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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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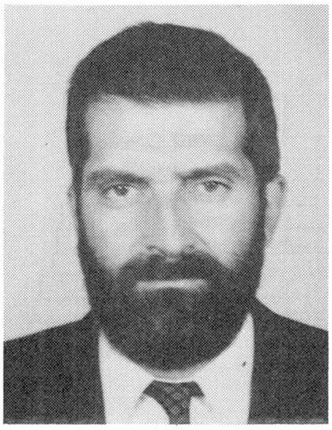

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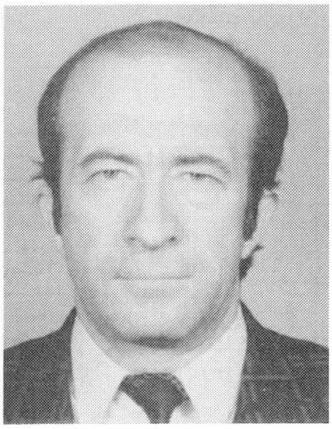

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SUMMARY

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.

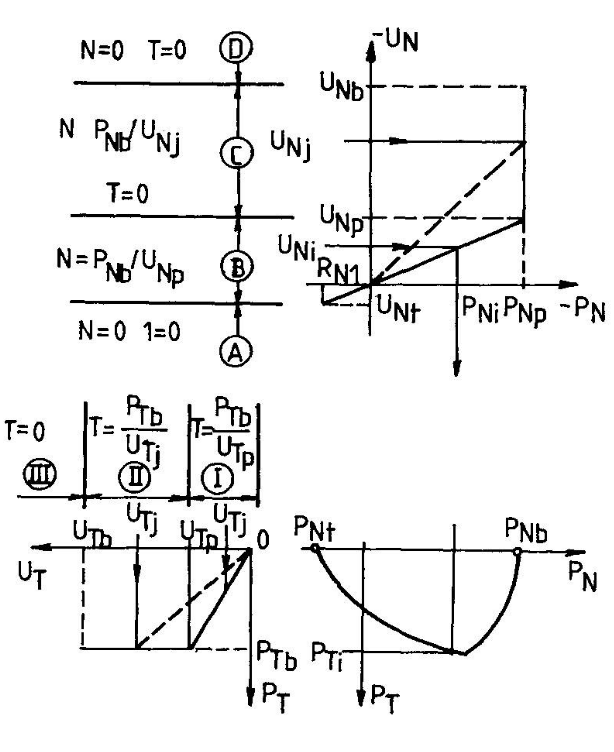

Fig.1 Interaction diagram P_N-P_T

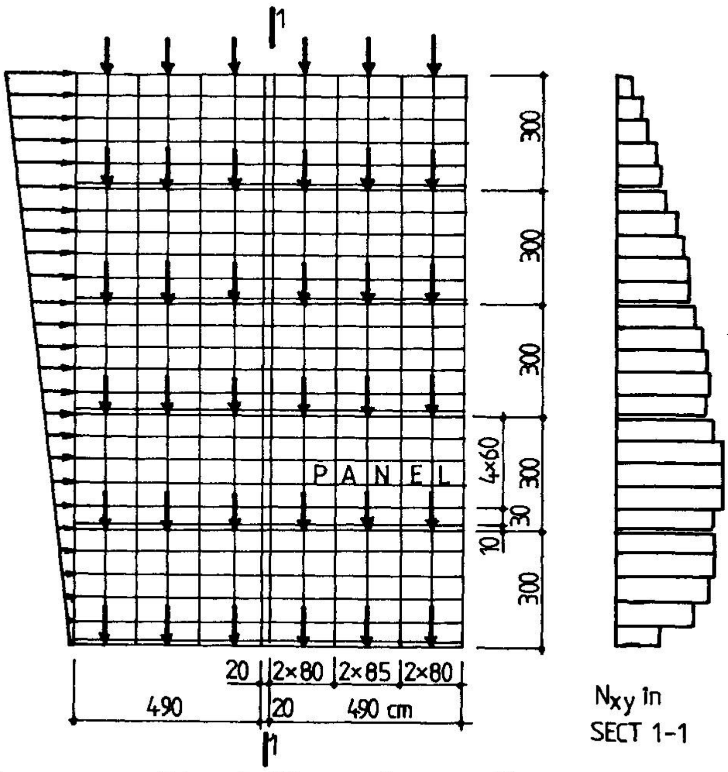

Fig. 2 Plane shear wall

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.

