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STIFFENING ANALYSIS OF BUILDINGS ERECTED IN THE IMS SYSTEM

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Abstract

The bracing of buildings of the prefabricated and prestressed IMS skeleton system is provided by sway-to-shear and sway-to-bending structural members. An analysis of their contribution to the lateral stiffness is presented in this study. The adopted method is subjected to N.Rosman's theory. An illustrating numerical analysis reveals that a casual deficiency of the framework rigidity of the nodes due to decrease of prestressing effect can cause a remarkable redistribution in the loading conditions of the bracing diaphragms.

Keywords: : prefabricated skeleton, prestressed nodes, bracing diaphragms, interaction between shear and bending stiffnesses, redistribution due to decrease of prestress.

Introduction

In the last two years in Hungary the necessity to check a great number of buildings erected in the prefabricated IMS system arose. This type of structure can be characterised by a special slab-to-column connection in which prefabricated ribbed floor panels are attached through prestressing to solid multistorey columns of rectangular cross-section. Damages on the connections due to chloride corrosion have been detected in ultimate time causing a considerable decrease of prestressing force and affecting thereby the stability and strength of the multistorey framework. The majority of the buildings in concern is of moderate height (4-6 storeys) and have a 'semi-sway' skeleton structure where the flexural stiffness of the nodes has been taken into account when checking its lateral stability.

During the statical review of these buildings we encountered the problem to analyse under realistic and and not too conservative asssumptions their lateral stability. In the following we shall attempt to summarize some conclusions gained in this work.

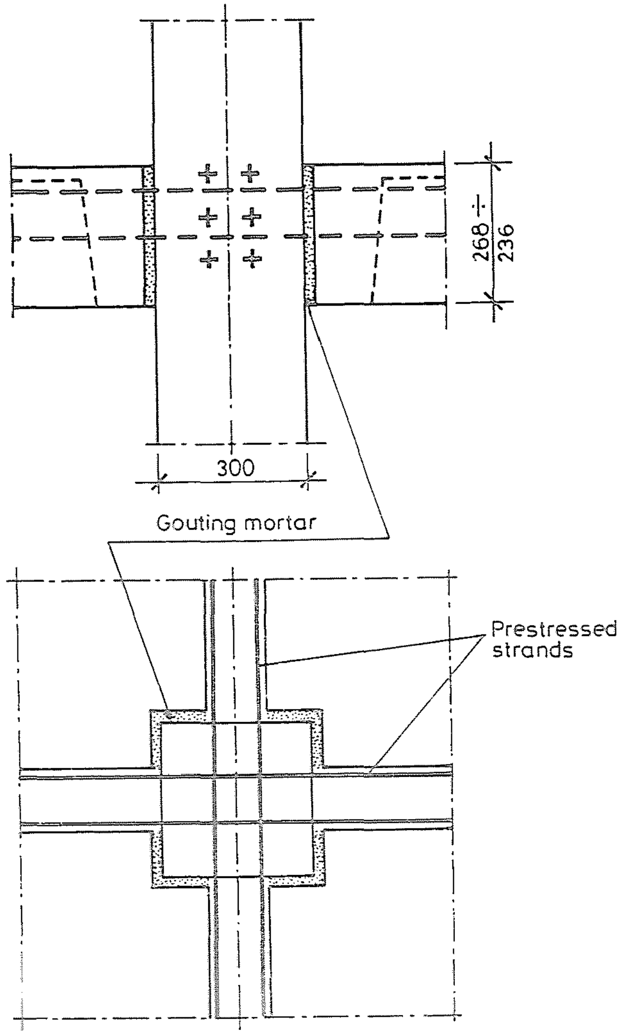

Fig. 1. Scheme of the column-floor connection

Structural Data

The IMS system operates with a two-way network of columns with a mesh of 3.60×3.60 up to 7.20 × 7.20 m. The prefabricated, ribbed and bottom-finished floor panels are of 236 or 268 mm thickness. They are pressed at their corners to the sides of 300×300 mm columns. The prestressing force is supplied by tendons of 3×∅ 17 mm wires. The nominal prestressing force varies between 125 and 130 kN, and has a negative eccentricity at the nodes in order to provide continuity in the slab under vertical loads (*Fig. 1*).

The statical scheme of the multi-storey IMS skeleton is visualized on *Fig. 2*.

