Calculation of internal efforts in combined multystoried frames taking into account changing settlement scheme

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Abstract. The article raises the topic of accounting for the history of erection and loading of prefabricated reinforced concrete frames of frame buildings, which are being built everywhere with the allowed violation of the technological sequence of mounting crossbars and slabs. Internal forces in the elements of the frame are determined taking into account the above factors, a comparison with the results of the classical calculation is made. The most vulnerable elements of frames, especially designed for a small temporary load, are identified. An example from modern construction is considered, when the design of the crossbars during the erection of a building was significantly different from the one, set in the usual design calculation, which led to significant changes in the magnitude of internal forces in the columns and bolts compared with the classical calculation.

1. Formulation of the problem
Early on, the author considered the issues of recording the sequence of mounting elements of multi-storey frames, but factors that differed in the internal forces of the frame elements were taken into account independently of each other [1-9]. Subsequently, a similar topic of accounting for the erection of the frame, but in a slightly different aspect, was considered by other authors [10-20].
Recall, briefly, the main points of the work [1]. Usually, when calculating a multi-storey frame, the designer works with the final design scheme, which has a design floor, and loads such a frame with permanent and temporary loads on all floors simultaneously. Meanwhile, the real settlement scheme with each floor is updated, which sometimes gives a significant discrepancy of internal efforts compared to the classical calculation. Consider the example of a two-story frame the value of the bending moments when loading the first floor bolt in two cases: a). The second-floor bolt is missing: at this stage, loads from the self-weight of the floor and, in some cases, from the weight of the bulky equipment installed by the crane (Figure 1a) act; b). The second floor bolt is mounted: at this stage the remaining project loads are applied (Figure 1b). Comparison of the two diagrams show that the calculated actual circuit under load pins of the ground floor there is much greater bending moments in the racks of the first floor and the middle section of pins that are not accounted for in the usual classic calculation. The second factor considered is the ubiquitous, practiced both earlier and now in the construction industry, a change in the sequence of work in the assembly of prefabricated reinforced concrete crossbars. Technological maps for the installation of crossbars provide for the bathroom welding of reinforcements of the crossbar and column immediately after installation in the design position. Actually, this only happens after the installation of slabs. The builders explain such a
violation of technology by the fact that "it is inconvenient to weld from the scaffolding". Due to this, the bolt takes up a significant part of the load, namely, the own weight of the overlap, leaning on the columns hinged. Naturally, in this case, the bending moment in the middle section of the crossbar is increasing, and the moment transmitted to them by the columns is reduced.

Figure 1. Bending moments in the frame when loading the crossbar of the 1st floor: a - in the absence of overlying girth rail; b - taking into account the girth rail of the 2nd floor.

2. Solution method
Consider the effect of modular buildings in a rack allows simultaneous consideration of the loading sequence floors and violations processing sequence mounting crossbars.

The calculation is feasible for a 2-span 4-storey industrial building according to the ИИ-20/70 series with a grid of 9x6m columns, with different ratios of the linear rigidity of the crossbars and racks. Characteristics of frame elements: columns in the first variant 0,4x0,4m, height 4.8m, $A_k = 0.16m^2$, $I_k = 2.133 \times 10^{-3} m^4$, in the second variant - 0,4x0,6m, height 6m, $A_k = 0.24m^2$, $I_k = 7.2 \times 10^{-3} m^4$, cross section girth rail $A_p = 0.3275m^2$, $I_p = 1.482 \times 10^{-2} m^4$. The proportions of the linear rigidity of the crossbars and racks will be: for the first variant – $i_p / i_k = 3.71$, for the second option - $i_p / i_k = 1.37$. Vertical loads are taken according to the table 3.2. On the crossbars, the total distributed load will be 82,2 kН/m, on the cover beam 62,4 kН/m.

For determine the size of the span of the crossbar with the hinged support, consider the junction of the bolt with the column for the structures of the series in question (Figure 2).

Figure 2. The calculation of the hinge position in relation to the geometric axes of columns in violation of the technological sequence of installation of crossbars.

