Design of vertical joints in precast reinforced concrete shear walls

Design of Vertical Joints in Precast Reinforced Concrete Shear Walls

Dimensionnement de joints à cisaillement vertical dans le cas de parois préfabriquées en béton

Bemessung von Fugen zwischen vorgefertigten Wänden

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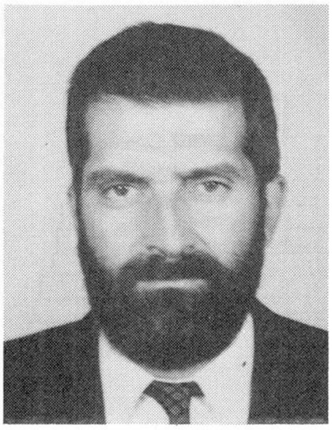

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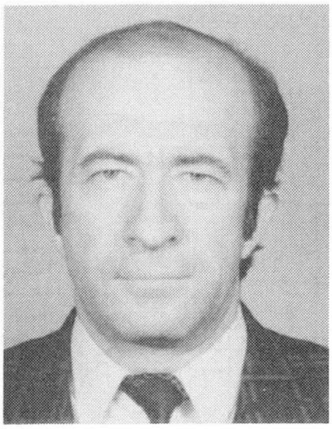

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SUMMARY

For vertical shear joints, new design formulae are presented in order to establish the most appropriate design requirements describing the real behaviour of these structural elements. The theoretical values evaluated using different formulae, are compared with the experimental shear forces. Numerical analyses were performed in order to complete the experimental tests and to offer adequate numerical procedures vs, simpler hand calculations for the shear force resistance of the vertical joints.

RÉSUMÉ

Pour ce type de joints, de nouvelles formules de dimensionnement sont présentées; les autres, déjà consacrées, sont rappelées en vue d'établir les meilleures recommendations possibles pour le projet de tels éléments de structure et d'en illustrer le comportement réel. Les valeurs théoriques calculées selon les différentes formules sont comparées avec des cisaillements expérimentaux. Des analyses numériques ont complété ces essais et permettent donc d'offrir les procédures numériques et manuelles les plus simples pour le calcul de la résistance au cisaillement des joints verticaux.

ZUSAMMENFASSUNG

Es werden Formeln für die Bemessung der auf Schub beanspruchten Vertikalfugen von Wandscheiben vorgestellt, die das reale Verhalten dieser Strukturelemente genauer veranschaulichen sollen. Die mit verschiedenen Formeln berechneten theoretischen Werte der Schubkräfte werden mit den Versuchsergebnissen verglichen. Diese Versuchsergebnisse wurden durch numerische Analysen ergänzt, um Berechnungsformeln für die auf Schub beanspruchten Vertikalfugen aufzustellen.

1.FORCE DISTRIBUTION IN VERTICAL SHEAR JOINTS

Known the shear force distribution in the vertical shear joints of the reinforced concrete shear walls assembled from large precast panels is a major task in the designing of these structural elements. In a shear wall or a floor assembled from large precast panels the joints are, in general, the weakest link within the system. Therefore an elastic material behaviour can be assumed for the panels, since cracks and shear deformations only appear in the joints.

In accordance with the above mentioned facts, in structural systems composed of rectangular subunits the panels can be discretized with rectangular elastic finite elements in the plane state of stress. The reinforcement can be introduced aproximately through modified modulus of elasticity for the concrete. As proposed in [1] the behaviour of the finite elements for the joints can be simulated by a pair of orthogonal springs at each end. Their characteristics diagram are coupled through an interaction diagram P_N-P_T as in fig.1. This characteristic and interaction diagram are based on a nonlinear incremental analysis or on results from suitable experiments. The latter procedure gives more realistic values, but is more expensive.

