Study of the distribution of wind loads between stiffening elements and framing of multi-storey buildings

Autor(en): **Kositsyn, B.A.**

Objekttyp: **Article**

Zeitschrift: **IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht**

Band (Jahr): **8 (1968)**

PDF erstellt am: **09.08.2024**

Persistenter Link: https://doi.org/10.5169/seals-8851

Study of the Distribution of Wind Loads Between Stiffening Elements and Framing of Multi-Storey Buildings

Etude de la distribution des charges du vent entre des éléments de la rigidité et des portiques des carcasses des immeubles géants

Untersuchung über die Verteilung der Windlasten zwischen Versteifungen und Stockwerkrahmen von Hochhäusern

B.A. KOSITSYN
Cand. Techn. Sc.
(USSR)

Modern multi-storey buildings consist, as a rule, of framing, vertical diametric walls, adding transverse rigidity to the building, floors and light railing structures attached to the framework. In order to design such buildings for horizontal loads it is necessary, in general, to determine the forces arising in the structures of the framework and its deformations. The latter is required to calculate the strength of railing structures, their connections with each other and with the framework. (When designing the framework, the influence of railing structures is, commonly, not taken into account).

The present paper gives some results of the investigation of the first, most difficult part of the problem of designing framed structures with due account of the spatial behaviour of the structures.

Horizontal loads in framed buildings are taken up by vertical diametric walls and frames. Floors joined in rigid disks, serve as bracing distributing the load between the load-bearing members of the building and equalizing their transverse displacements. In very high buildings with diametric walls closely arranged, disks of floors also exert a direct affect on longitudinal deformations of load-bearing members due to the resistance of floors to torsion at unequal angles of rotation of cross sections of diametric walls and frames.

When diametric walls are placed symmetrically in the plan, the three dimensional problem is reduced to a two dimensional one, viz. bending of each load-bearing member from an appropriate portion of the horizontal load, the size of which is to be determined; with asymmetric location the problem is divided in two: bending and torsion, the latter being set up by torsional moments distributed along the height of the building arising from non-coincidence of the resultants of horizontal loads with the line connecting the centres of torsion of horizontal sections of buildings.

Consider briefly the torsion.

Rigid horizontal disks formed by floors do not permit the framework to deform at torsion otherwise than by rotations of all sections round the vertical axis, and so V. Z. Vlasov's theory of thin-wall systems with closed undeformed contour of the cross section may be used for design. In fact, longitudinal forces arising in the members of the framework due to torsion may be neglected too, and making use of Bredt's postulate, shear forces in the planes of frames and diametric walls on the level of each floor can be determined on the basis of the diagram of torsional moments, which is easily drawn. These forces will give rise to local bending of posts (and rafters) of frames, and in continuous vertical diametric walls shear stresses will occur due to their effects.

When designing for bending, vertical diametric walls and frames of the framework, for simplification, may be grouped into two independent blocks with summed up parameters of stiffness, viz. flexural parameters (EJ_1 and EJ_2), characterizing the resistance of elements of blocks to sectional rotations, and shear parameters (GF_1 and GF_2) showing their resistance to shear-distortions / 2 / .

The present problem comes to determining forces $q(x)$ and $m(x)$ arising at the contact between two design blocks.

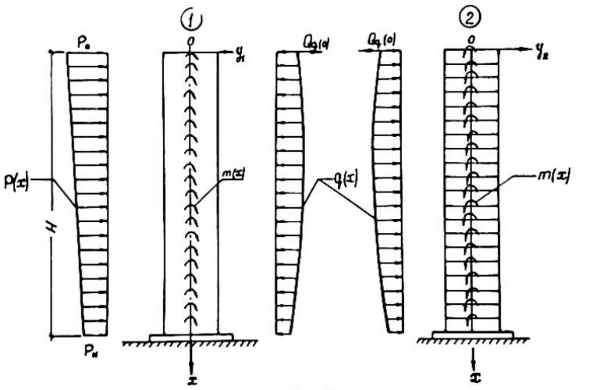

Рис. 1
(Fig 1)

Fig. I illustrates the design scheme corresponding to the three-dimensional behaviour of structures described above with a single foundation slab under the building.

If in the design scheme the block with index 2 is the framing, then $q(x)$ is the fraction of the horizontal load resisted by the framing. Concurrently with this load there occur generally distributed bending moments $m(x)$ due to the difference in the angles of rotation of sections and resistance shown by the floors to these displacements.

