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## COMPOSITE MULTIHOLLOW SLABS FOR REINFORCED CONCRETE MULTISTORY BUILDINGS

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### 1. Introduction

Frameless reinforced concrete residential buildings and offices from 5 to 16 stories can be constructed with cast-in-situ walls and girderless floors. Thickness of interior plane walls and floors of load-carrying frame type multistory panel systems are about 200 mm. In many cases it is rational to use thin precast reinforced concrete slabs as residual moulds and ceiling members for casting floors. Such slabs provide some technological advantages in concrete works. It is reasonable also to form hollows in cast-in-situ floor concrete and at the same to make them lighter and suitable for engineering communication installations inside a cast-in-situ structure [1].

Cast-in-situ concrete ensures the rigidity of frame type panel system joints and the possibility to transmit all action effects of floor and wall members to the system foundations. Therefore, a low-depth floor structure must be sufficiently strong and stiff under action effects caused by gravity and horizontal forces. Of special interest is the failure behaviour and adhesiveness between lower precast and upper cast-in-situ slab concrete of composite floor structure.

The main points of investigations presented below are:

- to research the unity of stay-in-place precast slabs and cast-in-situ concrete of flexural floor structures and measures ensuring their adherence in ultimate limit state;
- to assess failure performance of composite multihollow slabs as floor flexural members of multi-storey buildings exposed to gravity and lateral loading.

The deflection of composite multihollow slabs in the case of large spans was not under consideration, as the main purpose of research on the composite multihollow slabs was an analysis of cracking and failure performances and the strength behaviour of structures in wall-slab connection on the faces of joint core zone.

### 2. Test specimens

Seven composite multihollow specimens designed and casted by author [2] allowed to investigate the behaviour of slabs at their support zones under low-cyclic quasi-static loading (Fig 1). The cantilever floor structural fragments were of the natural cross-section size. Composite slab depth was of 200 mm (depth of precast slab in composite flooring was 60 mm), length of cantilever part of slab was 1100 mm (loading arm of 1000 mm), support length of 200 mm (the same as the wall thickness in panel-type buildings) and additional length to ensure reinforcement adhesion at the support of 200 mm. Total composite slab length was 1500 mm. The hollows in the in-situ concrete part of composite slabs were formed by  $\varnothing 76$  mm plastic pipes.

High-early strength cement and coarse aggregate were used for concrete mixes. The compression strength of the concrete for precast and in-situ parts of composite slabs, respectively, was  $f_{cp} = 30$  MPa and  $f_c = 24$  MPa. The yield strength of main longitudinal  $\varnothing 16$  mm bars was  $f_y = 380$  MPa. Joint stirrups, shear and precast slab reinforcement were made from  $\varnothing 6$  mm bars with yield strength  $f_y = 385$  MPa. Welded meshes made from  $\varnothing 3$  mm wires were used avoiding shrinkage cracks in the top of in situ-slabs.

Precast slabs were cast beforehand and were of four types: the plane concrete surface (S1 and S2), the concrete shear cotter (S3), the joint stirrups in the long (passed through the support part S4) and short (S5, S6 and S7) precast slabs. Topping concrete layer was casted on the precast slabs as the bottom moulds after the reinforcement and plastic pipes for the hollow arrangement were set.

Slab specimen S4 with long precast slab was used to clarify the role of the precast slab reinforcement on

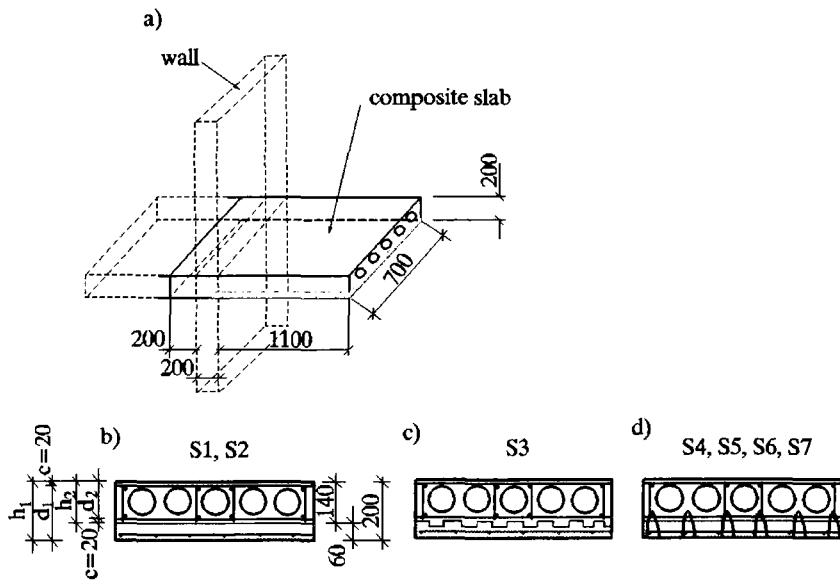


Fig 1. Outline (a) and cross-sections (b, c, d) of test composite multihollow slabs

the behaviour and ultimate bearing capacity of the flexural member in case of its anchorage in the joint zone.

