PROGRESSIVE COLLAPSE ANALYSIS OF LARGE PANEL BUILDINGS

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Summary

Paper deals with the dynamic elasto-plastic response of large panel structures when they are locally damaged by a high intensity pressure or impact loading such as gas explosion or vehicle collision. The analysis is based on a model, which consists of an assembly of rectangular rigid plates interconnected along their edges by elasto-plastic springs acting in tension, compression and shear.

A detailed parametric study based on a cantilever mechanism is presented. The results show the influence of the different parameters on the response of large panel structures under abnormal loading conditions.

Basic concepts

When a large panel (LP) building is locally damaged by a high intensity pressure or impact load one or more of the panels can be perfectly destroyed and this accident might cause the progressive collapse of the entire structure or a large part of it. The problem is to determine the static and dynamic response of the damaged building. Since the in situ joints are generally the weakest parts of a prefabricated structure in the present analysis it is assumed, that merely the joints undergo deformations. Thus, in the proposed model of the LP structure the panels are considered as perfectly rigid elements, while the joints are replaced by springs acting in tension, compression and shear (Fig. 1) [1]. The springs are assumed linear elastic-perfectly plastic with a limited deformation capacity (Fig. 2). For the strength and deformation characteristics in tension and in compression different values can be taken into account.

Quasi-static analysis

The goal of the quasi-static analysis is to determine the load carrying capacity of the damaged structure under quasi-static conditions. This is a simple problem of limit analysis which can be solved by mathematical programming [2]. When, however, the joints have limited deformation capacity a step-by-step load history analysis has to be conducted and after each step the deformations of the joints have to be compared with their capacities.

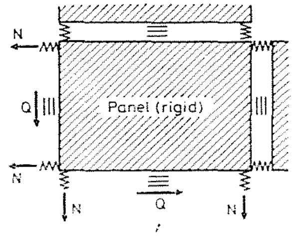

Fig. 1. Model of the structure

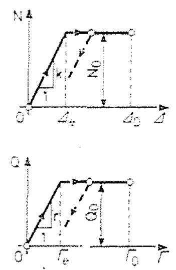

Fig. 2. Model of the joints

A parametric study has shown that the tensional joints along the vertical edges of the panels have the most significant influence on the load carrying capacity of the damaged structure [3].

Dynamic analysis

When in a wall of a LP structure one of the panels looses its load carrying capacity the vertical reaction force of this panel suddenly diminishes to zero (Figs 3 and 4). For the analysis of this dynamic effect the kinematical approximation can be used [4]. The idea of this approach is to impose a stationary, kinematically admissible, displacement field (e.g. the yield mechanism of the quasi-static solution) on the structure. In this manner a one-degree-of-freedom elasto-plastic system is obtained, the dynamic response of which can be followed by a step-by-step procedure. Next, this will be illustrated by a simple example.

Solution based on a cantilever mechanism

Figure 5 shows the cantilever mechanism of a LP wall where n panels above the damaged panel rotate as a rigid body by angle Φ about point 0. (The elastic, quasi-static analysis of this mechanism has been published else-

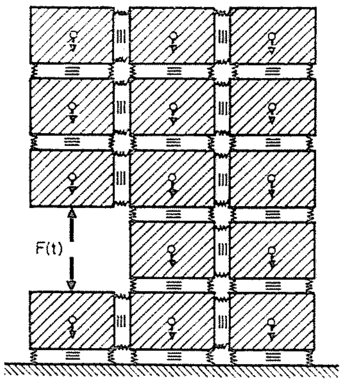

Fig. 3. Model of the dynamic analysis

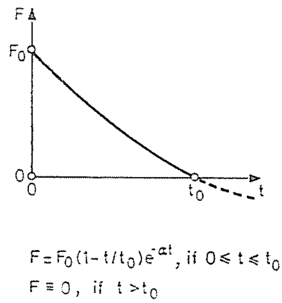

Fig. 4. The dynamic reaction force

where [5].) Considering Fig. 2. the forces of the horizontal springs (reinforcements) can be expressed as follows. a) In case of loading ($\varDelta_i \, d\varDelta_i > 0$):

$$N_i = \delta_i^e k_i + \delta_i^p N_{0i}, \text{ where } \begin{cases} \delta_i^e = \begin{cases} 1, \text{ if } \varDelta_i \leq \varDelta_{ei} \\ 0, \text{ if } \varDelta_i > \varDelta_{ei} \end{cases} \\ \delta_i^p = \begin{cases} 1, \text{ if } \varDelta_{ei} \leq \varDelta_i \leq \varDelta_{0i} \\ 0, \text{ if } \varDelta_i > \varDelta_{0i} \end{cases} \end{cases}$$

b) In case of unloading ($\varDelta_i d\varDelta_i \leq 0$):

$$dN_i = k_i \, d\varDelta_i .$$

Here $\varDelta_i = y_i \Phi$ and dN_i and $d\varDelta_i$ are the force and strain increments of the springs. The differential equation of motion has the form

$$\frac{G}{3g}(a^2 + H^2)\ddot{\Phi} + [\sum_i \delta_i^e k_i y_i^2]\Phi = \sum_i \delta_i^p N_{0i} y_i - \frac{a}{2}[G - F(t)].$$

This equation can be solved by the Wilson — Θ numerical integration method. In our parametric study the fixed values $a = 5.4$ m, $h = 2.8$ m, $H = n \times$