The calculation is carried out in succession in several stages.
Stage 1: a one-story frame is mounted with the hinged support of crossbars (before welding the valve outlets) in Figure 3. The load on the crossbars is evenly distributed from the weight of crossbars and slabs 29.7 kN / m.

Stage 2: Welding of reinforcements of crossbars on supports. The joints of the frame of the 1st floor are rigid. On the crossbar we apply a load of 6.6 kN / m from the weight of the partitions. The second floor is still missing (Figure 4).

Stage 3: The columns of the second floor are installed. The second floor girder is mounted hinged. To the crossbars of the second floor, we apply the weight of the overlap (crossbars and slabs) 29.7 kN / m (Figure 5).

Stage 4: Welding of reinforcement of crossbars of the 2nd floor on supports. All nodes of the frame are rigid. On the second floor girder we apply a load from the weight of the partitions of 6.6 kN / m. The third floor is still missing (Figure 6).

Stage 5: The columns of the third floor are installed. The third floor girders are mounted hinged. To the crossbars of the third floor, we apply the weight of the overlap (crossbars and slabs) 29.7 kN / m.

Stage 6: Welding of reinforcement of crossbars of the 3rd floor on supports. All nodes of the frame are rigid. On the 3rd floor girder we apply the weight of partitions 6.6 kN / m. Fourth floor yet.

Stage 7: Columns of the fourth floor are installed. The fourth floor girder is mounted hinged. To the crossbars of the fourth floor, we apply the weight of the overlap (crossbars and slabs) 29.7 kN / m.

Stage 8 of loading: All joints of the four-story frame are rigid. At the same time we apply the rest of the constant and all the temporary load on all the floors. On the overlapping beams: 82.2-29.7-6.6 = 45.9 kN / m, on the coverings of the cover 62.4-29.7 = 32.7 kN / m.

Summarize the efforts obtained for all stages of loading.

For comparison, let us perform the calculation using the classical scheme. To do this, we will apply simultaneously all the constant and temporary loads to the four-storey frame with rigid nodes: 82.2 kN/m on the crossbars, 62.4 kN / m on the coverings. The calculation results for the real and classical schemes for ip / ix = 1.37 are summarized in Table 1.
3. Analysis of results of calculation

The analysis of the obtained results for both variants of calculation shows that simultaneous consideration of the loading sequence of floors and the violation of the technological sequence of mounting the crossbars gives a small decrease in internal forces in the middle columns of the first floor (up to 5%), which goes to the margin of safety of these frame elements; a slight increase in internal forces in the extreme columns of the first floor (up to 5% in the longitudinal forces, up to 14% in the bending moments), a significant reduction in the design forces (moments) in the columns of subsequent floors to 30%, which indicates an unjustified overstatement of the strength of the cross-column sections. From this it follows that for the last columns of multi-storey buildings constructed in violation of the technological sequence of the installation of crossbars, the latter fact is a boon that facilitates the work of these columns under load. For them, the effect, taking into account the loading sequence and considered in Fig. 1, is significantly reduced. For crossbars of all floors, taking into account the unification of their sections, the maximum reference moment (for the ratio $i_p/i_k = 1.37$) is actually 431.6 kN ∙ m, and according to the classical calculation 606.8 kN ∙ m. From this it follows that in the cross section of crossbars the planned plastic hinge is not formed, because the strength of these sections is overestimated by 40% (compared to the required strength for real bending moments).

At the same time, the average cross-section of crossbars is overloaded in this case by more than 40%, and the crossbeams - by 46%. It should be noted that the greatest effect is created in the frame elements of buildings working under a small time load: low-loaded industrial buildings and especially in residential and public buildings with a frame frame. For the ratio $i_p/i_k = 3.71$, the dependences obtained for bolts remain, but the effect is somewhat lower: an increase in the maximum bending moments in the crossings of bolts in this case is 31-32%, a similar decrease in the reference moments.