With the aim of making acquaintance with the composite slab cracking and failure performance and the influence on the stress-strain behaviour in wall-slab connection joint core area, composite slab support and loading conditions were identical to those in actual structural members.

Special equipment with an actuator (Fig 2) was used by author for loading specimens by twelve-cycle

and increasing in course of test forces. One slab end was connected with an actuator and another one was fastened between two box-shape steel supports. The precast slab support length was 30 mm (only for the specimen S-4 precast slab passed through the steel supports and its length was 200 mm). Specimens were subjected to one direction monotonous backward loading (tensile stress generated in the upper part of cast-in-situ slab) or reversal cyclic loading.

The loading of specimens was provided by the displacement control of test slab rotations (Fig 3).

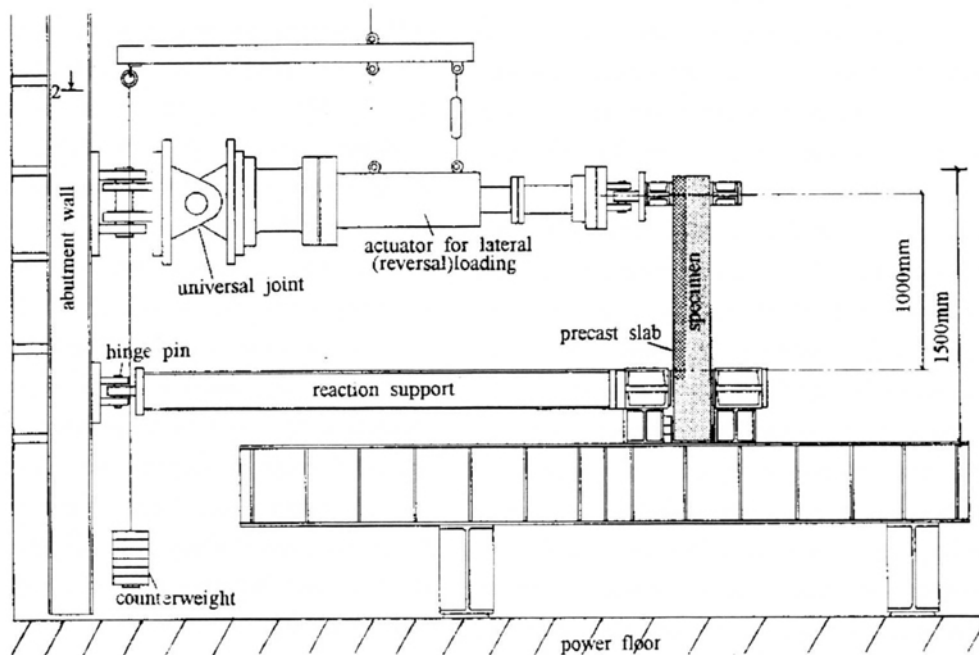


Fig 2. Special equipment with an actuator for specimens loading

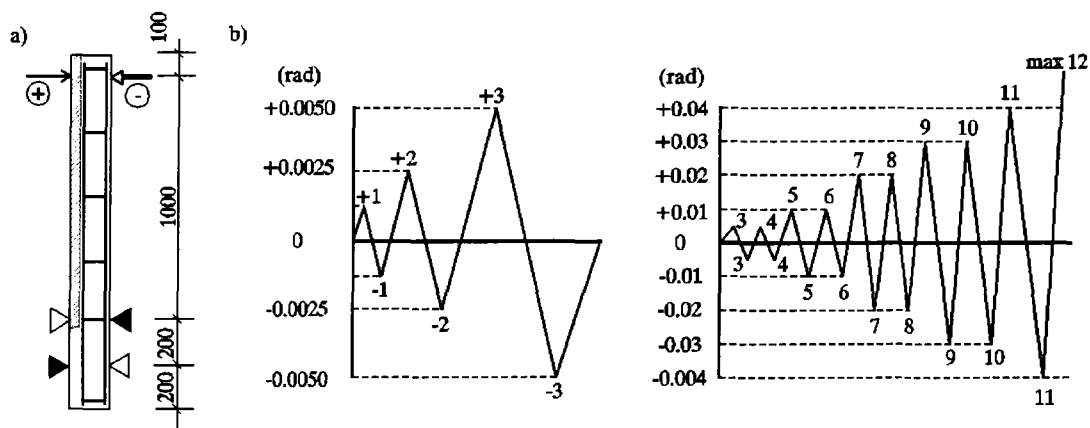


Fig 3. Loading scheme (a) and program (b) of composite slab specimens

Composite slab specimens under one-directional monotonous loading (S-2 and S-6) were subjected to maximum loading point displacement of 120 mm only in backward (+) direction. For composite slabs, which were subjected to the reversal lateral load (S-1, S-3, S-4, S-5 and S-7) from the 3rd cycle, the specimen was effected by the same displacement at the loading point repeatedly in both reversal directions. Load intensity, slab deflections, strains of in-situ concrete, reinforcement longitudinal bars and joint stirrups, and sleeps between precast and in-situ concrete parts of slabs were measured. All measurement instruments were connected with the computing logger and received by the personal computers.

### 3. Test results

At the forward loading of the test specimens the precast members were in the compressive zone of slabs. At the backward loading these members were under tension. As the support depth of the precast slabs was only 30 mm, they did not accept tension forces acting on joint core faces. Therefore, the effective depth of composite slabs was  $d_1 = h_1 - c$ , if they were in compression zone and  $d_2 = h_2 - c$ , if precast members were in tensile zone. The ratio  $d_1/d_2$  is close to the flexural moments  $M_1/M_2$  caused by gravity and lateral forces at side faces of slab-wall joints of multi-storey panel buildings.

Only small slip deformations were observed between precast and in-situ concrete surfaces until the composite slab flexural strength was reached. However, in the presence of the initial concrete shrinkage cracks composite slab flexural strength was reached.

