THE VERTICAL STABILITY OF SLENDER LOAD-BEARING FAÇADE SYSTEMS

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ABSTRACT

The vertical stability of a slender precast concrete system has been studied theoretically. The system has bearing façades composed of composite columns and deep spandrel beams. Floor elements are connected to the spandrel beams by tie arrangements. A theoretical model of the interaction between load-bearing floors, beams and columns is presented. Floor-beam-column subframes that have been tested experimentally have been analysed. The agreement between measured and calculated responses was good. The structural system in a building was analysed for several situations, both ordinary and accidental. It was found that the action of the beam-floor connections increased the torsional moment in the spandrel beams in all of the ordinary load cases studied. The analyses of the behaviour in accidental situations showed that the beam-floor connections used nowadays have too little deformability. As a consequence, in an accident the floor elements would fall down onto the floor below which probably could not resist such impact loading; the connections would probably not be able to prevent a severe collapse in accidental situations.

Key words: Concrete structures, load-bearing façade, theoretical analysis, spandrel beams, torsion, semi-rigid connections

1. INTRODUCTION

Precast concrete buildings in general have the disadvantage that they cannot easily be modified. There has been an increasing demand for flexible systems, and therefore a Swedish company called Strängbetong AB has tried to construct a flexible system, where no columns interfere with the free space. Their attempts resulted in a system with bearing facades, built of composite columns and deep spandrel beams with L-shaped cross-sections. The columns are made slender enough to be concealed in the walls. Hollow core floor elements are supported by the spandrel beams, and connected by tie arrangements, see Fig. 1.

A few buildings have already been built in this way, for example "Sky City" at Arlanda Airport, Stockholm. This slender facade system has proved to be a rational way to build. But since it is rather complex, there were uncertainties about how this type of flexible system behaves, especially for accidental actions. The present paper treats the vertical stability of the structural system. In such a system, the floors are normally assumed to be simply supported on the spandrel beams; however, since the tie arrangements can transfer some moment, it would be more accurate to view them as semi-rigid connections. Accordingly, the torsional moment in the spandrel beams is influenced. The torsional moment is transferred, in turn, to the columns, where it results in bending out of the plane.

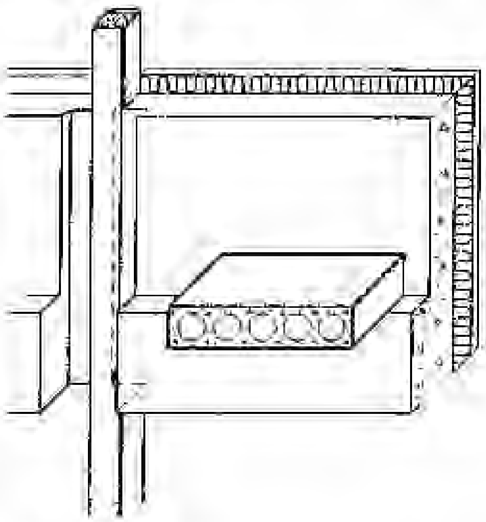


Figure 1 Part of the façade in the system studied.

A theoretical model of the structural system was worked out; it is briefly presented here in Section 2, further discussed in Section 4, and in Section 5 measurements from experiments are compared with results from the analysis. The model was used to study the effects of systematic parameter variation, presented in Section 3. Full-scale experiments, reported in Ref. /1/, provided necessary information about the behaviour of the individual components and connections.

2. THE THEORETICAL MODEL

The analysis includes a lot of iteration; therefore a computer program has been developed to make the work easier. This is briefly how it works:

- The theoretical analysis starts at the connection between spandrel beam and slab. The relationship between transferred moment and rotation is given as input.
- End angles of slabs are calculated for various end moments (for given load).
- This leads to a relationship between rotation of the spandrel beam and their torsional load.
- Relations between loads and deformations of the columns are set up in matrix equations. From these, a relationship between beam end rotation and torsional moment is determined. This relationship also depends on the spandrel beams beneath and above.
- The relationship between rotation of the spandrel beam and the torsional load, together with the relationship between torsional moment and distortion, gives a differential equation.
- This differential equation has the relationship between beam end rotation and torsional moment, and symmetry, as boundary conditions. It is solved for each floor. The boundary conditions depend on the spandrel beams beneath and above. Therefore starting values are assumed, and the problem is solved by iteration.

For more details about how the calculations were performed, see Ref. /2/. The basic assumptions that are made are presented in the following sections.

2.1 Columns

The columns are composite ones of tubular type with infilled concrete cast *in situ*. They are the type used in practice in the slender façade system. The steel tube is an H-section with cover plates welded along the edges of the adjacent flanges, see Fig. 2a. Since the columns consist mainly of steel, they remain elastic up to a large part of the maximum load, see Ref. /1/. Accordingly, the columns are assumed to be linearly elastic. Second order effects are taken into account by the analytical derived stiffness and flexibility functions which depend on the normal force in the column, Ref. /3. The columns are assumed to have different values of bending stiffness in the free parts between the spandrel beams, EI_f, and in the parts where the spandrel beams are connected, EI_c.

The model is applicable for the parts of the façade not affected by the corners of the building and where all of the beams are the same length, since the columns are assumed to be symmetrically loaded. They are restrained at the base by fixed ends, see Fig. 2b. The floors prevent deflections of the columns where they are connected; in Fig. 2b this is shown in the form of supports.

2.2 Connections between Columns and Spandrel Beams

The connections between the spandrel beams and the columns are assumed to be stiff. The torsional moment in the spandrel beams is transferred to the columns by horizontal forces that are linearly distributed, see Fig. 3. The vertical normal force from each floor is assumed to act on the column at the centre of torsion of the spandrel beam. If this force is not symmetric at the column, the normal force will have an eccentricity called e_c, see Fig. 3.

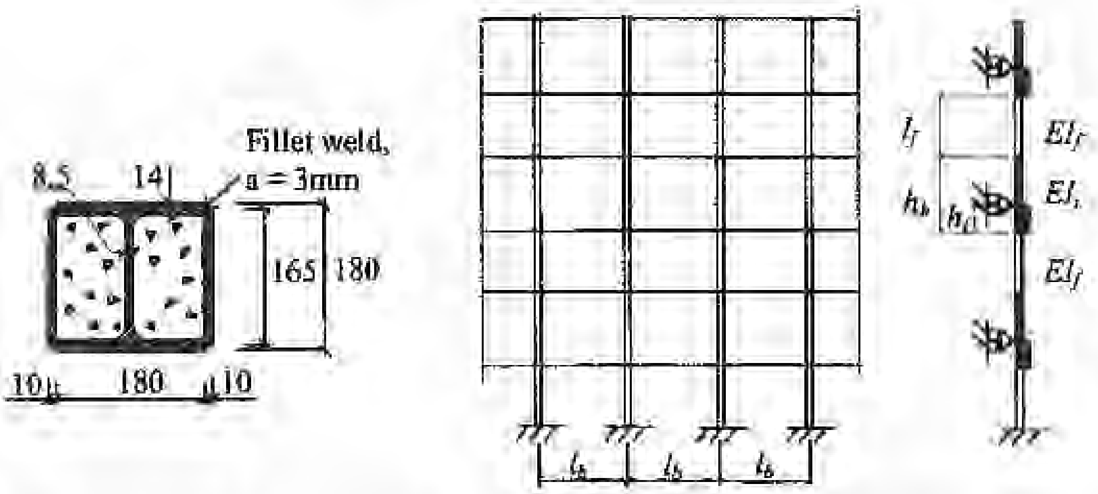


Figure 2 (a) Cross-section of columns; (b) Geometry and boundary of the façade columns.

