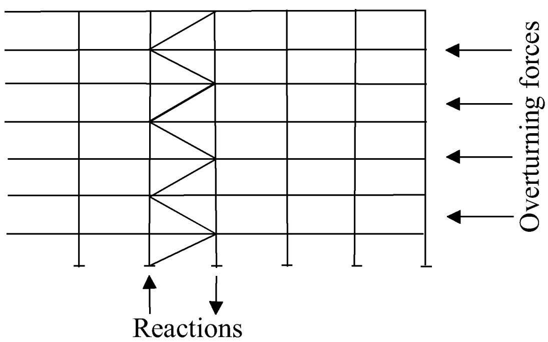


Figure 4. Bracing action under side loads

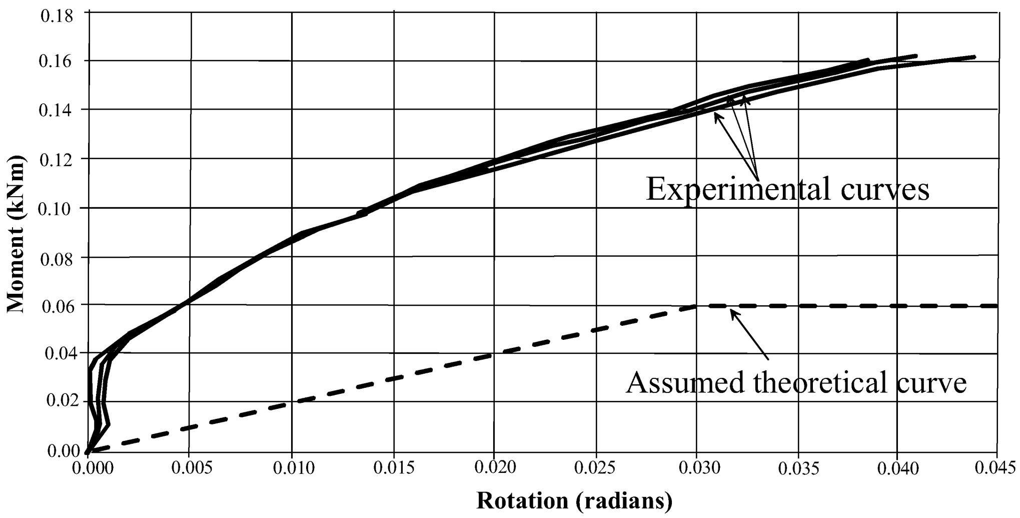


Figure 5. Moment-Rotation Characteristics for a Putlog Coupler

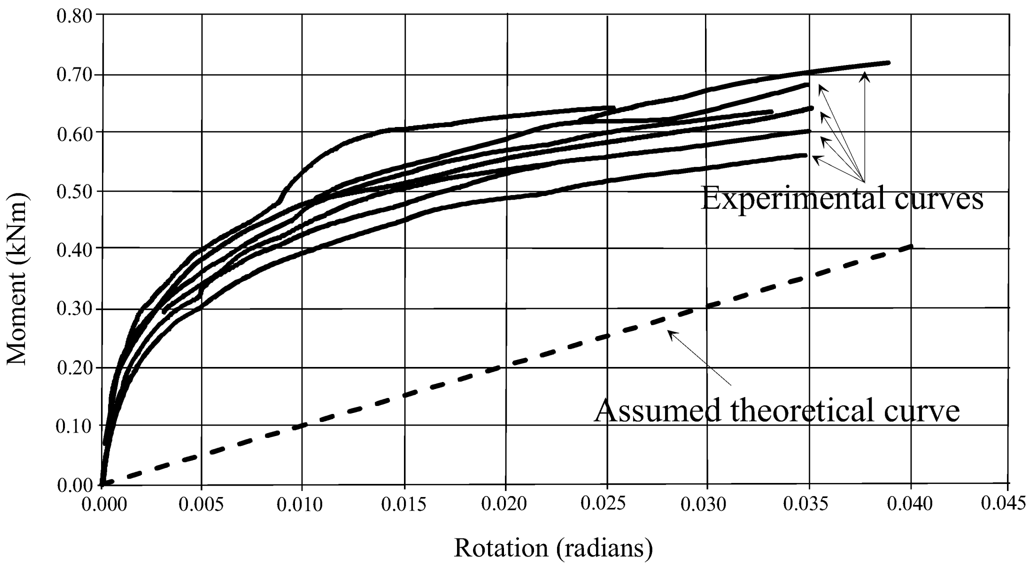


Figure 6. Moment–Rotation Characteristics for a Right Angle Coupler