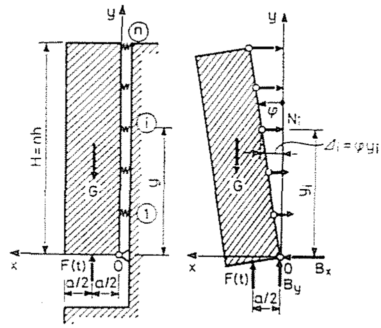

Fig. 5. The cantilever mechanism

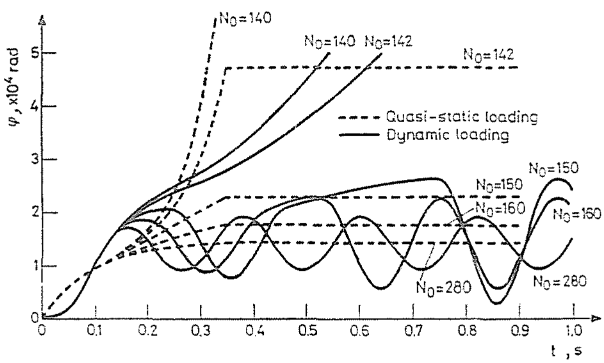

Fig. 6. Quasi-static and dynamic response of the structure

$\times$ 2.8 m, $\Delta_{0i} = 0.02$ m, $F_0 = G = n \times 800$ kN, $m = n \times 80\,000$ kg have been assumed, while the other data have been considered variable parameters.

Case 1.: $N_{0i} = N_0$ is a variable parameter, while the other data are: n = 10, $k_i = 5 \cdot 10^4$ kNm^{-1}, $F_0 = G = n \times 800$ kN and

$$F(t) = 800 \text{ n } (1 - t/t_0)e^{-7t}, \text{ if } 0 \leq t \leq 0.35 \text{ sec}$$

$$F(t) = 0, \text{ if } t > 0.35 \text{ sec}$$

Figure 6 shows the quasi-static and dynamic response of the structure at different values of N_0. The minimum value of N_0 at which the load carrying capacity of the damaged structure is large enough to support its weight under quasi-static conditions is $(N_0^s)_{\min} = 140.26$ kN. At lower values of N_0 Φ increases without limits, while at higher values after reaching a certain rotation the cantilever ceases to move. Under dynamic conditions the minimum value of N_0, at which Φ does not increase indefinitely, is $(N_0^d)^{\min} = 149$ kN. When $N_0 > (N_0^d)^{\min}$ the cantilever undertakes elastoplastic or/and elastic vibrations. When N_0 is larger than 200.4 kN and 271.3 kN, the response of the structure

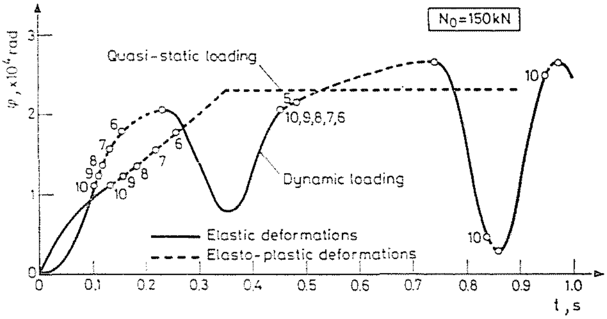

Fig. 7. Plastification of the joints

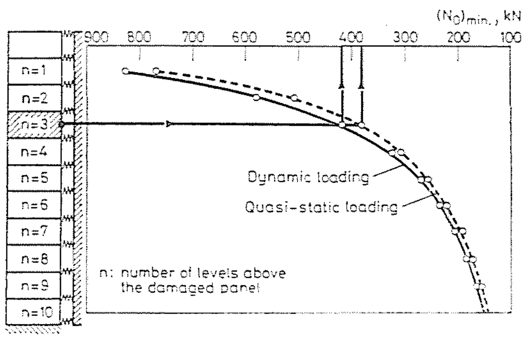

Fig. 8. $(N_0)_{min}$ in terms of the level of the damaged panel

is purely elastic under quasi-static and dynamic loading, respectively. Finally, Fig. 7 shows the plastification of the joints when $N_0 = 150$ kN. Δ_i nowhere exceeded the strain capacity of the joints which was $\Delta_0 = 0.02$ m.

Case 2.: Now t_0 is the variable parameter and has the values 0.1; 0.2; 0.3; 0.4 and 0.5 sec. All the other data are identical with those of case 1.

Evidently the quasi-static results are the same as in case 1. According to the dynamic analysis, the smaller is t_0 the stronger joints are needed to support the loads. When e.g. t_0 is 0.3 sec and 0.1 sec, then $(N_0^d)_{min}$ is 160 kN and 180 kN, respectively.

Case 3.: When $k_i = 2.5 \cdot 10^4$ kNm^{-1} and all the other parameters have the values of case 1, then $(N_0^S)_{min}$ remains unchanged (140.26 kN), the dynamic response of the structure is, however, different and $(N_0^d)_{min}$ has a higher value (159.5 kN).

Case 4.: Let us assume, that the level of the damaged panel is variable i.e. the number of the levels above the damaged panel varies from $n = 1$ to $n = 10$. Then, using the same data as in case 1, the minimum values of N_0 which are needed to support the structure under quasi-static and dynamic conditions are plotted in Fig. 8. It is to be seen that the higher is the level of the damaged panel the higher is the necessary minimum value of N_0. Thus, on higher levels stronger horizontal joints (reinforcements) are needed and they are more efficient.

Remarks

The investigation of other one- and multi-degree-of-freedom yield mechanisms and the elasto-plastic static and dynamic analysis of space problems are in progress.

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