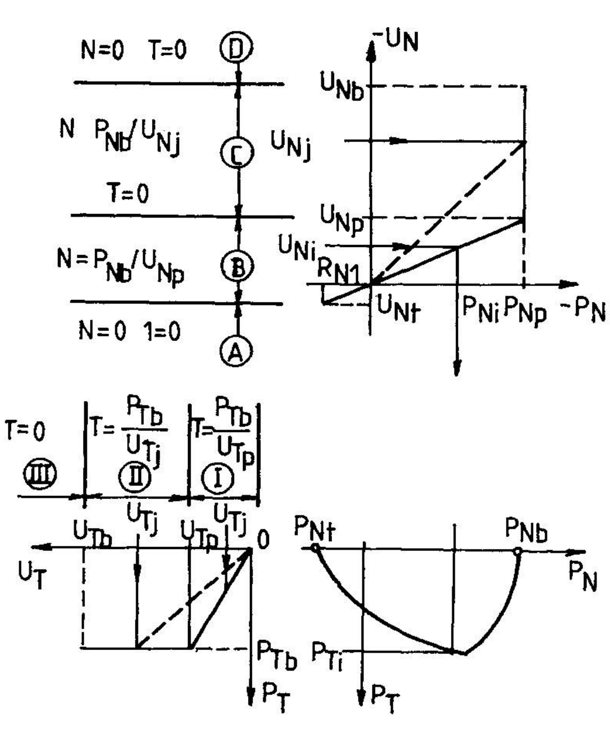


Fig.1 Interaction diagram P_N-P_T

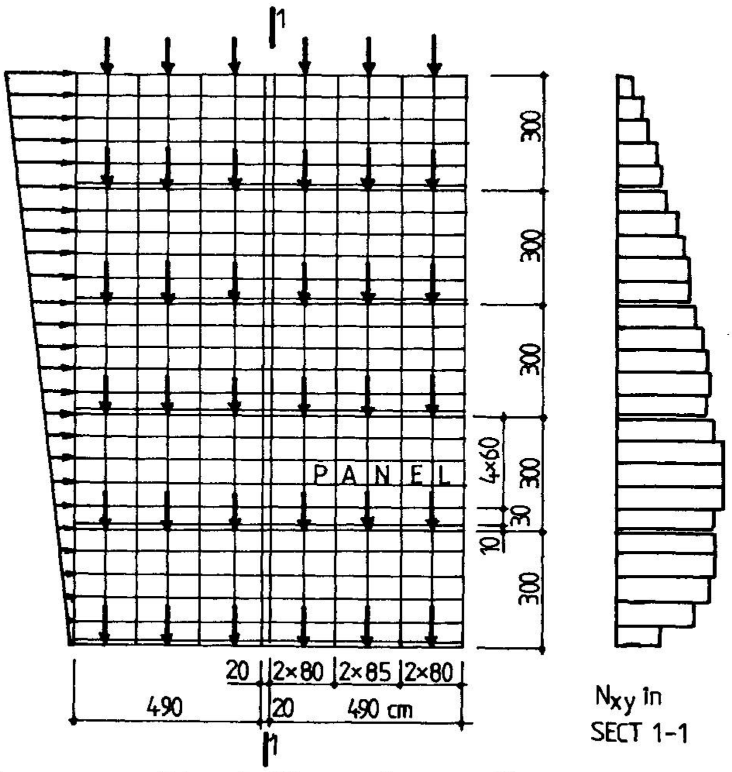


Fig. 2 Plane shear wall

The nonlinear analysis of the behaviour of the joints takes place iteratively, wereby within the succesive iteration steps, each step is calculated linearly-elastic. For the mean value of the normal and tangential displacements in a joint finite element, U_{Ni} and U_{Ti}, Fig.1 allows to obtain the updated rigidities N and T of the springs. It can be seen that the nonelastic behaviour is described by secant rigidities.

In order to establish the force distribution in the vertical shear joint, the plane shear wall in Fig.2, composed from ten large precast panels was analysed under given vertical and horizontal loads. It was found that all joints remain in the elastic range for a given combination of the horizontal and vertical forces which are applied on the shear wall in an structure analysis. For this reason the results for the vertical joint are comparable with results obtained with the approximative method for the analysis of shear walls with holes. Yet, the latter method cannot be applied for a nonlinear analysis of the coupling beams in shear walls. These conclusions are the basis for the new design formulae of the vertical shear joints.