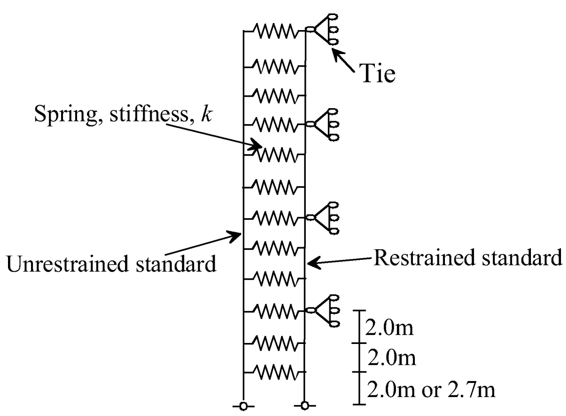


Figure 7. Two column Putlog model for buckling normal to the facade

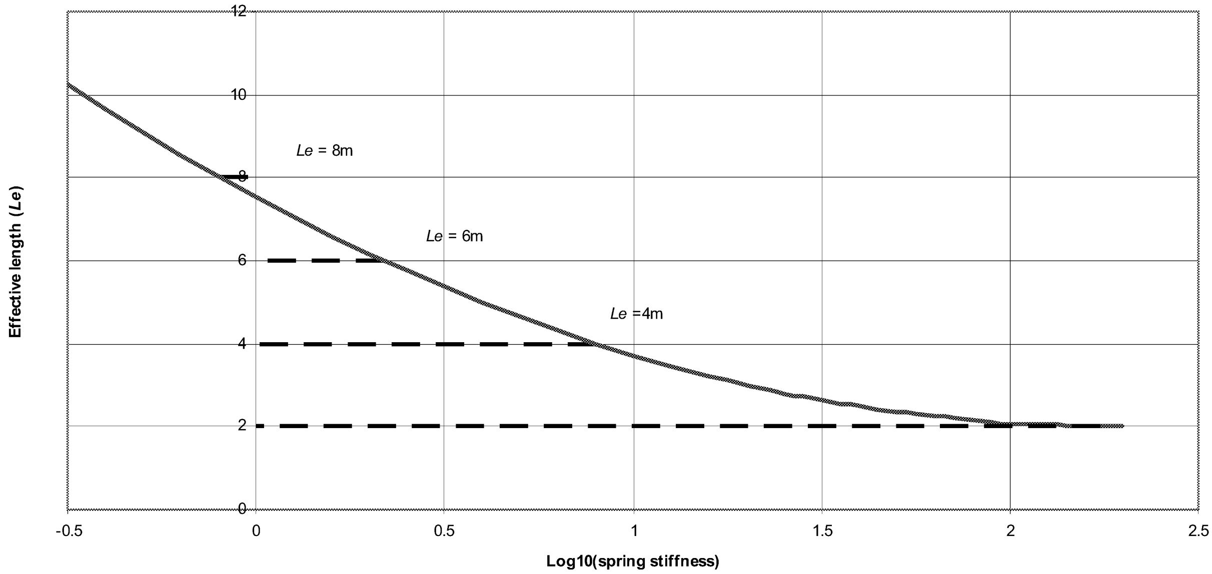


Figure 8. Variation of effective length with spring stiffness

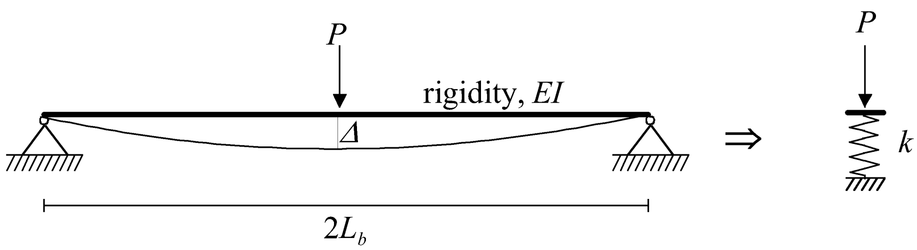


Figure 9. Derivation of stiffness for 'pinned' ledgers