The lateral stiffening of the structure has several components:

a./ the floor-column connection
b./ monolithic shear walls
c./ prefabricated concrete wall panels,
d./ walls in masonry

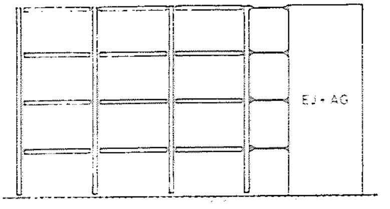

Fig. 2. Statical model

These components are of yielding-to-shear and of yielding-to-bending character. To the former group belong the items a./, c./ and d./, while to the latter belongs component b./. The effectiveness of the floor-column connection (a./) is greatly dependent on the actual value of the prestressing force. This interaction will be analysed in the next chapter.

Analysis of Stiffnesses

The Floor-Column Connection

Two phases in the mechanical behaviour of a node (*Fig. 2*) can be distinguished, the state of compression and that of decompression (*Fig. 3*). Let this – maybe a bit arbitrary – distinction denote that change of status in which the full contact between the surfaces of column and that of the floor panel begins to degrade and gaps appear. Once the moment of decompression is not reached, the connection behaves as a monolithic one, consequently, its resistance to horizontal displacement calculated under the customary condition that $J_{beam} \gg J_{column}$ is (*Fig. 4*):

$$\frac{Q}{\Delta} = \frac{12EJ_{col}}{m^3} \tag{1}$$

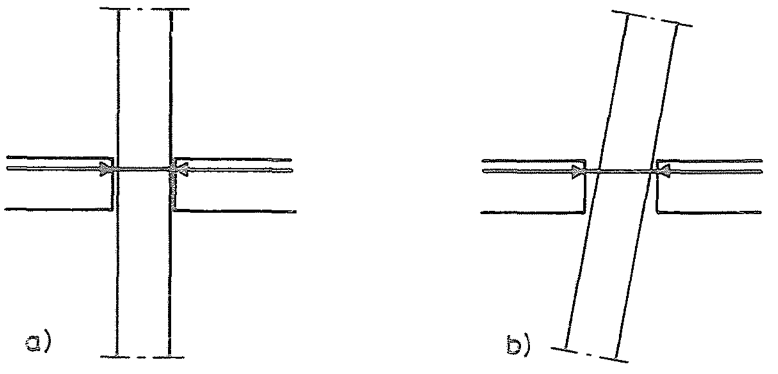

Fig. 3. States of compression and decompression

or

$$\frac{Q}{\varphi} = \frac{12EJ_{col}}{m^2} \tag{2a}$$

and

$$\frac{M}{\varphi} = \frac{6EJ_{col}}{m}, \tag{2b}$$

where m denotes the storey height and EJ_{col} the flexural stiffness of one column. Δ is the relative horizontal displacement between two storeys and φ the angle of inclination to the vertical.

The critical moment of decompression:

$$M_{decomp} = A_p \sigma_{p\infty} h,$$

with

A_p – as cross-sectional area of prestressing steel,
$\sigma_{p\infty}$ – as effective stress of prestressing and
h – as total depth of the floor slab.

In the state of decompression a widening of the gap by Δa between the surfaces of contact causes an increment in the prestressing force H, presumed that no effective bond is acting between prestressing strands and the grouting concrete of joint.

$$\Delta a = h.\varphi,$$

$$H = A_p \sigma_{p\infty} + (A_p E_p + A_s E_s)\frac{\Delta a}{a} = A_p \sigma_{p\infty} + (A_p E_p + A_s E_s)\frac{h}{a}\varphi.$$

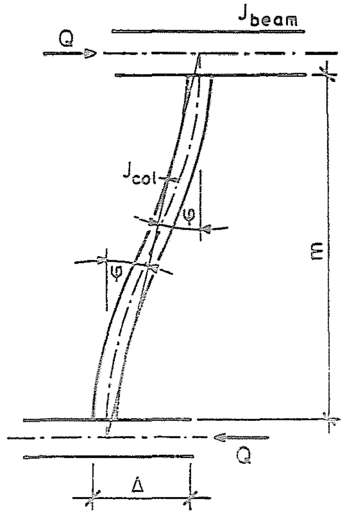

Fig. 4. Column deformation in state a./

where the further notations are:

A_p, A_s – cross-sectional area of the prestressing wires, and non-prestressed steel connectors, resp. which latter are sticking out re-bars of the panels welded together;

a – spacing of the columns

E_p, E_s – moduli of elasticity of the wires and connectors, respectively.