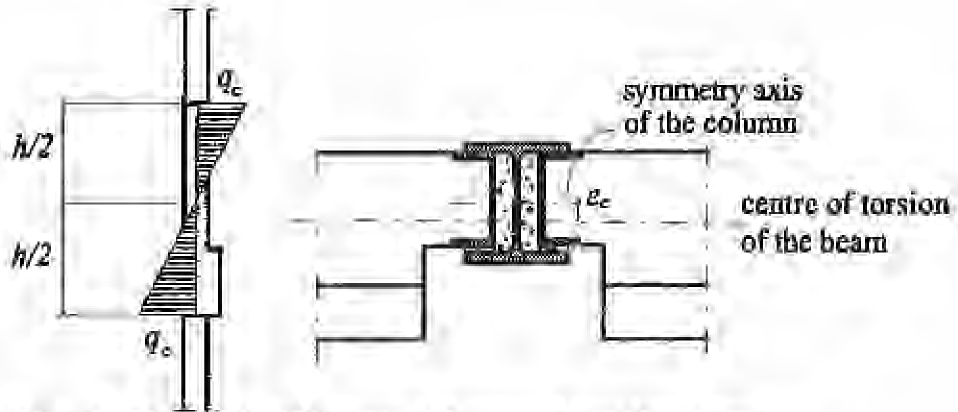


Figure 3 Connections between column and spandrel beams.

2.3 Connections between Spandrel Beams and Floor Elements

The connections between the spandrel beams and the floor elements are assumed to act continuously along the spandrel beams and to be semi-rigid. The connections can also transfer horizontal forces; these are approximated by rigid supports at the columns, see Fig. 2b. Although these forces act continuously along the beam, the model represents them only at the columns. The effect of the horizontal forces acting along the beam is further discussed in Section 4.2.

2.4 Floor Elements

The floor elements are assumed to have a linear strain distribution across the section. The response is elastic, and tensile stresses are disregarded in cracked sections. When the end rotation of the floor elements is calculated, only the additional deformation resulting from the working load q_w is taken into account. When the connection between the spandrel beam and the floor is grouted, the floor elements already have a deflection because of the dead weight. The corresponding end rotation does not contribute to any end moment. (The vertical load of the dead weight is, however, included in the analysis).

If the floor element is cracked, the cross-section is analysed in several sections. The bending stiffness is calculated for each section and the end rotation corresponding to the total load is found by integration. The floor element is assumed to be uncracked for the dead load only. The end rotation that causes moment is calculated from the total end rotation minus the rotation for dead load only.

3. PARAMETER VARIATION STUDY

3.1 General

The theoretical model described in Section 2 was used to analyse an example of a slender façade system. Basic input data were chosen to simulate a typical system with normal geometry and stiffnesses. Some parameters which were difficult to get information about were varied: whether the connections between beams and floor elements were cracked or not; the bending stiffness of the columns in the connection zone; and the eccentricity of the normal force from each floor.

Accidental situations were also analysed. The input data are described in Section 3.2, and the accidental situations studied are presented in Section 3.3. In the calculations, no safety factors were applied to load, capacity and stiffness values.

3.2 Input Data

3.2.1 Columns

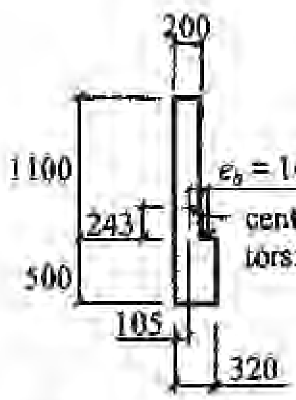
The tubular composite columns had a cross-section as shown in Fig. 2a, and the total number of floors was 8. The geometry and values of the column bending stiffness are listed in Table 1. The stiffness in the free part between the spandrel beams is based on the experimental studies of slender composite columns in Ref. /1/. The bending stiffness of the columns in the connection zone was evaluated to 8 MNm^2 in tests, and the eccentricity at the connection between a spandrel beam and a column, e_c, was evaluated to 19 mm, see section 5.1. It was, however, of interest to study how sensitive the system was for variations in these input data; therefore they were varied as shown in Table 1.

Table 1 Geometry and stiffness of columns

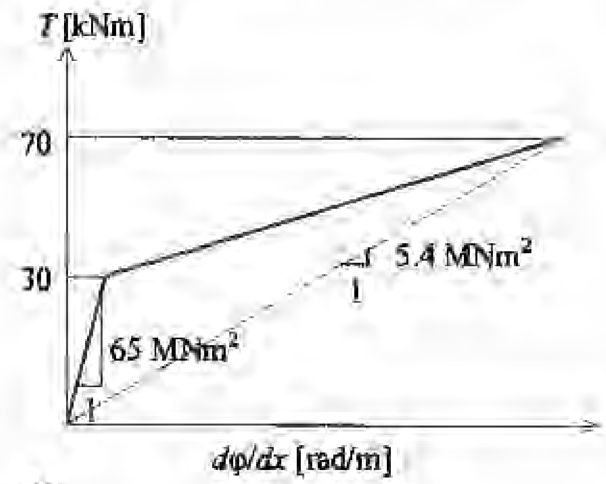
	l_f [m]	h_b [m]	h_i [m]	e_c [mm]	EI_f [MNm^2]	EI_i [MNm^2]
Basic input data	1.4	1.6	0.6	0	10	30
Variations	-	-	-	30; 50	-	5; 10; 100; $\rightarrow\infty$

3.2.2 Spandrel beams

The spandrel beams were assumed to have a length l_b of 6 m. Their cross-section and assumed torsional response is shown in Fig. 4. They had the same cross-section as the beams tested in Ref. /1/, and the torsional response was evaluated from the same experimental study, where concrete of grade C55 was used.



(a)



(b)

Figure 4 (a) Cross-section of the spandrel beams; (b) Assumed torsional response of the spandrel beams loaded in torsion, bending and shear

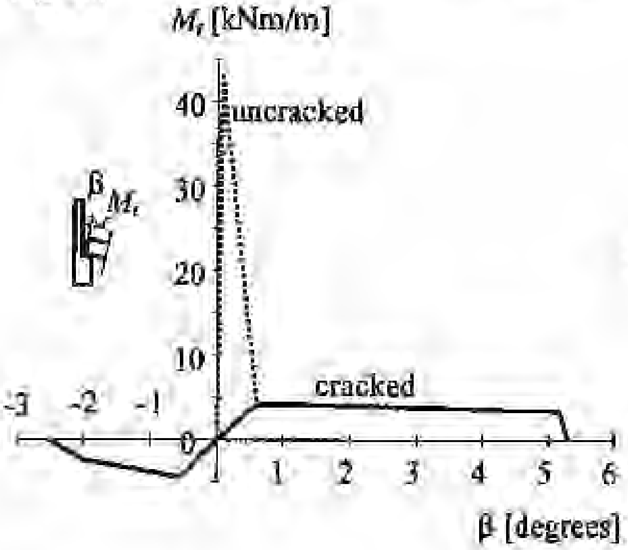
3.2.3 Connections between spandrel beams and floor elements

Two variations were checked for the connections between the spandrel beams and the floor elements; if they were cracked or not. When they were cracked (no adhesive bond), they were assumed to exhibit a characteristic behaviour as shown in Fig. 5a. The assumption is based on the experiments in Ref. /1/, and holds for the tie arrangement shown in Fig. 5b.