2. NUMERICAL ANALYSIS OF THE SHEAR FRACTURE PROCESS

Experimental and numerical analysis of the shear fracture shows important features about the contribution of the tensile strenght in this process. Therefore, the authors developed experimental and theoretical research programms aimed to establish the contribution of the tensile strenght to the total amount of the shear strenght in the shear structural elements.

The numerical analysis is performed with anisotropic reinforced concrete elements [3]. This modell was adopted because of his well known performancies in modelling plain and reinforced concrete.

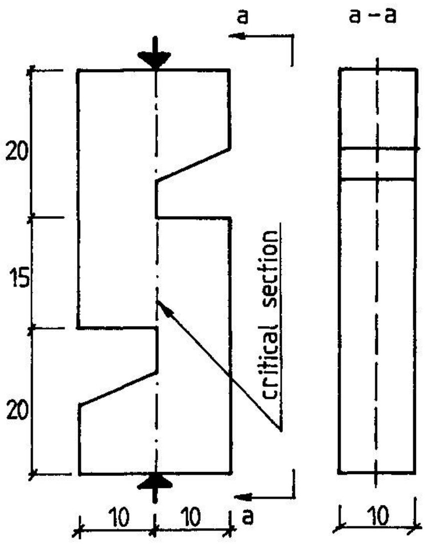
The nonlinear process hich may be developed in the structure after reaching the elastic limit are: crack formation, crack closing, crack reopening, plasticity of uncracked or cracked concrete, crushing of the compressed concrete.

Element stiffness D_{RC} is formed for every physical state of the material by superposing concrete stiffness D_C and reinforcement stiffness D_R, taking into account the reinforcement ratio μ:

$$D_{RC} = D_C + \mu D_R \tag{1}$$

Reinforcement is assumed to be uniformly distributed over the finite element, with perfect and continuously adherence to the concrete. Cracks are considered as smeared cracks. For the concrete a combined v.Mises-Navier behaviour criterion is considered, while for the reinforcement an elastic-plastic bilinear behaviour is assumed [2].

In [4] a set of 22 experimental models were analysed. The model is presented in Fig.3. An typical representation of the shear and tensile strenght in the critical section of the element is illustrated in fig.4.



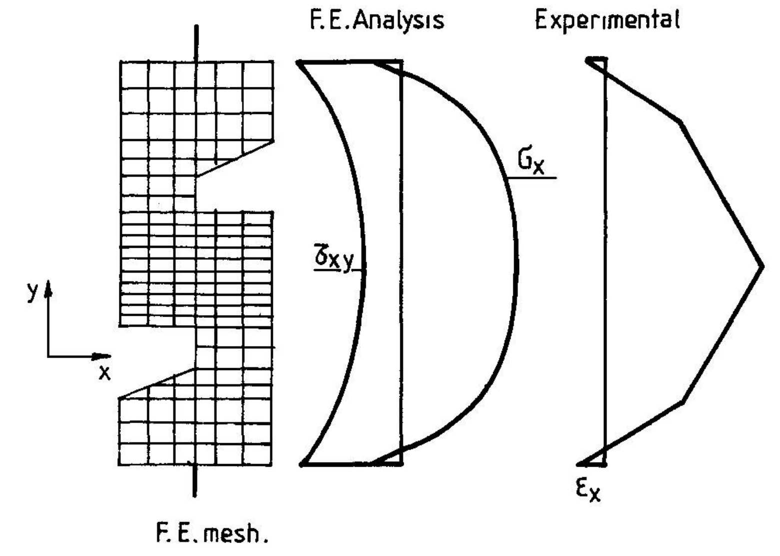


Fig.3 Shear modell

Fig.4 Strenght in the critical section

The main conclusion of these analyses taking into account also the experimental results is that between the shear strenght f_m and the tensile strenght of the concrete f_{ct} there is the following relation:

$$f_m = 1.5 f_{ct} \tag{2}$$

The proposed value $f_m = 1.5f_{ct}$ can be adopted for the design of plain or reinforced concrete structural elements in shear.