Assuming the beginning of the coordinates on the upper end of the building where, as it is clear beforehand, there are no concentrated imposed bending moments, interconnected deformations of the blocks may be described by the following system of equations:

$$\left\{\begin{aligned} & EJ_1 y_1'' - \frac{EJ_1}{GF_1} M_q'' + M_q + \frac{EJ_1}{GF_1} M_m'' - M_m = \frac{EJ_1}{GF_1} |M_p|'' - |M_p|, \\ & EJ_2 y_2'' + \frac{EJ_2}{GF_2} M_q'' - M_q - \frac{EJ_2}{GF_2} M_m'' + M_m = 0, \\ & (y_1 - y_2) C = M_q'', \\ & -\frac{M_m''}{\varepsilon} = \frac{1}{EJ_2}\left(M_q - M_m\right) - \frac{1}{EJ_1}\left(-|M_p| - M_q + M_m\right) \end{aligned}\right. \qquad \text{(I)}$$

In Equations (1):

M_q, M_m, M_p are the bending moments from unknown contact forces $q(x)$ and $m(x)$ and outer load $p(x)$ respectively;

$C\,[kg/cm^2]$ is the rigidity of floors measured by the value of the linear force on contact $\bar{q}\,[kg/cm]$ causing reciprical transverse displacement of the blocks by a linear unit $[cm]$ in the direction of the force;

$\varepsilon\,[kg\,cm/cm]$ is the rigidity of floors thought of as longitudinal bracing measured by the value of a linear torsional moment $\bar{m}\,[kg\,cm/cm]$ causing torsion of the floors by an angle equal to unity.

It should be noted that we find $M_q(x)$ and $M_m(x)$, but not directly $q(x)$ and $m(x)$. The loads are easily determined by the formulae:

$$q(x) = M_q''(x),$$
$$m(x) = M_m'(x) \tag{2}$$

Solution of the system of equations (1) gives the following resolution differential equations:

$$M_m^{VI} - \left[\varepsilon\left(\frac{1}{EJ_1} + \frac{1}{EJ_2}\right) + C\left(\frac{1}{GF_1} + \frac{1}{GF_2}\right)\right] M_m^{IV} + C\left(\frac{1}{EJ_1} + \frac{1}{EJ_2}\right) M_m'' =$$
$$= -\frac{\varepsilon}{EJ_1}|M_p|^{IV} + C\varepsilon\left(\frac{1}{EJ_1 \cdot GF_2} - \frac{1}{EJ_2 \cdot GF_1}\right)|M_p|''; \tag{3}$$

$$M_q = -\frac{M_m''}{\varepsilon\left(\frac{1}{EJ_1} + \frac{1}{EJ_2}\right)} + M_m - \frac{EJ_2}{\sum EJ}|M_p|$$

Boundary conditions added to Equations (3) to find the constants of integration are:

$$x=0 \qquad M_m=0, \quad M_q=0, \quad M_q'=0;$$

$$x=H \qquad M_m'=0, \quad M_q'=\frac{GF_2}{\sum GF}|M_P|', \quad M_q''=0.$$

The sign convention is as follows: the bending moment rotating the blocks counter clockwise and shear force of the same direction are thought of as positive. A negative sign of $M_q(x)$ and $M_m(x)$ will indicate that the direction of $q(x)$ and $m(x)$ on the scheme was assumed improperly.

When designing buildings for wind load, the latter is as a rule defined by expression $P(x)=P_0-\alpha x$, where $\alpha=\frac{P_0-P_H}{H}$ (the diagram is of a trapezium form). Floors are often taken as absolutely rigid in the plane of discs ($c=\infty$) and flexible out-of-plane ($\mathcal{E}=0$). Then $m(x)=0$, the appropriate fraction of wind load resisted by the framing and forces may be determined by the formulae:

$$M_q(x)=A\left\{\frac{B}{K}Sh\,Kx-\frac{1}{K^2}\left[(Ch\,Kx-1)+\alpha x\right]\right\}+\frac{EJ_2}{\sum EJ}\left(\frac{P_0x^2}{2}-\frac{\alpha x^3}{6}\right);$$

$$Q_q(x)=A\left[B\,Ch\,Kx-\frac{1}{K}\left(P_0\,Sh\,Kx+\frac{\alpha}{K}\right)\right]+\frac{EJ_2}{\sum EJ}\left(P_0x-\frac{\alpha x^2}{2}\right);$$

$$q(x)=A\left(KB\,Sh\,Kx-P_0\,Ch\,Kx\right)+\frac{EJ_2}{\sum EJ}\left(P_0-\alpha x\right);$$

$$A=\frac{EJ_2}{\sum EJ}-\frac{GF_2}{\sum GF}; \tag{4}$$

$$B = \frac{1}{Ch\kappa H}\left[\left(\frac{Sh\kappa H}{K} - H\right)P_0 + \left(\frac{H^2}{2} + \frac{1}{K^2}\right)\alpha\right];$$

$$K^2 = \frac{\dfrac{1}{EJ_1} + \dfrac{1}{EJ_2}}{\dfrac{1}{GF_1} + \dfrac{1}{GF_2}} .$$