Thus the moment of resistance:

$$M = h.H = A_p \sigma_{p\infty} h + (A_p E_p + A_s E_s) \frac{h^2}{a} \varphi.$$

The corresponding bilinear characteristic is shown in *Fig. 5.* A numerical evaluation of these parameters proves that under current conditions the post-decompression branch of the M vs. φ curve is flat, so the resulting stiffness of the structure against additional lateral movement will drop to a negligible value. By this reason excess of the moment of decompression should not be allowed. Therefore, when assessing the lateral stiffness of the framework, the phase prior to decompression may be regarded.

On the basis of the above considerations an equivalent 'smeared' shear stiffness can be defined, derived from relation (1), for the total of columns:

$$(AG)_{frame} = \frac{12E \sum J_{col}}{m^2}.$$

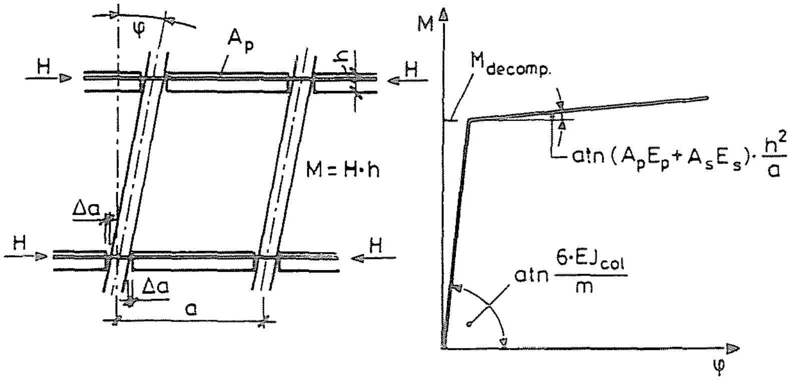

Fig. 5. Additional column deformation in state b./

Solid Diaphragms

– Concrete shear walls

$$(AG)_c = 0.4 \ \ E\sum A_c$$

– Shear walls in masonry

$$(AG)_{mas} = 0.4 \ \ \delta S\sum A_{mas},$$

where the notations are
$\sum A_c, \sum A_{mas}$ – the total cross-sectional area of the concrete, and masonry diaphragms resp.
δ – the modular ratio masonry vs. concrete.

The above shear stiffness components can be summarized into one replacement parameter:

$$(AG)^* = (AG)_{frame} + (AG)_c + (AG)_{mas}.$$

The summarized flexural stiffness of all properly reinforced, yielding-to-bending concrete walls may be found in a similar way

$$(EJ)^* = E\sum J_{wall}.$$

Distribution of Forces with the Rosman Method

The stiffening problem of multi-storey skeletons braced by shear- and bending-diaphragms respectively is thoroughly discussed in [1]. The partition of horizontal loads between two systems of bracing members is charged with problem of the non-affin character of the corresponding displacement curves. The solution given by Rosman is based upon the minimum condition of complementary deformation work and as a result yields the following procedure of computation.

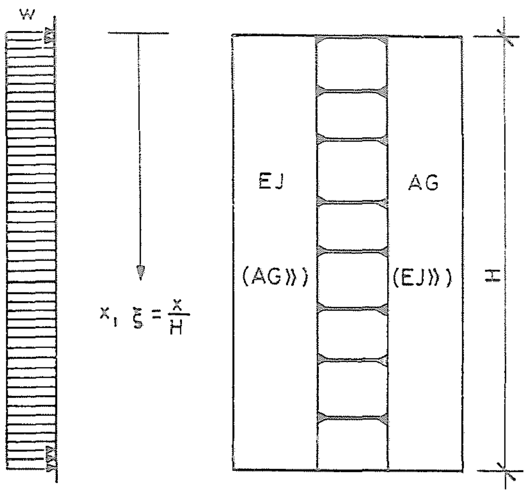

Fig. 6. The statical model in Rosman's method

Let's consider the continuous model consisting of a pair of coupled bending and shear diaphragms which are connected by the series of rigid floor slabs. The shear forces and bending moments are jointly resisted by the two components:

$$Q_{AG} + Q_{EJ} = Q, \tag{3a}$$

$$M_{AG} + M_{EJ} = M, \tag{3b}$$

where on the righthand side the actions due to external loading stand. Taking the function M_{EJ} as unknown – under the condition of the joint action of the two cantilever members – the governing differential equation will result in the form:

$$M''_{EJ} - \alpha^2 M_{EJ} = w, \tag{4}$$

where
$\alpha = \frac{(AG)^*}{(EJ)}$ – the stiffness parameter, and
w – the load function.
For the load case $w(x) = \text{const}$, the solution will be:

$$M_{EJ} = \frac{w\varsigma^2}{A^2}\Big[\frac{A - shA}{chA}shA\xi + chA\xi - 1\Big] \tag{5}$$

and from this the shear force by derivation:

$$Q_{EJ} = \frac{w\varsigma^2}{A^2}\Big[\frac{A - shA}{chA}shA\xi + shA\xi\Big] \tag{6}$$

with $\xi = \frac{x}{\varsigma}$, and $A = \alpha\varsigma = \sqrt{\frac{(AG)^*}{(EJ)^*}}\varsigma$,
being ς the total height of the skeleton.