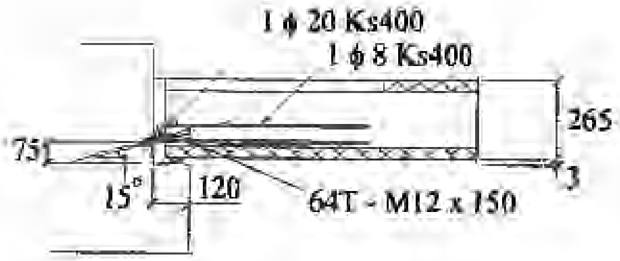
Since the tests in Ref. /1/ showed that the adhesive bond in the connections between the beams and the floors could be very strong, the effect on the structural system of this adhesive bond was analysed. According to the tests, the adhesive bond causes a cracking moment that corresponds to the tensile strength of the concrete in the joint. The cracking moment was calculated as

$$M_{cr} = \frac{f_{ctk,max} \cdot bh^2}{6} \tag{1}$$

The tensile strength of the joint concrete, $f_{ctk,max}$, was assumed to have a characteristic high value, as in Ref. /4/. The joint concrete was of grade C20, and the corresponding value of $f_{ctk,max}$ is 2.9 MPa. The characteristic behaviour of the connections was therefore modified according to Fig. 5a.



(a)



(b)

Figure 5 (a) Assumed behaviour of the beam-floor connection of ordinary type;
(b) Connection of ordinary type between spandrel beams and floor elements.

3.2.4 Floor elements

The floor was a hollow core one, type HDF 120/27. Each element had a width of 1.2 m, a depth of 0.265 m and a span of 12.0 m. The modulus of elasticity of the concrete was 30 GPa. The elements were prestressed; the reinforcement had an area of 500 mm², and a prestress of 1000 MPa. The working load on the floors was 1 kN/m². As an alternative, the working load on all eight floors was increased to 2.5 kN/m². This increased load was combined with all of the other variations used in the analysis.

3.3 Accidental Situations

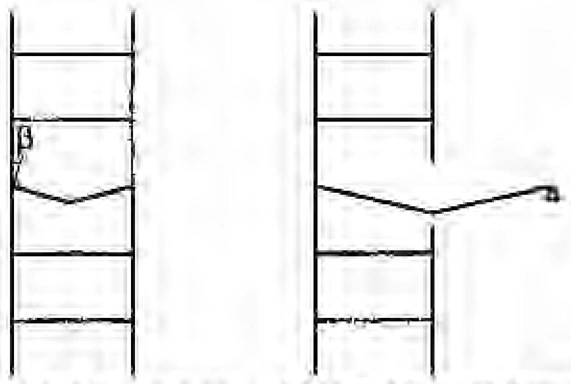
Three different accidental situations, cases A, B and C, were studied:

Case A At one floor, the beam-floor connection was just about to fail. It was strained to its maximum rotation capacity. The beam-floor connection was of an ordinary type, see Fig. 5b, with a characteristic behaviour as shown in Fig. 5a.

Case B All of the floor elements on one floor have lost their support on the interior side, see Fig. 6. The beam-floor connections were either of *improved type*; with high deformability and ductility, or of *deformable type*; with large deformability but ordinary resistance. With the improved type, it was possible to bridge an internal damaged support by catenary action. The weight of the hollow core floor system, without a working load, can be carried by the catenary system. Methods for the analysis of collapse situations are presented in Ref. /5/. This case is an extreme one for the façade, as the tensile force from the catenary system is transferred to the spandrel beam via the tie arrangements.

Case C All floor elements on one floor had collapsed, because of fire or other action, see Fig. 6. When fire is the cause, the bending stiffness for the column at the floor below the collapse is reduced to 70 % of the initial bending stiffness. Both the improved type and the deformable type of beam-floor connections were used.

With the improved connection, each floor element was tied to the edge support by 4 tie bars ϕ 16 Ss26 anchored in the centres of the cores. The characteristic behaviour of this connection was assumed to be as shown in Fig. 7. The behaviour was established by calculations according to the methods in Ref. /5/. The ultimate displacement of the connection loaded in pure tension was assumed to be 120 mm. The floor connections at the interior supports had four tie bars placed across the support.



Case A: the connection is stretched to its maximum capacity.

Case B: the floor elements have lost their support on the interior side.

Case C: the floor elements have collapsed.

Figure 6 Three different accidental situations.

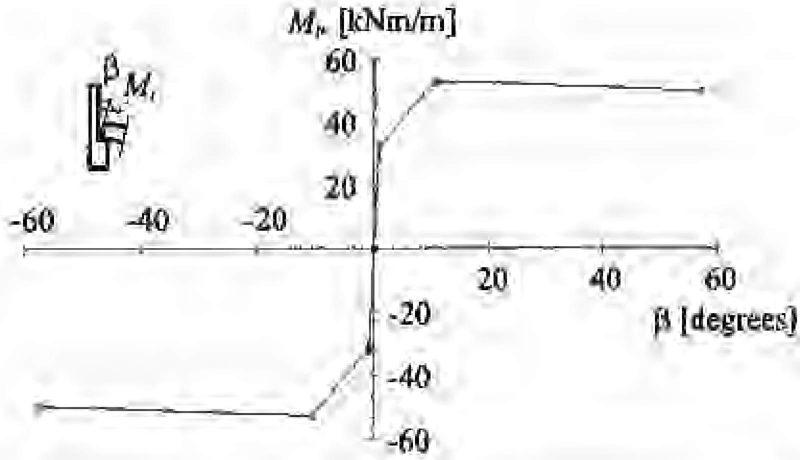


Figure 7 Assumed characteristic behaviour of the improved beam-floor connection with high deformability and ductility.

3.4 Results

The results from the analyses are given as equations describing the deformations and sectional moments and forces along the columns, beams, and floor elements. In Fig. 8, an example of the deflection and bending moment along the columns are shown. In the following sections, the main results from all analyses are presented.

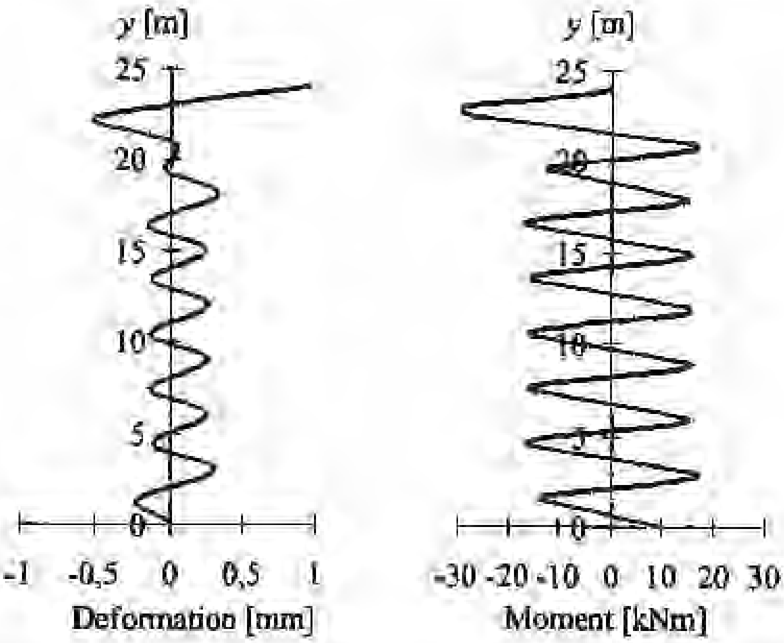


Figure 8 Deformation and bending moment along the columns. In this example, 1 kN/m² was applied on all floors, the beam-floor connections were assumed to be uncracked, and the basic input data were used for the columns. The thicker lines indicate the zones where the beams were connected.