The equation for the axis of a bent building is:

$$(\Sigma EJ)y = \left(\frac{EJ_2}{GF_2} - \frac{EJ_1}{GF_1}\right)\left[M_q(H) - M_q(x)\right] + \frac{EJ_1}{GF_1}\left[|M_p(H)| - |M_p(x)|\right] - x\left(\frac{P_0H^3}{6} - \frac{\alpha H^4}{24}\right) + \frac{P_0x^4}{24} - \frac{\alpha x^5}{120} + \frac{P_0H^4}{8} - \frac{\alpha H^5}{30}. \qquad (5)$$

The limit values of angles of distortion of railing structures and inner partition walls which characterize their strength and deformability, provided they are fastened rigidly to the cross frames of the framework, must not be less than the values obtained by the formulae:

$$\gamma = m_\gamma \frac{Q_q(max)}{GF_2}; \qquad (6)$$

where m_γ is the safety factor stipulated by the Building Code of a particular country. The zone of the largest distortions, where $Q_q(x)$ reaches its maximum is located at one-third of the building height counting from the ground base.

To study the nature of the distribution of horizontal loads between the elements of the framework, Central Research Institute for Building Structures of Gosstroy of the USSR has carried out experiments on a large model of 30-storey buildings of an original design which were then built in a slightly changed version in Moscow / 3 /. The buildings under consideration have a broken configuration in the plan (30° angle), they consist of a rigid core, two additional vertical diametric walls on the ends of the building and framework which is enclosed between the diametric walls and the core and which bears almost all the useful load. The framework rests upon a solid foundation slab reinforced in the plane of posts of

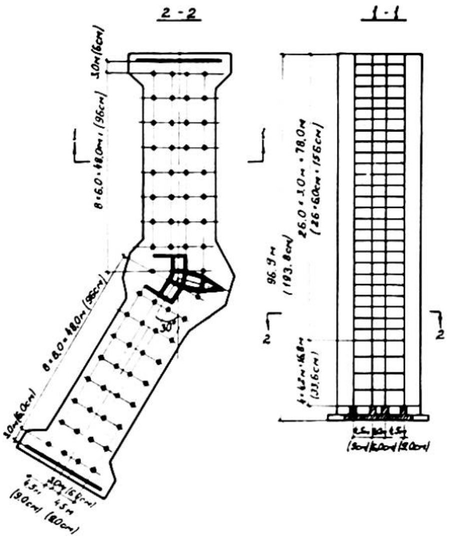

Рис 2
(Fig. 2)

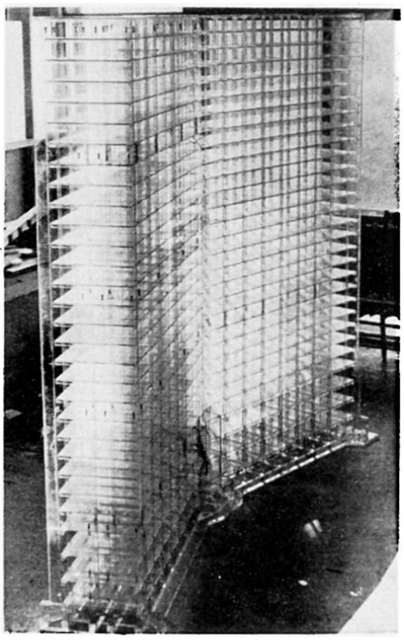

Fig. 3

frames by longitudinal and transverse ribs. Fig. 2 shows the plan and cross-section of the building, figures in brackets relate to the model.

The model was made of organic glass following the similarity conditions with the scale 1/50 (Fig. 3). The vertical load on floors was produced by small shots, and the horizontal load, the diagram of which is shown in Fig. 4, was set up through a distribution arrangement by loads hung on elastic threads pulled over pulleys.

Longitudinal deformations of the material were measured by wire strain gauges and displacements of the model were measured by indicators of 0.01 to 0.001 mm sensitivity.

The measurements have shown that transverse displacements of the model frames, core and diametric walls were practically the same. This testifies to great rigidity of discs formed by the floors despite a significant space between the diametric walls and core.