3. EXPERIMENTAL PROGRAMME

The accomplished researches were meant to observe the hysteretic behaviour of vertical joints under cyclic - alternating loads, their capacity to absorb and dissipate energy, the cracking and collapse mechanism. The experimental shear forces were compared to theoretical values calculated with various formulae, including the relation proposed in the new version of the Romanian technical instructions concerning the design of building with large, precast panel structures [5].

Four models of the same vertical joint were tested. The geometrical scale used was that of 1:1 (M1-M4). The experimental model shaped of two panel parts and the one-storey high vertical joint is given in fig.5.

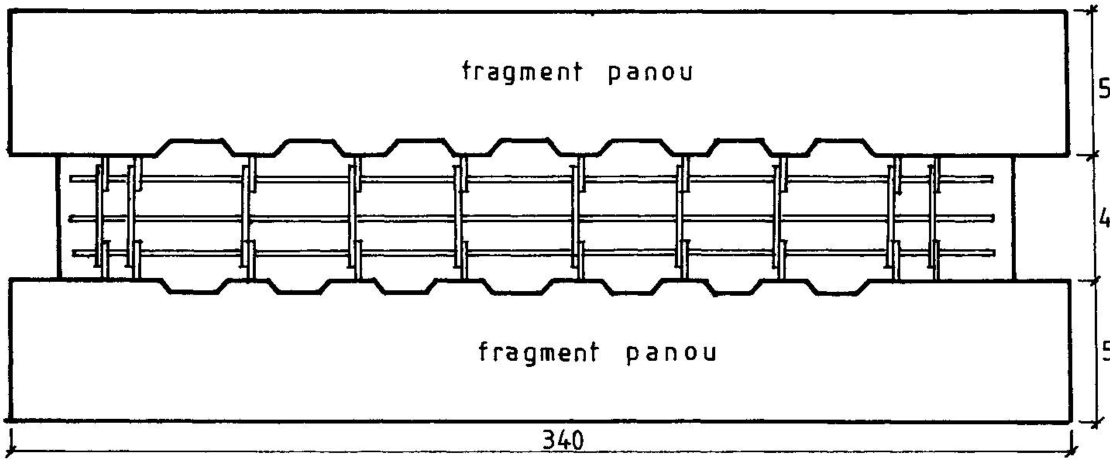


Fig. 5 Experimental model

The joint was tested in turned down position, on the narrow side, the model being inserted into the testing device schematically rendered in fig.6.

The testing methodology applied in the mentioned experiments was taken from the RILEM specifications concerning cyclic load testing. The monotonous M1 model testing served to establish the reference data required in the testing of the other three models under cyclic-alternating loads, according to the imposed deformation methodology [6].

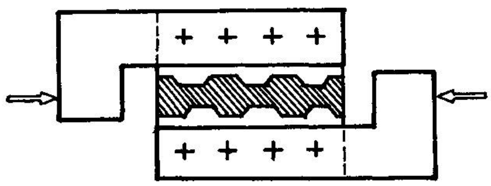


Fig. 6 Testing device

4. EXPERIMENTAL MODEL BEHAVIOUR

Generally, the behaviour of the experimental models points out cracks at the interfaces panels-joint, followed by the cracking of the joint in-situ concrete. The collapse results from shear failures of the keys in the joint. Consequently, the contribution of the in-situ concrete to the resistance of keyed joints is more dependent upon the resistance of cconcrete to tension than upon its resistance to compression.

At collapse, the cracking image typical for the tested models is that shown in fig.7.

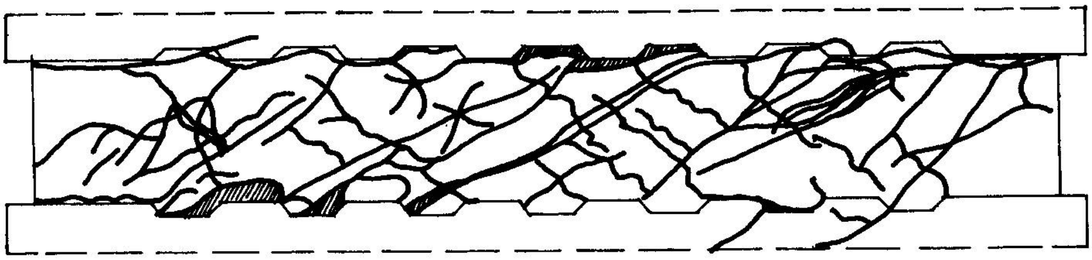


Fig.7 Cracking at collapse

The ultimate loads (capable shears) of the experimental vertical joints were calculated in view of comparing them to the experimental results.