3.4.1 Ordinary load cases

For all of the ordinary load cases examined, the torsional moment in the beams increased because of the action of the beam-floor connections. The beam-floor connections exhibited an elastic response in all of the cases, including the one with adhesive bond in the beam-floor connection. Since the connections remained uncracked, a high moment was transferred. This resulted in high torsional moment, large enough to crack the beams in combined bending, torsion and shear.

The façade system was not sensitive to variations in the bending stiffness EI_c of the columns in the connection zone; the effect of the assumed variations was negligible. An increased eccentricity of the normal force at the beam-column connection, e_c, resulted in a small additional effect on the columns. When e_c was 50 mm, the effect on the columns was increased to about 13 % more than when e_c was zero. This indicates that these factors do not have any significant effect on the structural system. When calculated deflections of the columns were compared with measured deflections in a full-scale test, the best agreement was found when the bending stiffness in the connection zone was set to 8 MNm^2, and the eccentricity at the connection between the spandrel beam and the column was 19 mm, see Section 5.1. The columns remained in the elastic stage for all of the load cases studied.

3.4.2 Accidental situations

For accidental Case A, with the beam-floor connection stretched to its maximum rotation capacity, the results were approximately the same as for Case C when the beam-floor connections used were of the deformable type. The torsional moment in the beams was relatively low, as was the effect on the columns. The action of the beam-floor connections resulted in increased torsional moments in Cases A and C, but not in Case B which had the improved connection. In Case B the connection transfers pure tension. Since the centre of torsion is above the connecting tie steel, the result is a torsional moment in the reverse of the normal direction.

When the accidental situations were analysed, the ordinary beam-floor connection could be expected to fail in Cases B and C. As a consequence, the floor elements would fall down onto the next floor, which probably could not resist this impact loading. This means that the connections would not be able to prevent a severe collapse in the actual situations.

With the improved connection acting to its capacity, such large torsional moment was obtained in the spandrel beams that their capacity was reached. This situation could cause failure, depending on the rotational deformation capacity of the beams. Appropriate information about the deformation capacity in torsional loading is essentially unavailable.

In all of the accidental situations studied, the bending moment in the columns was relatively low. The second order effects were small. Accordingly, the columns withstood all of the accidental situations while still remaining in the elastic range.

4 DISCUSSION OF THE THEORETICAL MODEL

4.1 Simplifications

The theoretical model takes into account the non-linearity of the components by partwise linear relationships between load and deformation. However, some other effects are not included:

- Horizontal forces can be transferred from the floors to the beams continuously along the beams. This effect is treated further in Section 4.2. The assumption made about rigid supports acting at the columns is shown to be an acceptable approximation.
- When the beam rotates, the eccentricity of the vertical load from the floor decreases, see Fig. 9. For the cases studied, this has little enough effect to be negligible.
- Long term effects can be very important; this should be investigated further.

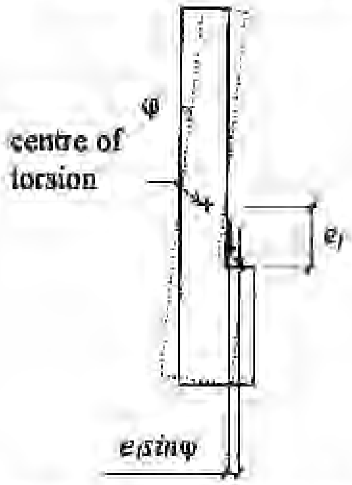


Figure 9 Reduction of the eccentricity of the vertical load from the floor, due to rotation of the beam.

4.2 Horizontal Forces

The connections between the spandrel beams and the floor elements are assumed, in the theoretical model, to transfer bending moment only. Actually, they can also transfer horizontal forces continuously along the beam. This effect was not taken into account in the model; instead, the horizontal restraint was formed by rigid supports at the columns. The effect of the horizontal forces acting along the beam was examined for the special case when the cross-sectional rotation and the vertical deflection of the beam were zero at the column.

The floor elements were assumed to be supported by spandrel beams at both ends. Only elastic elongation of the floor elements was taken into account, i. e. the horizontal deformation in the connections was disregarded, see Fig. 10. The transferred horizontal force can then be calculated as

$$N = 2\left(\frac{EA}{l}\right)_{slab}\delta \qquad (2)$$

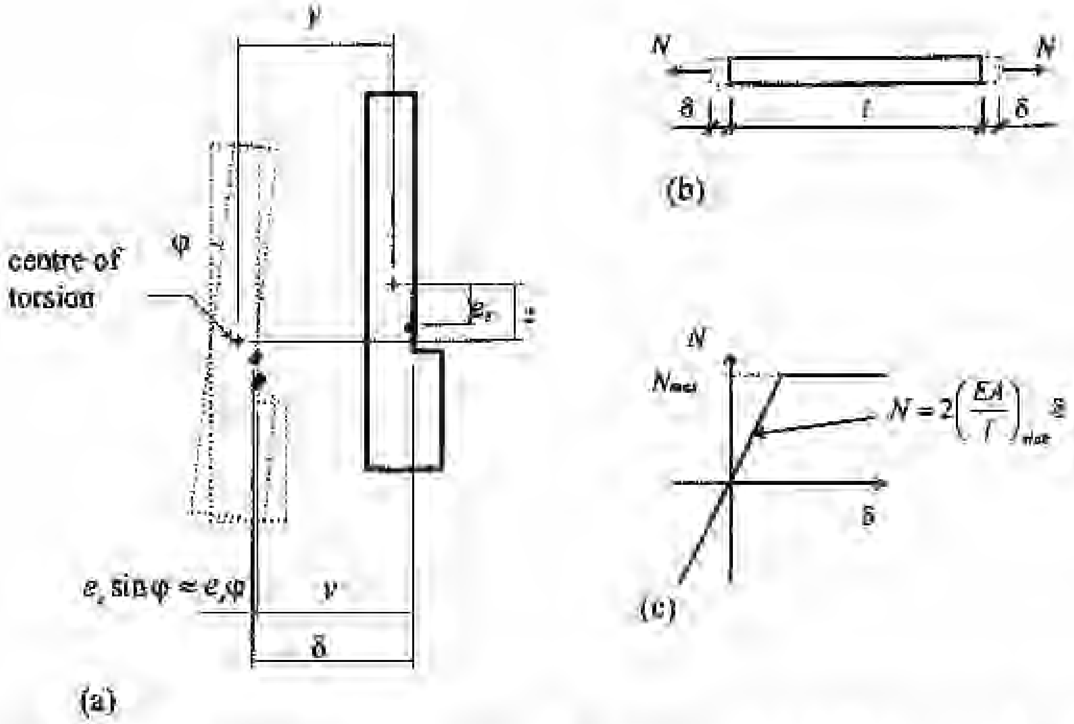


Figure 10 (a) Rotation and displacement of the cross-section; (b) Deformation of the floor; and (c) Relationship between horizontal force N and horizontal deformation δ (N_{max} is the upper limit of the connection).