Actions in the shear-resisting members can be obtained from the equilibrium equations:

$$M_{AG} = M_{tot} - M_{EJ} = \frac{w\varsigma^2}{2}\xi^2 - M_{EJ}, \tag{7}$$

$$Q_{AG} = Q_{tot} - Q_{EJ} = w\varsigma\xi - Q_{EJ}. \tag{8}$$

And finally the lateral displacement on the top of the cantilevers:

$$\Delta = \frac{\varsigma Q_{AG,max}}{(AG)^*}[1 - \frac{2}{A^2}(\frac{1}{chA} + A\frac{shA}{chA})].$$

The above procedure is focused on the problem of a compatible joint action between shear and flexural stiffening members and does not consider the effects of torsion due to asymmetrical arrangements. Nevertheless, for a class of buildings forming a great part of the IMS houses this method is a useful tool for a rapid assessment of stability reserves. This will be demonstrated through a short numerical example.

Numerical Example

A 5-s storey 'IMS' skeleton building is to be considered (*Fig.7*):

a./ Geometrical and structural data:
- Sizes in plan: 86.40×14.40 m

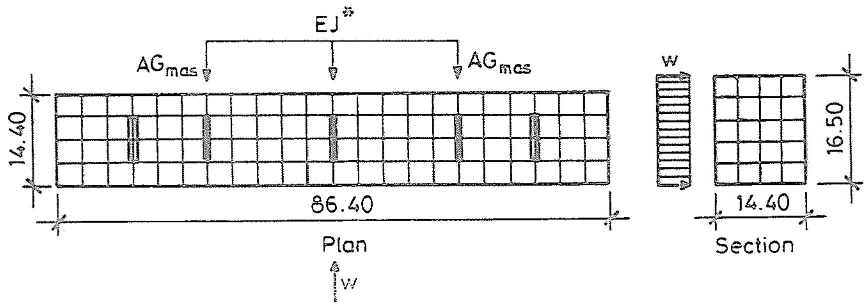

Fig. 7. Scheme of the building

- Height: $H = 16.50$ m
- Storey height: $m = 3.30$ m
- Columns
  - cross-section : 300 × 300 mm
  - moment of inertia : $0.675.10^{-3}$ m^4
  - total number: 5 × 21 = 105
- Solid diaphragms in the shorter direction: shear walls in masonry: 2 × 3.30 × 0.30 m, bending walls in concrete: 6 × 3.30 × 0.12 m, arrangement: symmetrical
- Floor slabs,
  - total depth: 236 mm
  - prestressing steel: 4 × 3∅7 / column

  $A_p = 462$ mm^2

  $\sigma_{p\infty} = 900$N/mm^2
  - steel connectors: 4 × ∅12 / column

  $A_s = 452$ mm^2

b./ Moduli of elasticity
- concrete: $E = 15.10^3$ N/mm^2
- masonry: $E_{mas} = 10^3$N/mm^2($\delta = 1/15$)
- steel: $E_s = 206.10^3$ N/mm^2
- prestr.steel:
  - $E_p = 195.10^3$ N/mm^2

c./ Load:
- wind load increased by a safety factor (resultant over the longer side)

$w = 86.40 \times 1.2 \times 1.2 \times 0.80 = 99.5$ kN/m

d./ The stiffness parameters:

- $(AG)_{frame} = 105 \times 0.675 \cdot 10^{-3} \times 12 \times 15.10^6/3.30^2 = 1.171 \cdot 10^6$ kN
- $(AG)_{mas} = 2 \times 0.4 \times 3.30 \times 0.30 \times 15.10^6/15 = 0.792 \cdot 10$ kN
- $(AG)_c = 0$
- $(AG)^* = 1.963 \cdot 10^6$ kN
- $(EJ)^* = 6 \times 3.30^3 \times 0.12/12 \times 15.10^6 = 32.34 \cdot 10^6$ kNm^2

f./ Analysis of actions

The computational results obtained with formulae (5) to (8) are summarized in *Tables 1* and *2*.