However, in the presence of the initial concrete shrinkage cracks around the attachment zone (initial cracks were observed in specimens S-2, S-4, and S-6), adhesion between cast-in-place and precast slabs was somewhat weaker and it could influence the ultimate bearing capacity for specimens without joint stirrups. At the backward loading, when the precast slab reinforcement received compressive stress, precast slab concrete compression failure at the support was observed. And it was believed that the displacement between slabs in this case could be initiated. However, at the forward loading a crack in the precast slab edge at the support developed in all specimens. Besides, at high rotational displacement both slabs with cotter attachment worked jointly without considerable slip. The largest relative displacement at the flexural strength arose in the distance of the 35...50 mm from composite slab support for the specimens with plane concrete surface and concrete shear cotters when some longitudinal cracks between precast and cast-in-situ slab parts were developed. There were no considerable differences in the shape of hysteresis loops and final cracking patterns among all specimen type (Fig 4). The joint stirrups did not influence the strain performance in the main longitudinal reinforcement. At ultimate loading state the tensile and compression reinforcement bars behaved in yield and elastic conditions, respectively.

The test data and results presented in Table 1 confirmed the structural performances of composite multi-hollow slabs. The theoretical values of ultimate shear forces were expressed by the formula:

$$V_{u,cal} = M_{u,cal} / l = mbd^2 f_c / l \quad (1)$$

Type of slabs	Surface of precast slab	Reinforcement of in-situ slabs	Cracking force, $V_{cr}$ , [kN]	Ultimate force $V_u$ , [kN]					
				Forward loading			Backward loading		
				$V_{u,obs}$	$V_{u,cal}$	$\frac{V_{u,obs}}{V_{u,cal}}$	$V_{u,obs}$	$V_{u,cal}$	$\frac{V_{u,obs}}{V_{u,cal}}$
S1	Plain concrete	Double	9.9	32.5	40.1	0.81	66.4	64.4	0.97
S2	Plain concrete	Double	9.4	-	-	-	74.4	66.8	1.11
S3	With cotter attachment	Double	9.7	36.0	38.8	0.93	65.4	60.8	1.07
S4	With joint stirrups	Double	9.4	46.7	45.2	1.03	67.1	61.3	1.09
S5	With joint stirrups	Double	10.7	34.5	38.9	0.89	70.5	62.3	1.13
S6	With joint stirrups	Double	10.1	-	-	-	60.8	63.0	0.97
S7	With joint stirrups	Single	8.35	46.7	45.2	1.03	26.9	24.1	1.11

where the flexural strength of doubly reinforced concrete section  $M_{u,cal}$  was calculated by recommendations of the Architectural Institute of Japan [3].