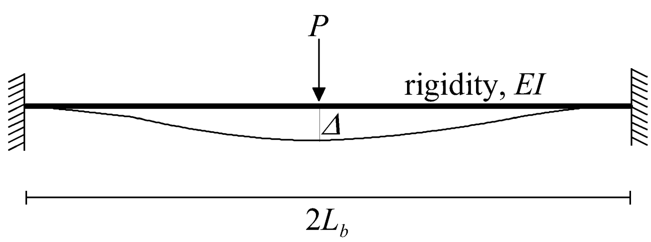


Figure 10. Derivation of stiffness for 'fixed' ledgers

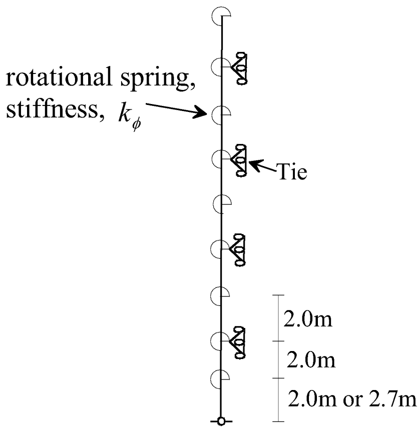


Figure 11. Single column model for the rear face of a tied scaffold

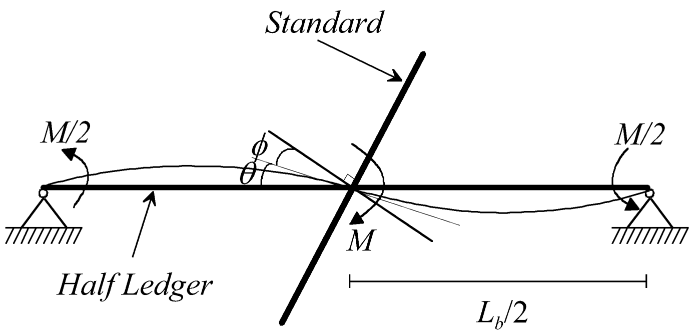
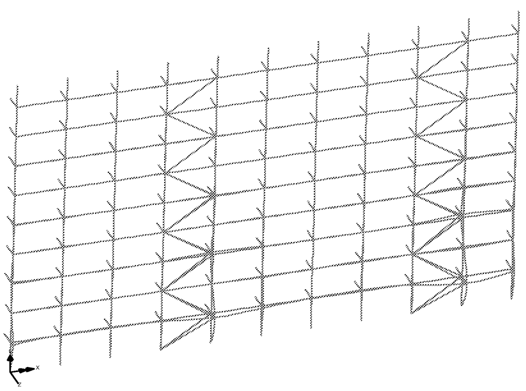
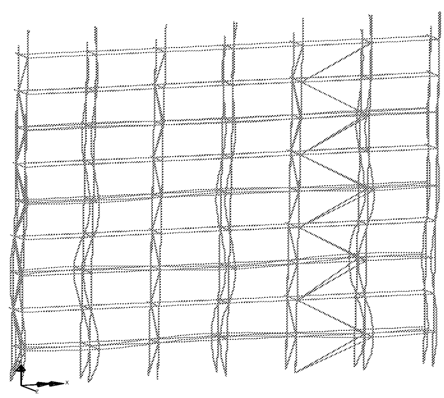