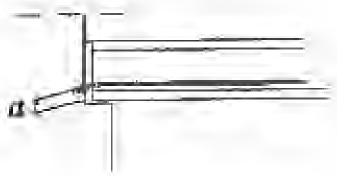
A limit of the transferred force was introduced, N_{max} , which depends on the limited capacity of the connection and on the transferred moment, M_t. If all tension forces are assumed to be transferred by the tie arrangements, N_{max} can be estimated as

$$\left.\begin{aligned} N_{max} &= N_{max,0} - \frac{M_t}{a} \\ N_{max,0} &= f_{sy}A_s\frac{n}{l} \end{aligned}\right\} \tag{3}$$

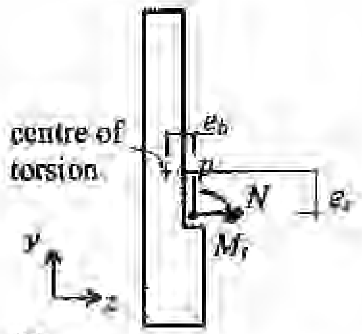
where $f_{sy}A_s$ is the maximum capacity of one tie bar, n is the number of bars along the beam, l is the length of the beam, and a is the inner lever arm in the connection, see Fig. 11a. Note that equation (3) is valid only when M_t is positive, as it was in the case studied.

The torsional load on the beam is decreased due to the horizontal force, see Fig. 11b. The torsional load can be calculated as

$$\frac{dT}{dx} = -pe_b - M_t + Ne_r \tag{4}$$



(a)



(b)

Figure 11 (a) Inner lever arm a in the connection; (b) Torsional load on the beam.

The deflection of the beam was analysed in order to find the deformation of the floor elements, δ. According to the theory of elasticity,

$$\frac{d^2y}{dx^2}=\frac{1}{E(I_yI_z-I_{yz}^2)}(I_{yz}M_y+I_yM_z). \tag{5}$$

Actually, the response of the beam is elastic only for low loads. When the beam cracks, the bending stiffness is reduced and the deformations are increased. As an approximation, the bending stiffness was assumed to be decreased to 30 % of the elastic values.

The moment, M_y, depends on the vertical load from the floor, p, which is constant along the beam:

$$M_y=\frac{px}{2}(l_b-x). \tag{6}$$

The moment M_z, which depends on the horizontal force N from the floor, varies along the beam:

$$\left.\begin{array}{l}\dfrac{dM_z}{dx}=V_z\\ \dfrac{dV_z}{dx}=N\end{array}\right\}\Rightarrow\frac{d^2M_z}{dx^2}=N. \tag{7}$$

The problem was solved by numerical integration along the beam. The same basic input data was used as in the analysis of the entire system, see Section 3.2. Two cases were studied: with and without adhesive bond in the connection between beam and floor.

The analysis showed that the upper limit of the connection N_{max} was not reached for either case. Some results from the case without adhesive bond are shown in Fig. 12. They show that the horizontal deflection of the beam is effectively prevented by the connections. But the torsional moment is only slightly influenced by the transferred horizontal forces. The same conclusion was drawn for the other case studied, the one with adhesive bond. Accordingly, it could be concluded that the horizontal forces acting continuously along the beam do not have any significant effect on the system as a whole. Hence, the assumption about rigid supports acting at the columns was shown to be an acceptable approximation.

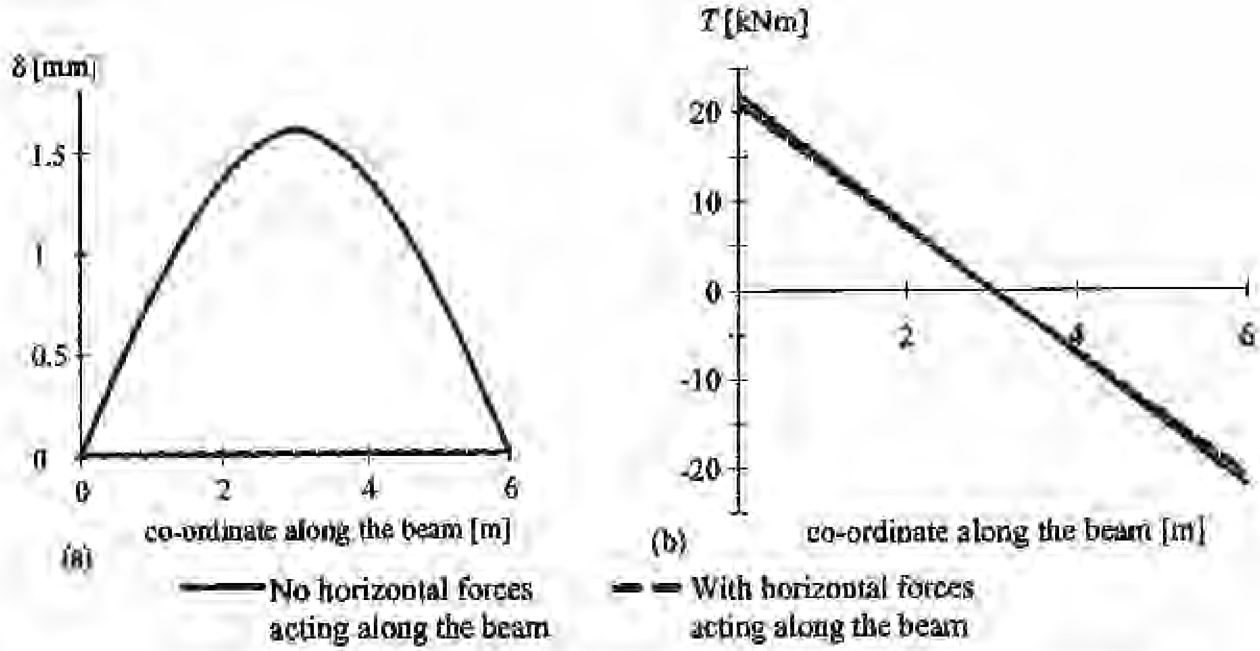


Figure 12 (a) Calculated horizontal deflection of the beam where the floor elements are connected, δ, versus a co-ordinate along the beam (The connection is cracked); (b) Torsional moment along the beam, T, versus a co-ordinate along the beam (The connection is cracked).

5 COMPARISON BETWEEN THE THEORETICAL MODEL AND TESTS

5.1 Deflection of Columns

The theoretical model was verified by analyses of parts of the façade that were tested experimentally. In the experimental studies that were a part of this work, two test specimens consisting of the end zones of two adjacent spandrel beams and a column were tested, see Fig. 13. The length of the column was determined by the distance between two assumed points of contra-flexure. The test specimens were loaded by one concentrated load on the ledge of each beam element. The loading on the beam elements was balanced by passive horizontal supports at the upper corner of the outermost edges of the beam elements. For more information about test details, see Ref. /1/.