Characteristics of the materials used (f_c, f_{ct}, f_y), the geometrical (A_{key}, A_{crush}) and reinforcement (A_s) features have been considered in this respect.

Four estimating relations were used:

- the relation proposed by CEB [7]

$$R_{jv} = \beta_1 A_{key} f_c + \beta_2 A_s f_y \tag{3}$$

- the relation proposed by P101-78 [5]

$$R_{jv} = A_{crush} f_c + 0.8 A_s f_y \tag{4}$$

- the relations proposed by Tassios and Tsoukantas [8],[9]:

$$R_{jv} = A_j \tau_u = A_j (0.15\lambda f_c + \mu \rho f_y + 1.8\rho f_{ct} f_y) \tag{5}$$

- the relation proposed in the new version of P101, as a result of the experimental behaviour of the joints in conjunction with relation (2):

$$R_{jv} = \min \left\{ \begin{array}{c} 1.5 A_{key} f_{ct} \\ A_{crush} f_c \end{array} \right\} + 0.8 A_s f_y \tag{6}$$

The estimated collapse force values compared to the experimental ones are given in Table 1:

Exp. mod.	f_c (MPa)	f_{ct} (MPa)	f_y (MPa)	A_{key} (cm²)	A_{crush} (cm²)	A_s (cm²)	R_{jv} [kN] calc. with rel. (3)	rel. (4)	rel. (5)	rel. (6)	Exp. val. R_{jv} [kN]
M1	23.54	2.17	360	1904	336	11.3	680	1116.5	1168.7	946	1150
M2	21.27	2.04	360	1904	336	11.3	650	1000	1103.7	905	1070
M3	30.53	2.58	360	1904	336	11.3	773	1351	1368.2	1064	1170
M4	29.96	2.55	360	1904	336	11.3	765.5	1332	1352	1054	1110

Table 1 Experimental and theoretical results.

The notations of the material characteristics used in Table 1 are those from the CEB Draft Guide [7].

5. CONCLUSIONS

As far as the possible shears in the joint are concerned, relations (5) and (6) offer values closer to the experiment. It is obviously that the relation (6) gives values under the experimental ones.

The experimental behaviour of the models shows that the contribution of the in-situ concrete to the resistance of keyed joints is more dependent upon the strength of concrete to tension than upon its strength to compression.

The joint behaves well with cyclic-alternating loads, since the joint resists without cracking to larger shear effort values as compared to the maximum shears that might occur in the joint in case of an earthkuake.

The adequate joint behaviour to cyclic-alternating loads is confirmed by the curve position in fig.8 as well

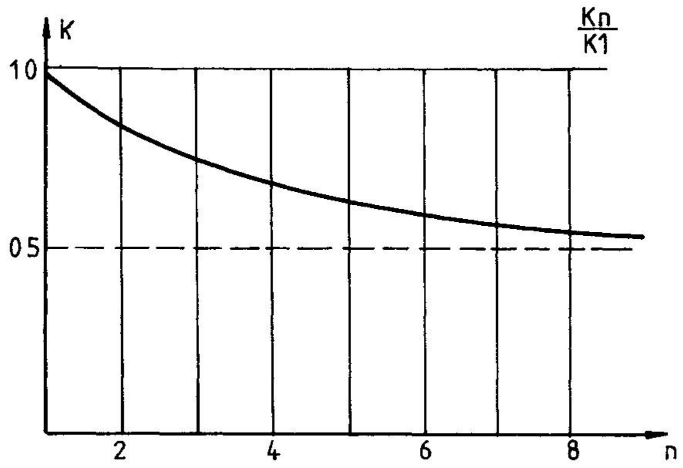


Fig.8 Stiffness decrease

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