### Table 1. Comparison of calculation results for real and classical schemes a 4-storey frame.

| Frame element | Effort | Classic calculation | Results taking into account loading history of schemes 1-8 | Deviation in % (more than 5%) |
|---------------|--------|---------------------|----------------------------------------------------------|-------------------------------|
| Extreme       | Q      | -54.36              | -51.0                                                    | -6.2                          |
| column 1 floor| $M_{lower}$ | 87.81             | 81.7                                                    | -7                            |
|               | $M_{top}$  | -173.1             | -162.6                                                  | -6                            |
| Extreme       | Q      | -99.39              | -74.85                                                  | -24.7                         |
| column 2 floors| $M_{lower}$ | 243.39             | 162.1                                                   | -33.4                         |
|               | $M_{top}$  | -233.68             | -167.2                                                  | -15.6                         |
| Extreme       | Q      | -94.66              | -73.0                                                   | -22.9                         |
| column 3 floors| $M_{lower}$ | 227.45             | 154.3                                                   | -32.2                         |
|               | $M_{top}$  | -226.94             | -196.2                                                  | -13.5                         |
| Extreme       | Q      | -111.63             | -75.0                                                   | -32.8                         |
| column 4 floors| $M_{lower}$ | 251.3              | 157.2                                                   | -37.4                         |
|               | $M_{top}$  | -284.54             | -202.7                                                  | -28.8                         |
| Rigel 1 floor | $M_{left}$ | -416.5             | -324.7                                                  | -22.0                         |
|               | $M_{right}$ | -606.78            | -430.9                                                  | -29.0                         |
|               | $M_{average (max)}$ | 320.6            | 454.5                                                   | +41.8                         |
| Rigel 2 floor | $M_{left}$ | -461.14             | -351.5                                                  | -23.8                         |
|               | $M_{right}$ | -571.97            | -431.6                                                  | -24.5                         |
|               | $M_{average (max)}$ | 315.7            | 452.1                                                   | +43.2                         |
| Rigel 3 floor | $M_{left}$ | -478.23             | -353.4                                                  | -26.1                         |
|               | $M_{right}$ | -555.4             | -405.4                                                  | -27.0                         |
|               | $M_{average (max)}$ | 315.46            | 452.9                                                   | +43.6                         |
| Rigel 4 floor | $M_{left}$ | -284.54             | -202.7                                                  | -28.8                         |
|               | $M_{right}$ | -448.1             | -283.8                                                  | -36.7                         |
|               | $M_{average (max)}$ | 265.5             | 388.6                                                   | +46.4                         |
Consider an example of modern construction. We will estimate the strained-deformed state of the crossbars of the shopping and entertainment complex "ALIMPIC", built in Astrakhan.

Characteristics of the building: 3-storey building with prefabricated frame, height of floors - 5.6 m, grid of columns 9x9m, with transverse carriers and longitudinal curtains. Columns cross-section 40x40 cm, at the level of junction of crossbar through (there is only longitudinal reinforcement), concrete class B30. Rigel bearing length of 8.5 m cross section 40x60 cm, concrete B30, has releases of transverse reinforcement to a height of 210 mm. After installing the bolt in the design position and laying the slabs having a length of 8.7 m and a support depth of only 5 cm, an additional reinforcement and concreting of the interlacing space above the bolt is performed. Thus, the height of the girth rail in the design position reaches 82cm. In the supporting part (30 cm from each side), the girth rail has a trough-like shape, with the outlets of the reinforcement. The middle (solid) part of the girth rail is prestressed. Plates of overlap pre-tensioned with a height of 220 mm, with oval voids. Longitudinal crossbars have the dimensions of section 40x40 cm before the etching of the interlite space and 62 cm in the design state. The support of the crossbars before the joint is made of concrete placed on the collars of steel corners attached to the column. The depth of support of the longitudinal crossbars is also 5cm. Concreting of the joint of longitudinal and transverse bolts with the column should provide a rigid support unit.

According to the recommendations for the construction of the frame, temporary supports must be installed along the entire length of the dead bolt, which are retained even after the strength of the concrete is fixed in the concrete of the joint between the crossbar and the column. The real situation is presented in the photograph (Figures 7).

As can be seen from Figure 7, on the lower floor under the crossbars there are supports only on the left, about a quarter of the span. The fittings at the joints are not yet welded, the joints operate articulately. According to the design of the work, welding and embedment of joints is carried out after laying the slabs.

![Figure 7. The real situation with the arrangement of intermediate supports under the carrying beams.](image)

We will calculate the similar 3-storeyed frame for two variants of loads.