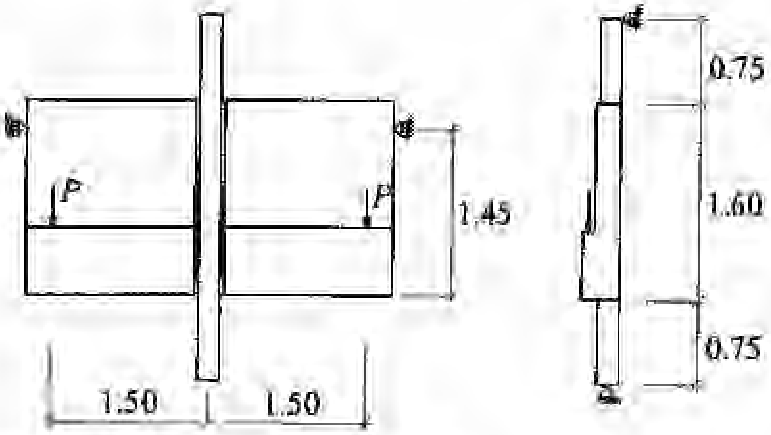
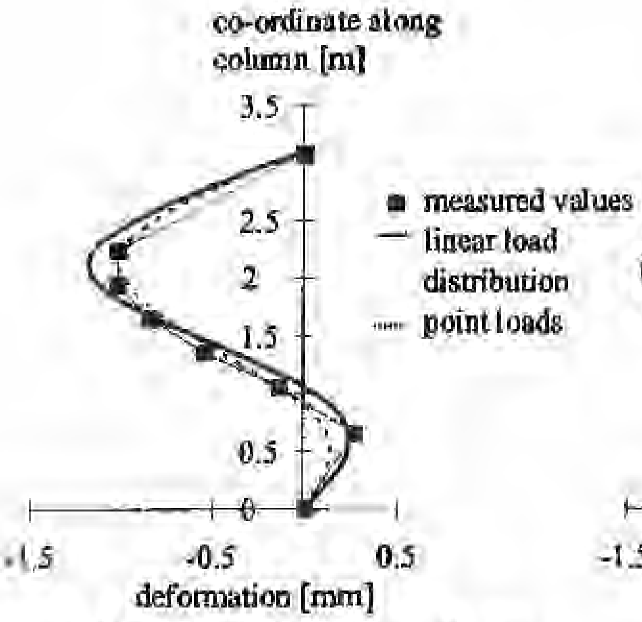


Figure 13 Principle of the test arrangement.

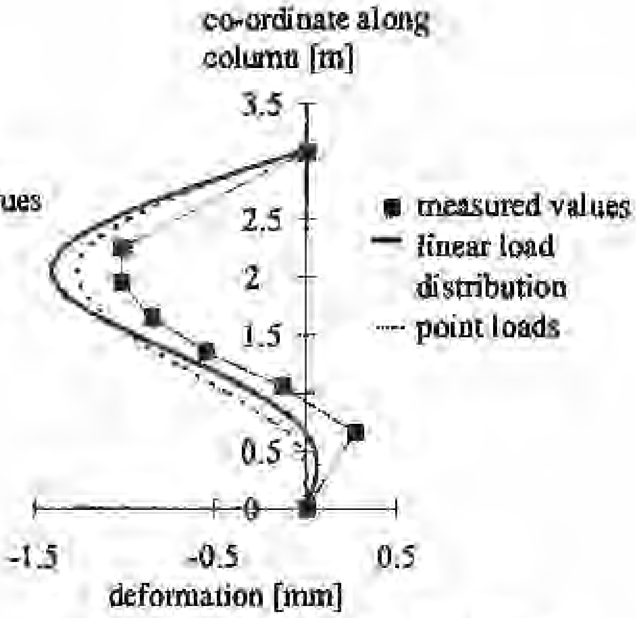
The deflection of the column was calculated according to the assumed model of the connection between spandrel beam and column, see Section 2.2. Two possible ways for the torsional moment to be transferred from the beam to the column were checked: by a linear load distribution (as in the model of the entire system), or by point loads acting at the edges of the beam, see Fig. 14d. The normal force from the beam was transferred to the column with an eccentricity e_c. The column was assumed to be linear elastic. Some examples of calculated column deflection compared with measured displacements are shown in Fig. 14.

At the free parts of the column, the bending stiffness was set to 10.5 MNm^2, as evaluated from the experiments on columns, see Ref. /1/. In the connection zone, the beams were believed at first to have a stiffening effect on the column. However, when comparing measured and calculated deflections, it was found instead that the bending stiffness decreased in comparison with the free parts. A value of 8.0 MNm^2, which corresponds to the steel H-section only, gave good results. The best agreement between calculated and measured deflections was found when using the linear load distribution. The analyses showed good agreement for loads up to about half of the maximum load. For higher loads, the agreement was still acceptable, see Fig. 14c.

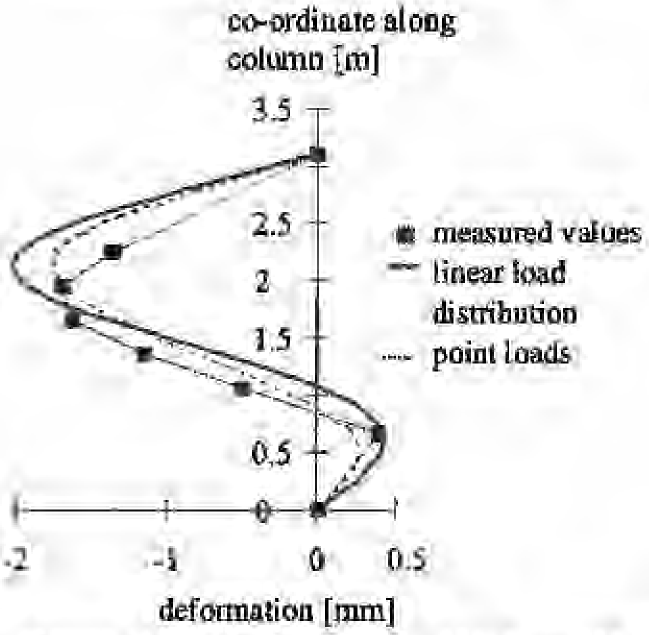
The distance between the theoretical centre of torsion in the beam and the symmetry axis of the column is actually 29 mm. However, better agreement was found when this eccentricity was set to 19 mm. The torsional moment to be transferred was increased by a corresponding amount. A probable explanation is that the centre of torsion is affected by the detailing at the connection to the column, especially by the recess at the beam end, see Fig. 15.



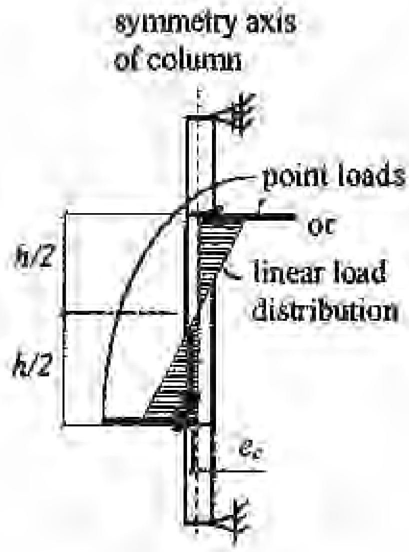
(a) Vertical load 303 kN, e_c = 19 mm.



(b) Vertical load 303 kN, e_c = 29 mm.



c) Vertical load 523 kN (maximum load), e_c = 19 mm.



(d) Model of column.

Figure 14 (a), (b) and (c) Comparison of measured and calculated deflections; (d) Model of column. Two different ways for transferring the torsional moment from the beam to the column were checked: linear load distribution and point loads.

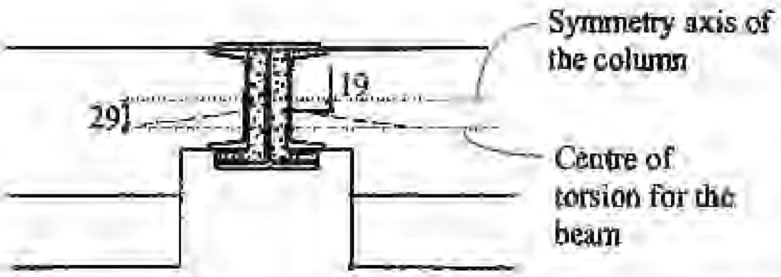


Figure 15 The centre of torsion tends to move towards the symmetry axis of the column, at the edge of the beam.