Case I: Full framework effect $[(AG)_{frame} = 100\ \%]$,

Table 1

$\xi = \frac{x}{\varsigma}$	M_{EJ} (kNm)	Q_{EJ} (kN)	M_{AG} (kNm)	Q_{AG} (kN)
0	0	−347	0	347
0.2	−704	−103	1246	431
0.4	−756	70	2923	587
0.6	−190	291	5066	694
0.8	1387	716	7281	597
1.0	5076	1642	8468	0

maximum displacement on the top: $\Delta_{max} = 9$ mm

Case II: No framework effect $[(AG)_{frame} = 0]$

Table 2

$\xi = \frac{x}{\varsigma}$	M_{EJ} (kNm)	Q_{EJ} (kN)	M_{AG} (kNm)	Q_{AG} (kN)
0	0	−382	0	382
0.2	−762	−91	1304	419
0.4	−625	176	2792	481
0.6	450	490	4426	495
0.8	2755	938	5913	375
1.0	6920	1642	6624	0

maximum displacement on the top: $\Delta_{max} = 17$ mm

The corresponding M and Q plots are given in *Fig.8*.

g./ Interpretation of the results with respect to the framework connections.

– The maximum shear force acting on the total of sheared members

$$Q_{AG,max} = 694 \text{ kN}$$

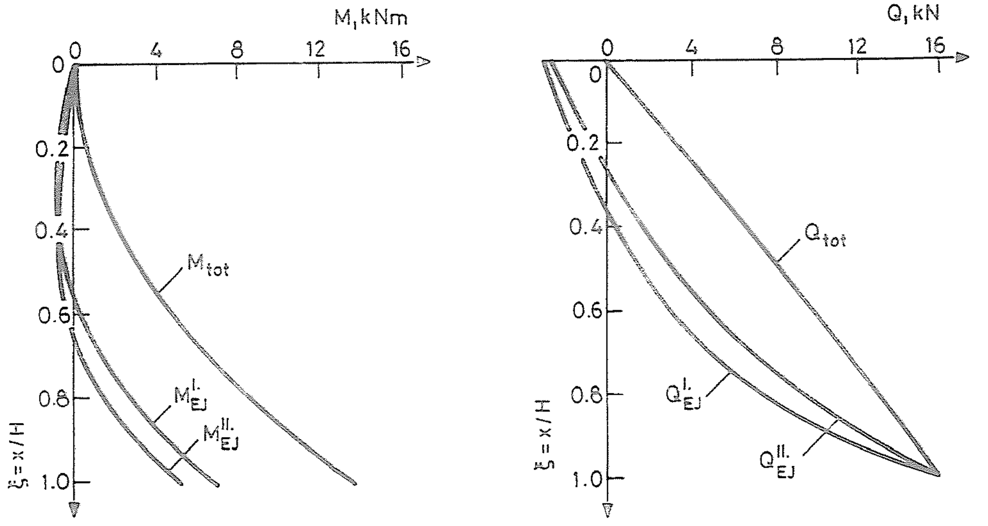

Fig. 8. Moment- and shear force diagrams

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$$Q_{AG,max} \frac{(AG)_{frame}}{(AG)^*} = 414 \text{ kN}$$

Action on one column:

$$Q_{col,max} = \frac{414}{105} = 3.95\text{kN}$$

Bending moment acting at the column-floor connection :

$$M_{node} = Q_{col,max}.m = 11.85\text{kNm}$$

The moment of decompression :

$$M_{decomp} = A_p \sigma_{p\infty} h = 462 \times 900 \times 236 = 98.1\text{kNm}.$$

That means that the connections have a ninefold reserve against decompression. If the compression gets loosed due to a general corrosion the shear force in the masonry walls would increase from

$$Q_{mas,max} = Q_{AG,max} \frac{(AG)_{mas}}{(AG)^*} = 280 \text{ kN}$$

to

$$Q_{AG,max} = 495\text{kN} \qquad \text{(see Table 2.)},$$

that would be crucial for the walls. As a further consequence, the maximum bending moment in the concrete bending walls would increase from 5076 kNm to 6920 kNm, the effect of which would claim a separate check.

It is worth mentioning also the increase of the displacement on the top from 9 to 17 mm, i.e. by 100 %, which lies beyond the allowable limit (1/1000).

References

1. ROSMAN, R.: Statik und Dynamik der Scheibensysteme des Hochbaues. Springer Verlag, Berlin, 1968.

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