In the first variant (real), two calculation schemes work in sequence: firstly, loads acting on the weight of the overlap (including the own weight of the crossbar) act on the hinged bolt, \( q_1 = 36.3 \text{ kN/m} \), then, after the units are grounded, on the frame with rigid knots we apply the rest part of the constant and time load \( q_2 = 64 \text{ kN/m} \). Calculation of loads is performed according to Table. 2.

We will limit ourselves to an example of the calculation of a 3-span frame. From the action of \( q_1 \) with the hinged support, the bending moment in the middle of the girth rail is:

\[
M_{\text{max}} = \frac{q_1 l^2}{8} = \frac{36.3 \cdot 8.5^2}{8} = 327.8 \text{kNm}.
\]

Internal forces in the girth rail of the first floor with rigid support from the action of the load \( q_2 \):
bending moments \( M_{\text{left}} = -251.77 \text{ kNm}, \ M_{\text{right}} = -485.25 \text{kNm} \); transverse forces \( Q_{\text{left}} = 262.06 \text{kN}, \ Q_{\text{right}} = -313.94 \text{kN} \); maximum bending moment in the span girth rail \( M_{\text{max}} = 284.74 \text{kNm} \), moment in the middle of the girth rail \( M_{\text{average}} = 279.5 \text{kNm} \). The result for the first variant: \( M_{\text{average}} = 607.3 \text{kNm} \).
In the second variant of loading, we apply the total load \( q = q_1 + q_2 = 36.3 + 64 = 100.3 \text{ kN/m} \) to the frame with rigid knots. Calculation results for the first floor bolt:

- Bending moments \( M_{\text{left}} = -394.57 \text{ kNm}, M_{\text{right}} = -760.48 \text{ kNm} \);
- Transverse forces \( Q_{\text{left}} = 410.7 \text{ kN}, Q_{\text{right}} = 492.0 \text{ kN} \);
- Maximum bending moment in the span \( M_{\text{max}} = 446.2 \text{ kNm} \).

For the middle crossbar of the first floor, the maximum bending moments in the span of the girth rail will be respectively 526.5 kNm on the first and 311.4 kNm for the second option.

Table 2. Calculation of loads on the overlap in "ALIMPIK".

| No.  | Load                                            | Normative, kN/m² | \( \gamma_i \) | Calculated, kN/m² |
|------|-------------------------------------------------|------------------|----------------|------------------|
| 1    | Hollow-core reinforced concrete slab            | 3                | 1.1            | 3.3              |
| 2    | Reinforced concrete girth rail overlapping      | 0.67             | 1.1            | 0.73             |
|      | Total:                                          |                  |                | 4.03             |
| 3    | Weight of partitions                            | 1                | 1.1            | 1.1              |
| 4    | Weight of floors                                | 1                | 1.2            | 1.2              |
| 5    | Temporary for trading rooms                     | 4                | 1.2            | 4.8              |
|      | Total:                                          |                  |                | 7.1              |

\( q_1 = 4.03 \times 9 = 36.3 \text{ kN/m} \)

\( q_2 = 7.1 \times 9 = 64 \text{ kN/m} \)

4. Conclusions

- Drawing up of calculation schemes without taking into account the sequence of installation leads to a distortion of the real picture of the distribution of forces and displacements in the elements of the skeletons.
- The calculation of the sequence of installation of structures allows you to obtain a true distribution of internal forces in the frames. Under real loading, most of the constant load, and in some cases, part of the temporary load, does not affect the forces in the structures of the upper floors. The greatest effect occurs when the constant load over the time is exceeded, i.e. in the frames of civil buildings and industrial, designed for a small payload.
- With increasing flexibility of crossbars, the effect of taking into account the sequence of erection of the frame increases.
- The calculation of the hinged support of the girth rail to the bathroom welding of reinforcement bars and column increases the maximum bending moment by 37% in the span of the dead bolt and by 69% in the span of the middle bolt. At the same time, the bearing moments of the girth rail decrease, and, as a consequence, they transmit the bending moment to the columns (up to 36%), with the greatest effect occurring on the extreme columns of the frame.

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