5.2 Composite Action between Floors and Beams

Elliott, Davies and Adlparvar in Nottingham have carried out tests on the composite action between floor elements and L-shaped edge beams, see Ref. /6/. Although the edge beams in their investigation are much smaller than those studied here, the same theoretical model ought to be applicable; therefore, their tests have been analysed.

Three tests were performed, C1a, C1b and C2. The principal test arrangement is shown in Fig. 16. The test specimen consisted of hollow core elements that were placed between two edge beams, which, in turn, were supported by columns. A uniform line load was applied asymmetrically on the floor elements in tests C1a and C1b. In test C2, there were two symmetrical uniform line loads. The edge beams in test C2 were of slightly different geometry from those used in tests C1a and C1b. In test C1a, one end of the floor elements rested on rollers, while in tests C1b and C2, both ends had concreted connections.

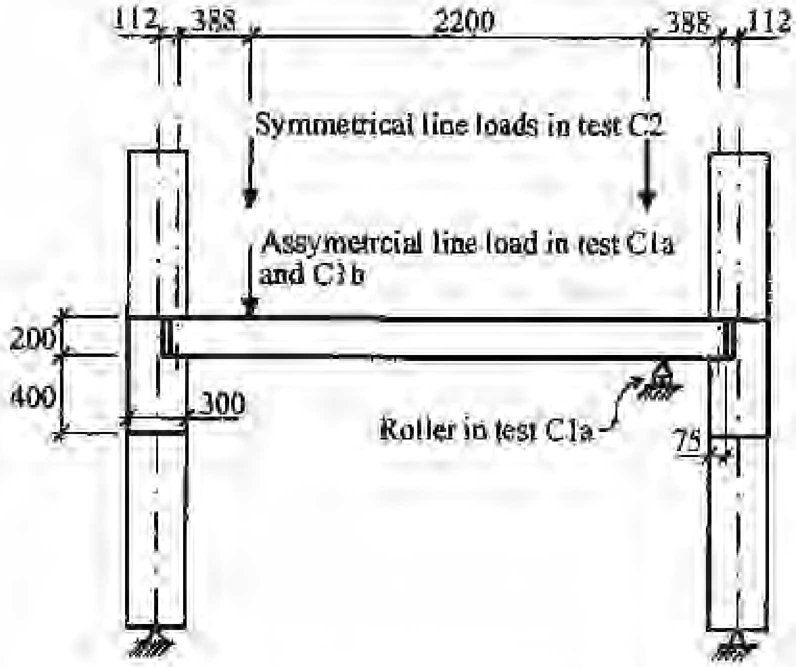
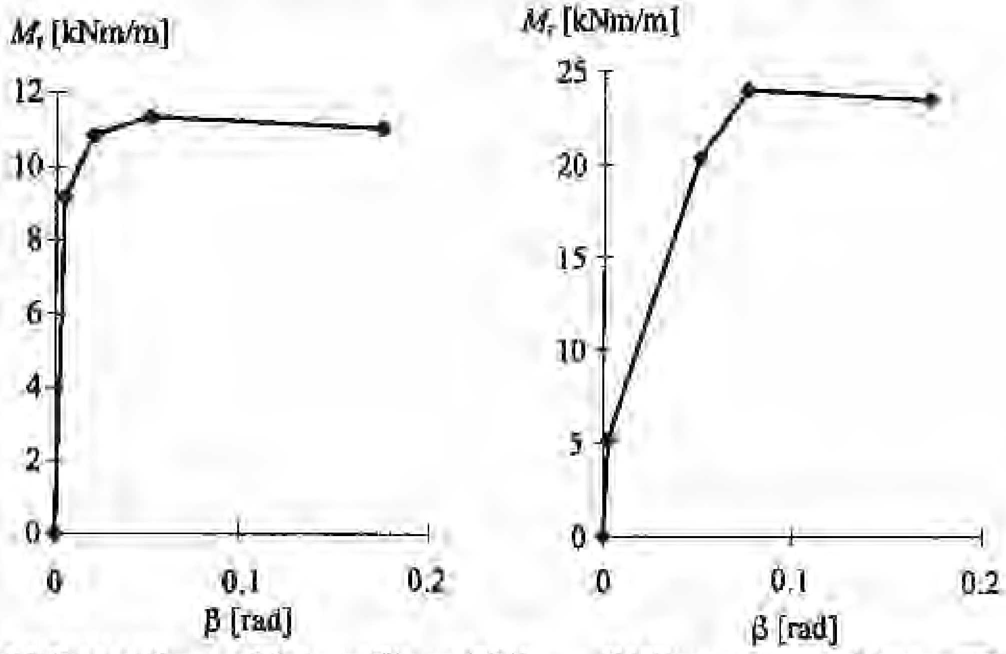


Figure 16 Principle of the test arrangement, Ref. /6/.

Beams similar to those used in the composite tests have been tested separately by Elliot *et al.* /7/. Values of their torsional stiffness were determined from these tests. The columns and the connections between the columns and the edge beams have not been analysed here. Instead, the measured values of the cross-sectional rotations of the beams at the ends were used as input values. The reason for this was that the beam-column connections were quite different from the ones assumed in the theoretical model.

As the connections between the beams and the floor elements had not been tested separately, the relationship between transferred moment M_t and rotation β was estimated by calculations according to Ref. /5/. The connections reported there were symmetrical, so that the tie bars could slip on both sides. Here, they could slip mainly on only one side; therefore, the calculated rotations were divided by two. The assumed relationships between transferred moment and rotation used in the analyses of the subframes are shown in Fig. 17.



(a) Connection used in tests C1a and C1b. (b) Connection used in test C2.

Figure 17 Estimated relationships between transferred moment M_t and rotation β used in the analyses.

The results of the analyses are shown in Figs. 18 and 19, where the total applied load is plotted versus the rotation of the beam in the midsection. The theoretical cases of the simply supported floor and the floor fully restrained to the beam are also plotted. Unfortunately, for the subframes tested, the composite action had very little influence. For these spans and stiffnesses, the torsional rotation of the beam is about the same as the end rotation of the floor. Consequently, all of the curves calculated are very close to each other.

The calculated responses show a good agreement with the measured ones for low loads. For higher loads, the calculated rotations are too low in tests C1a and C1b. This was probably due to the failure mode which, in all three tests, was shear failure of the floor elements; the theoretical analysis does not include shear deformations. When the floor elements were close to shear failure, it would have been more appropriate to assume that they were "hanging" in the connection to the beam. This means that the maximum moment capacity of the tie connections would have been reached along the whole beam. When the rotation of the beam was calculated in this way in tests C1a and C1b, good agreement was found for the maximum load. For test C2, the calculated rotation at maximum load was a bit too high. In this test, although the failure mode was still shear failure of the floor elements, the connections between the columns and the beams were also close to failure. Therefore, the beams had large rotations at the ends and the angle between the beams and the floor elements, β, was not large enough to cause the maximum moment in the tie connections.