#### 4. Safety assessment

The probability distributions of flexural member resistance  $R_{pr} = M_{pr}$  by (1) is close to the normal or lognormal laws. The bending moments  $M_g$  and  $M_q$  caused by permanent and long duration live loads, respectively, also obey these distribution laws. Thus, the normal probability distribution law is acceptable for the cumulative function of random vectors

$$R_w = R_{pr} - M_g - M_q \quad (2)$$

as the conventional resistance in bending of design members.

The probability distribution curves of reiterated transient bending moment  $M_w$  caused by stochastically independent accidental horizontal or gravity forces differ from the normal law considerably. Usually, the probability distribution of extreme accidental action effects obeys the extreme distribution law of the Type 1 [4, 5].

A failure of slabs on dangerous sections belonging to floor and wall joint faces may occur due to nature uncertainties of basic variables of action, material and geometrical properties of members together with uncertainties of design mechanical models as they are recommended by International Standard ISO 2394 [6]. Therefore, the performance function of continuous multihollow floor slabs can be written in the form:

$$Z = \xi_R R_w - \xi_w M_w = (\xi_{pr} R_{pr} - \xi_g M_g - \xi_q M_q) - \xi_w M_w \quad (3)$$

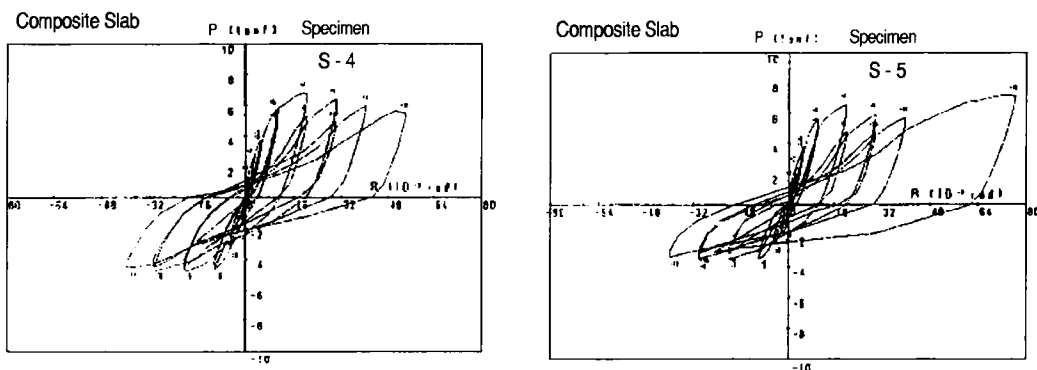


Fig 4. Hysteresis loops for slab load versus rotation angle for specimens S-4 and S-5

where  $\xi_{pr}$ ,  $\xi_g$ ,  $\xi_q$  and  $\xi_w$  are the factors of mechanical model uncertainties of slab resistance  $R_{pr}$  and action effects  $M_g$ ,  $M_q$  and  $M_w$ , respectively.

According to recommendations [7-9], the means and standard deviations of probability distribution of modelling uncertainty factor for flexural members resistance and moments can be evaluated by the values  $\xi_m = 1$  and  $\delta\xi \approx 0.1$ . Thus, the statistical parameters (means and variances) of the components of slab performance function by (2) can be expressed by the values:

$$R_{wm} = \xi_{Rm} R_{wm} = R_{pr,m} - M_{gm} - M_{qm} \quad (4)$$

$$\sigma^2 R_w = \sigma^2(\xi_R R_w) = R_m^2 (\delta^2 R_{pr} + \delta^2 \xi_{pr}) + M_{gm}^2 (\delta^2 M_g + \delta^2 \xi_g) + M_{qm}^2 (\delta^2 M_q + \delta^2 \xi_q) \quad (5)$$

$$M_{wm} = \xi_{wm} M_{wm} \quad (6)$$

$$\sigma^2 M_w = M_{wm}^2 (\delta^2 M_w + \delta^2 \xi_w) \quad (7)$$

where  $\delta R_{pr}$ ,  $\delta M_g$ ,  $\delta M_q$  and  $\delta M_w$  are the coefficients of variation of probability distribution of slab performance function components due to uncertainties of basic variables. It must be called an attention that for multihollow slabs the variability in dimensions has a significant effect on statistical parameters of their resistance in flexure.