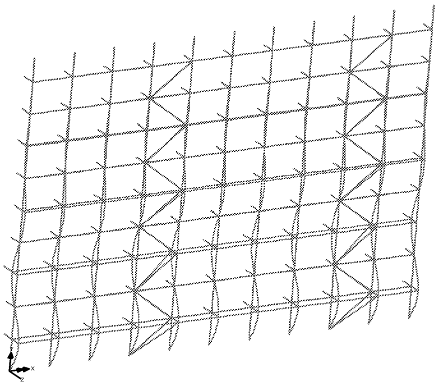
Figure 12. Derivation of rotation stiffness for standard-ledger connection in double curvature



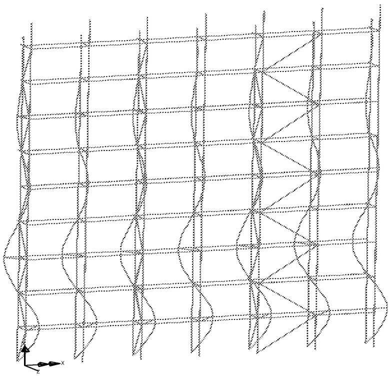
(a) Buckling of leeward braced standard normal to the facade for a Putlog scaffold



(b) Buckling of independent tied scaffold normal to the façade



(c) Uniform buckling of Putlog scaffold normal to the facade



(d) Buckling of tied scaffold parallel to the facade (only the rear face buckles)

Figure 13. Examples of buckling modes

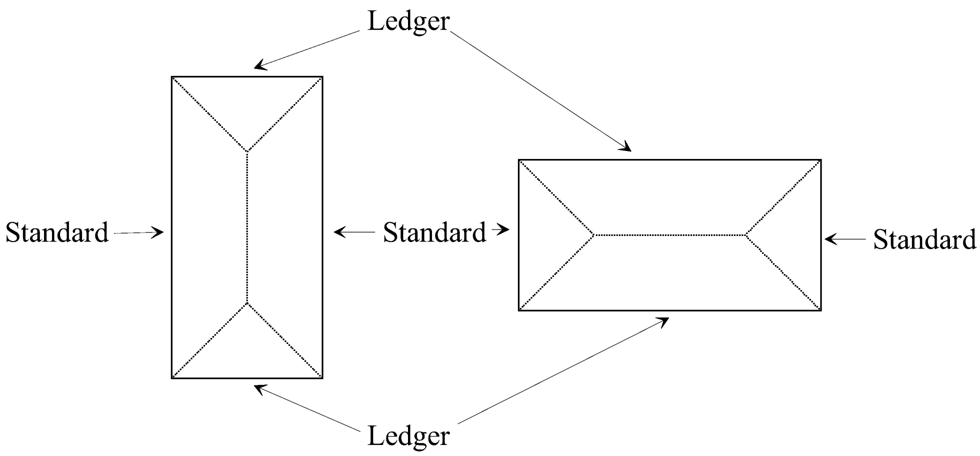
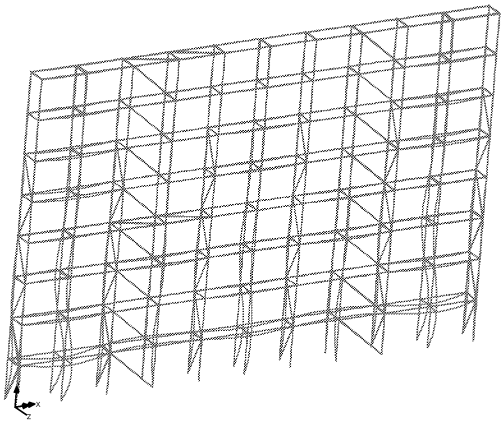
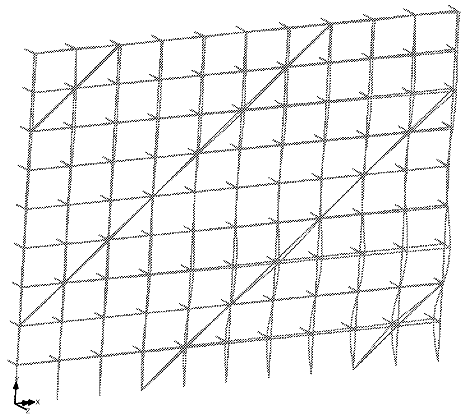