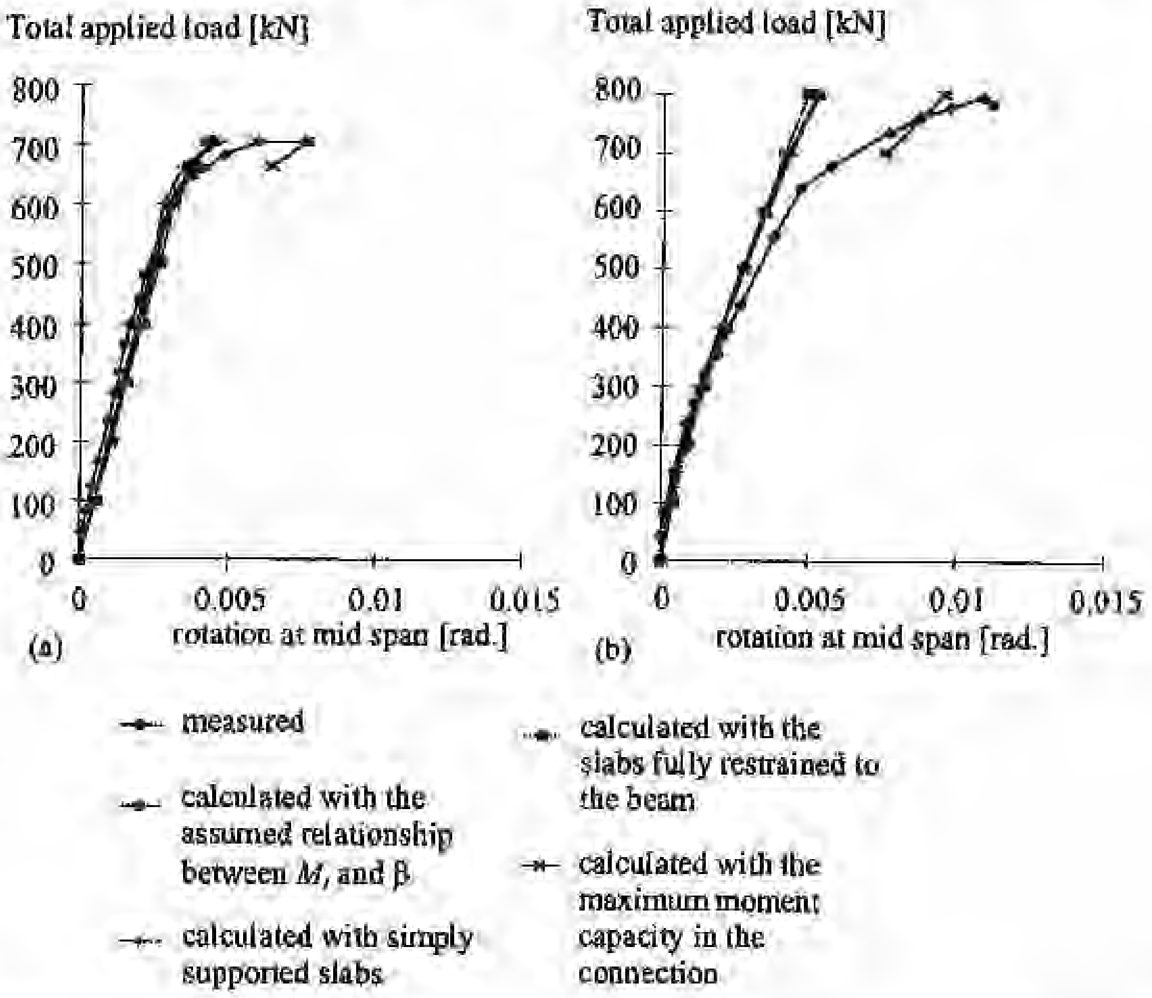


Figure 18 Total applied load versus sectional rotation at mid span in (a) test C1a; (b) test C1b.

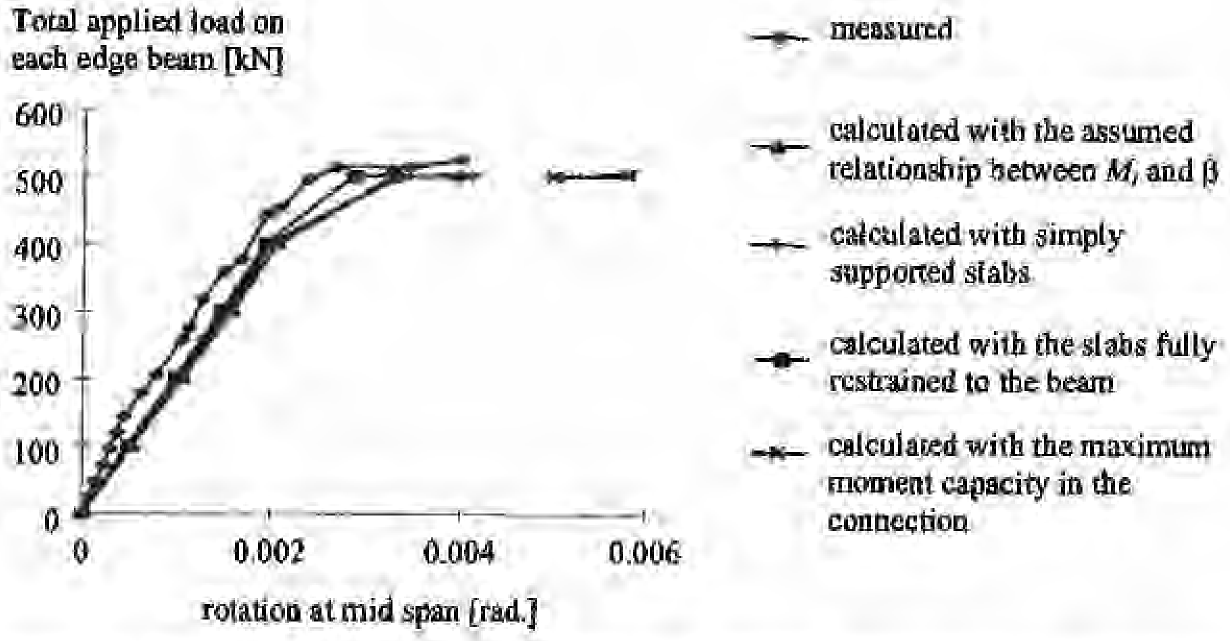


Figure 19 Total applied load on each edge beam versus sectional rotation at mid span in test C2.

6 CONCLUSIONS

The behaviour of a load-bearing façade, such as the one studied here, is a result of a balance between the structural components, including the connections. If the columns are very weak, the beams will have large rotations and the torsion will be counteracted by the floors. On the other hand, if the floors have very long spans, they will have large end rotations and the torsion in the beams will be increased. Since all of the components show linear responses only for low loads, the balance between them changes for different load situations. This makes it necessary to have an understanding of the complete structure when the components are designed. Simplified design recommendations, for example about the need to reinforce edge beams against torsion, must always clearly specify the spans and stiffnesses for which they are valid.

For the façade analysed here, the torsional moment in the spandrel beams increased due to the action of the beam-floor connections in all of the ordinary load cases studied. When there is an adhesive bond at the joint interfaces, these connections can remain uncracked, which would result in the transfer of large moments. The torsional moment in the beams was found to be more than doubled compared with the case when the floor elements were assumed to be simply supported on the beam. For ordinary load cases, the resulting torsional moment in the beams could be great enough to result in torsional cracking of the beams.

In accidental situations, for example a collapse of the floor elements on one storey, the analyses showed that the beam-floor connections of the types used today would fracture. As a consequence, the floor elements would fall down onto the next floor, which probably could not resist this impact loading. This means that the connections could not prevent a severe collapse in the accidental situations. Therefore, a connection of improved type, with high deformability and ductility, was assumed. However, when this improved type was used, a very high ultimate capacity of the connection resulted in too large torsional moments in the spandrel beams. This situation could be dangerous, depending on the deformation capacity of the beams. Accordingly, the ultimate capacity of the beam-floor connections must be limited to avoid large torsional moments in the beams, while still remaining great enough to slow falling floor elements, for which high ductility and deformability are required. However, even in accidental situations, the analyses showed that the columns remained in the elastic stage.

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