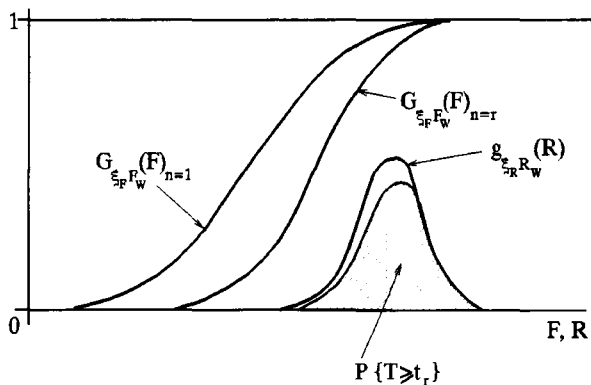


Fig 5. Model for calculation of the slab safety index  $P\{T \geq t_r\}$  by (8)

The conventional member resistance  $\xi_R R_w$  can be presented as a stationary random process. Therefore, according to the model presented in the Fig 5, the long duration safety index of floor slabs at dangerous sections can be calculated by the formula:

$$P\{T \geq t_r\} = P\{\xi_R R_w > \xi_w F_w\}_r = \int_0^{\infty} g_{\xi_R R_w}(R) G_{\xi_w F_w}(R) dR \quad (8)$$

Here  $T$  is the lifetime of a slab till its failure as a random variable;  $t_r$  is the preset service duration time of a building;

$$g_{\xi_R R_w}(R) = \frac{1}{\sigma(\xi_R R_w) \sqrt{2\pi}} \exp\left[-\frac{(R - R_{wm})^2}{2\sigma^2(\xi_R R_w)}\right] \quad (9)$$

is the probability density function of the slab conventional resistance  $\xi_R R_w$ , where  $R_w$  is by (2) and

$$G_{\xi_w M_w}(R) = \exp\left\{-r \exp\left[\frac{M_{wm} - R}{0.7794\sigma(\xi_w M_w)} - 0.5776\right]\right\} \quad (10)$$

is the probability distribution function of the bending moment  $\xi_w M_w$  caused by accidental horizontal or gravity forces the reiteration number of which in course of slab service life is  $r$ .

The slab safety index by (8) may be computed by numerical integration and Monte-Carlo simulation methods and must be no less than specified degree of structural safety.

## 5. Conclusions

1. Composite plane continuous multihollow concrete floors with stay-in-place precast mould slabs are to be used in erection of multi-storey cast-in-situ buildings.

2. Joint stirrups guarantee good adherence of precast and cast-in-situ concrete parts of floors in flexural failure.

3. Structural safety of continuous slabs of frame type panel systems can be assess by the probabilistic index (8) based on the normal distribution law of a multi-component vector of long duration variables and on the extreme distribution law of the Type I of reiterated transient actions.

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## KOMPLEKSNĖS DAUGIAKIAURYMĖS PLOKŠTĖS GELŽBETONINIUIOSE DAUGIAAUKŠČIUOSE PASTATUOSE

Alg. Kudzys

### S a n t r a u k a

Nagrinėjama galimybė daugiaaukščių pastatų monolitines perdangas įrengti iš kompleksinių gelžbetoninių plokščių, vietoj klojinių naudojant gamyklines plonas plokštes. Mažacikline kartotinė ir vienos krypties apkrova eksperimentiškai iširta septynių kompleksinių plokščių darbo geba ir stiprumas. Bandymams buvo naudotos trijų skirtingų iš anksto gamykloje išbetonuotų nenuardomų klojinių vaidmenį turinčios atlikti plokštės. Jų sandūros su monolitiniu betonu paviršiai buvo lygūs, su betoniniais kamšteliais ir su plieninėmis kilpomis. Sukloti ant klojinių plastikiniai vamzdžiai leido suformuoti daugiatsuštimę perdangos plokštę. Natūralaus didumo kompleksinių perdangų eksperimentinių bandymų rezultatai sugretinami su jungties stiprumo skaičiavimais.

Analizuojant kompleksinių plokščių patikimumą, pateikiamas praktiškas metodas nekarpytųjų perdangos plokščių tikimybiniam saugiumi įvertinti. Skaičiuojant atsižvelgiama į skirtingus ilgalaikių ir trumpalaikių kartotinių sistemos jėgų skirstinius bei į mechaninių modelių neapibrėžtumus.

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