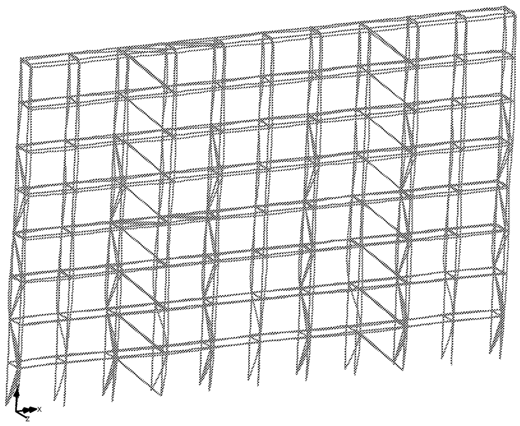
Figure 14. Wind load distribution patterns



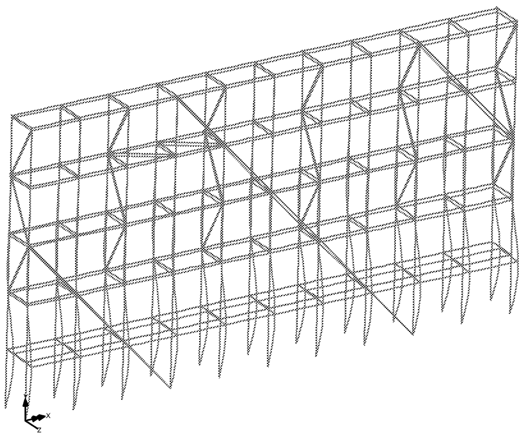
(a) tied scaffold buckling normal to facade



(b) Putlog scaffold buckling normal to facade



(c) tied scaffold with rear face buckling parallel to facade



(d) tied scaffold buckling normal to facade with no ledger bracing in bottom two lifts

Figure 15. Examples of ultimate failure modes

Table 1. Effective lengths for Putlog scaffolds found from the two-dimensional model

	Tie Interval	Span = 2.7m	Span = 2.1m	Span = 1.8m
Simply Supported Ledgers	2m	3.94	3.17	2.77
	4m	4.06	4.00	4.00
	6m	6.00	6.00	6.00
	8m	8.00	8.00	8.00
Fixed Ended Ledgers	2m	2.64	2.18	2.02
	4m	4.00	4.00	4.00
	6m	6.00	6.00	6.00
	8m	8.00	8.00	8.00

Table 2. Tie Patterns

Case	Tie position above base (m)								
1	2.7	4.7	6.7	8.7	10.7	12.7	14.7	16.7	18.7
2	2.7		6.7		10.7		14.7		18.7
3	2.7			8.7			14.7		18.7
4	2.7				10.7				18.7
5	2.0	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0
6		4.0		8.0		12.0		16.0	18.0
7			6.0			12.0			18.0
8				8.0				16.0	18.0
9	2.0		6.0		10.0		14.0		18.0
10	2.0			8.0			14.0		18.0
11	2.0				10.0				18.0

Table 3. Effective lengths of Putlog scaffolds for different bay widths buckling normal to the façade determined by the three-dimensional analysis

Case	1.8m	2.1m	2.7m
1	2.83	3.17	3.94
2	3.91	3.92	3.99
3	5.03	5.04	5.06
4	7.24	7.24	7.26
5	2.77	3.16	3.94
6	3.90	3.91	4.03
7	6.00	6.00	6.00
8	7.20	7.21	7.25
9	3.90	3.91	3.99
10	4.99	4.99	5.02
11	7.39	7.20	7.35