The influence of torque rigidity of the floors on longitudinal deformations of posts of frames and diametric walls has appeared to be also negligible as it should have been expected.

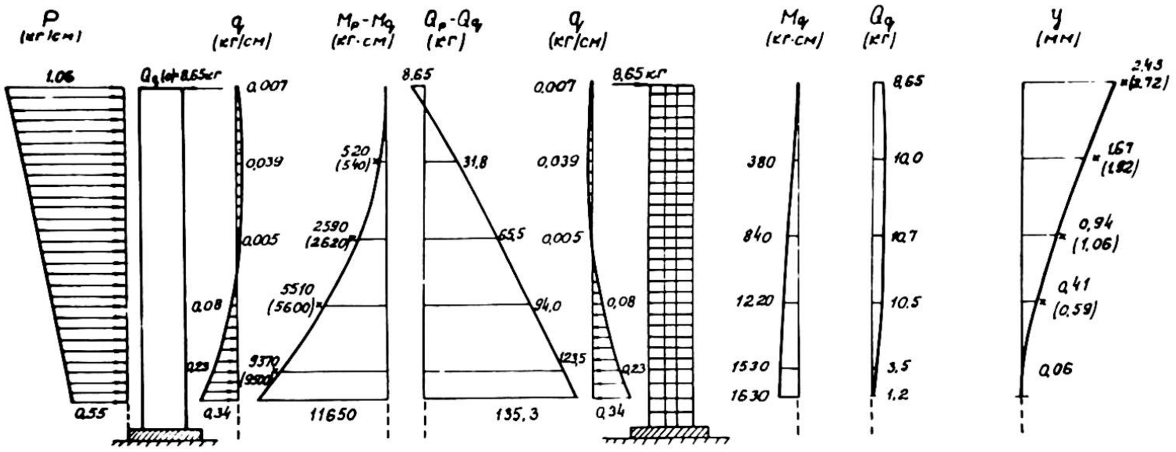

Рис. 4
(Fig 4)

The results of the design of the model under the supposition of embedment of the framework in the plane of the floor over the second floor by formulae (4), the diagrams of loads acting on framing and core, of bending moments and shear forces are given in Fig. 4. On the diagram of bending moments small crosses show the data obtained from the experiment by gauges attached to the core of the building and processed appropriately.

The data of these gauges were most stable, as well as of the indicators measuring lateral displacements; the results of measurements are also illustrated in Fig. 4 in the same way.

BIBLIOGRAPHY

1. V. Z. Vlasov, "Thin-Wall Three-Dimensional Systems", Moscow, Gosstroyisdat, 1958.

2. Transactions of Central Research Institute for Building Structures of Gosstroy of the USSR "Static Design of Large-Panelled Buildings" edited by B. A. Kositsyn, Moscow, Gosstroyisdat, 1963.

3. B. P. Voljvson, I. K. Shevchenko, D. M. Beniaminov "Model Test of a Building Being Built in Kalinin Prospect in Moscow", Moscow, Gosstroyisdat, J. Stroitelnaia Mekhanika i Raschiot Sooruzhenii, No. 5 (63), 1967.

SUMMARY

The theoretical investigations and model test results enable to conclude the following. When designing multi-storey buildings it should be born in mind that wind loads are distributed between the stiffening elements and the frames of the framework and overload the latter. The degree of the distribution of wind loads depends on the rigid characteristics of the framing and diametric walls and may be determined with a sufficient degree of precision by the approximate design method discussed in the paper.

RÉSUMÉ

Lors des calculs des immeubles géants avec des carcasses et des panneaux il faut prendre en considération les charges du vent distribuées entre des éléments de la rigidité et des portiques des carcasses ce que fait venir la sur charge les derniers. Le degré de la distribution des charges du vent dépend du rapport des caractéristiques de la rigidité des éléments de la carcasse et ce degré est déterminé avec la precision parfaite à l'aide de la méthode du calcul exposé au rapport.

ZUSAMMENFASSUNG

Man kann aus den theoretischen Untersuchungen und Versuchsergebnissen folgende Schlussfolgerungen ziehen: Bei der Berechnung der Fachwerkgebäude mit Bauplatten ist zu berücksichtigen, dass die Windbelastung eine Überbelastung des Fachwerkrahmens hervorruft und zwischen den Elementen der Steifen und des Fachwerkrahmens verteilt ist. Der Verteilungsgrad der Windbelastung hängt von dem Verhältnis der Steifigkeit der Fachwerkelemente ab und kann laut Bericht nach der Methode der Näherungsrechnung ziemlich genau bestimmt werden.

Leere Seite
Blank page
Page vide