Table 4. Effective lengths of independent tied scaffold with full ledger bracing

	Parallel to façade			Normal to facade		
Case	1.8m	2.1m	2.7m	1.8m	2.1m	2.7m
1	2.39	2.29	2.29	2.83	3.17	3.94
2	3.42	3.42	3.43	2.85	3.19	3.95
3	4.16	4.16	4.18	2.87	3.21	3.95
4	5.03	5.04	5.01	2.90	3.27	4.01
5	1.90	1.90	1.90	2.77	3.16	3.94
6	3.56	3.56	3.56	2.78	3.18	3.95
7	4.81	4.81	4.83	2.90	3.25	3.98
8	5.55	5.57	5.57	3.29	3.18	3.96
9	3.40	3.41	3.41	2.84	3.19	3.95
10	4.12	4.13	4.13	2.86	3.21	3.95
11	5.00	5.02	5.04	2.90	3.26	4.01

Table 5. Effective lengths of independent tied scaffold with partial ledger bracing

		Parallel			Normal		
Description	Case	1.8m	2.1m	2.7m	1.8m	2.1m	2.7m
No bottom brace	2	3.46	3.47	3.47	3.21	3.37	3.57
	9	3.45	3.45	3.45	3.20	3.36	3.56
No brace in bottom two levels	2	3.46	3.47	3.47	3.21	3.38	3.57
	9	3.45	3.45	3.45	3.20	3.36	3.56
No brace in second and third levels from bottom	2	3.42	3.42	3.42	3.26	3.40	3.58
	9	3.40	3.41	3.41	3.22	3.37	3.56
No ledger brace	2	3.41	3.42	3.43	3.70	3.74	3.81
	6	3.56	3.56	3.56	3.75	3.77	3.81
	9	3.40	3.40	3.41	3.69	3.73	3.81

Table 6. Weights of scaffold components

Component	Weight
Coupler	1.8 kg
Ledger, standard, putlog, guard-rail	4.37 kg/m
Toe board	5.36 kg/m

Table 7. Table of loads

Class	Load intensity (kN/m^2)	Reduction factor	No. of Boards	Bay width (m)
1	0.75	0.00	3	2.7
2	1.50	0.25	4	2.4
3	2.00	0.25	5	2.1
4	3.00	0.50	5	1.8

Table 8. Scaffolds Analysed

Scaffold type	Ref. No.	Class	Sheeted or not	Boarded or not	Ledger bracing	Wind Storm param. **S**	T or C	Dist. to Sea	No of levels	Facade bracing
Putlog	1	1	not	yes	N/A	20	T	100	6	2 bays in 5
Putlog	2	2	not	not	N/A	24	C	0.1	9	5 bays in 5
Independent	3	2	not	yes	part	20	T	10	8	1 bay in 5
Independent	4	3	debris	not	full	24	C	10	17	2 bays in 5
Independent	5	4	sheeted	yes	part	28	T	0.1	5	5 bays in 5
Putlog	6	1	not	not	N/A	28	T	10	6	2 bays in 5
Independent	7	3 cant	not	yes	part	40	T	0.1	8	1 bay in 5
Independent	8	4 cant	not	not	full	40	C	100	17	5 bays in 5
Independent	9	1	debris	yes	full	32	T	0.1	8	2 bays in 5
Independent	10	4	sheeted	not	part	32	T	100	5	5 bays in 5

Table 9. Results of analyses

Ref. No.	Load case giving maximum limit state	Max k at $T_\lambda = 1$	Max T_λ	Mode
1	Service wind, normal to façade	0.696	6.360	Buckling normal to façade
2	Service wind, normal to façade	0.375	2.130	Buckling normal to façade
3	Service wind, normal to façade	0.730	1.845	Buckling normal to façade
4	Service wind, parallel to façade	0.458	1.947	Buckling parallel to façade
5	Out-of-service wind, parallel to façade	1.006	2.641	Buckling normal to façade
6	Out-of-service wind, normal to façade	0.549	3.000	Not buckled, maximum increments exceeded
7	Out-of-service wind, parallel to façade	0.984	1.575	Not buckled, convergence failure
8	Out-of-service wind, parallel to façade	1.072	2.407	Not buckled, excessive bending in ledger
9	Out-of-service wind, parallel to façade	0.975	3.000	Not buckled, maximum increments exceeded
10	Out-of-service wind, parallel to façade	0.730	3.000	Not buckled, maximum increments exceeded