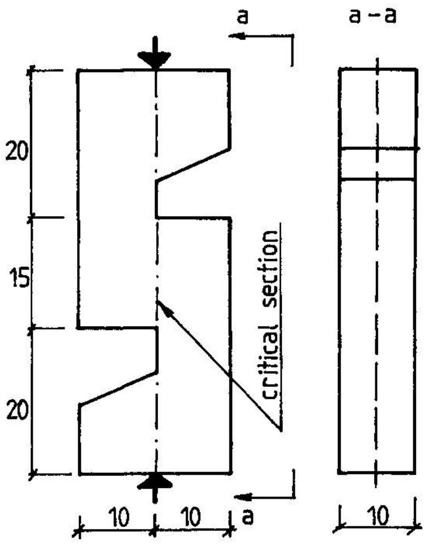

Fig.3 Shear modell

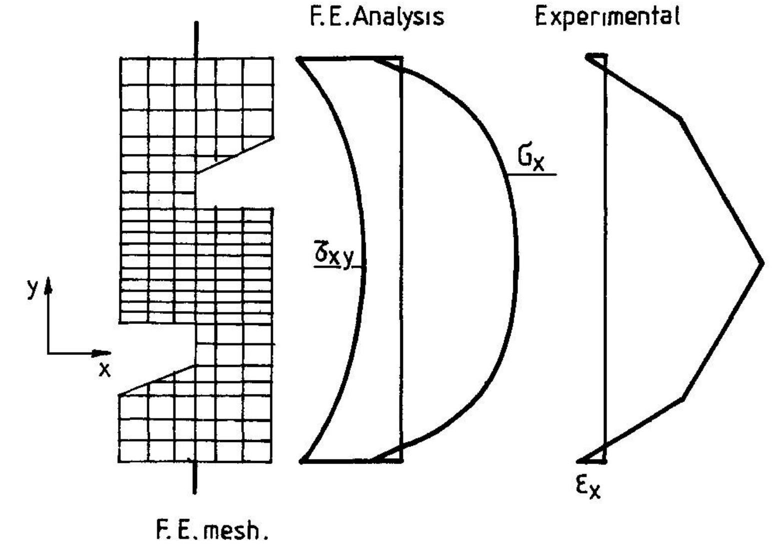

Fig.4 Strenght in the critical section

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

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

The proposed value $f_{\bullet} = 1.5f_{ct}$ 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.

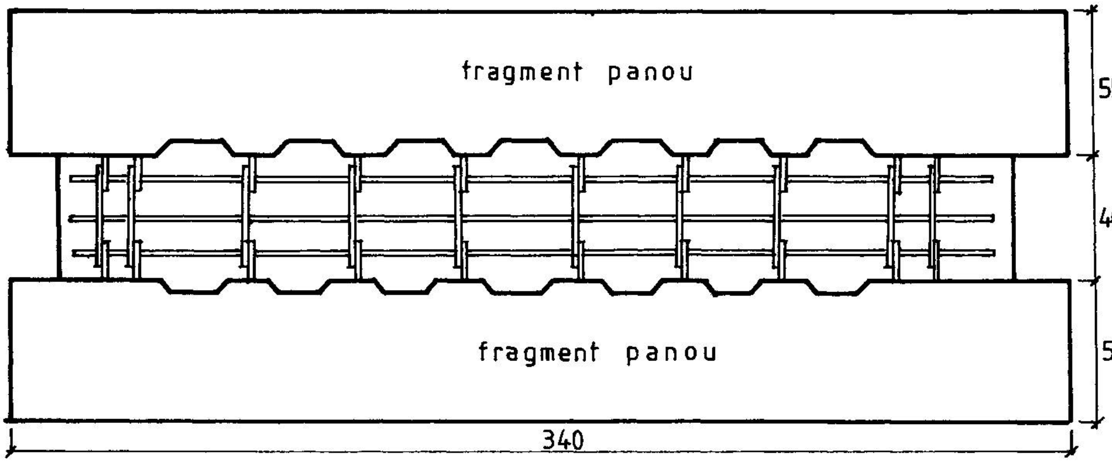

Fig. 5 Experimental model

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].

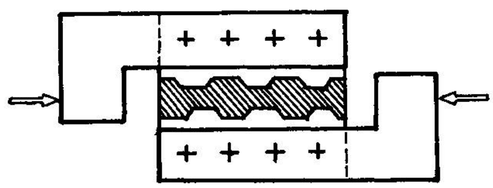

Fig. 6 Testing device

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.

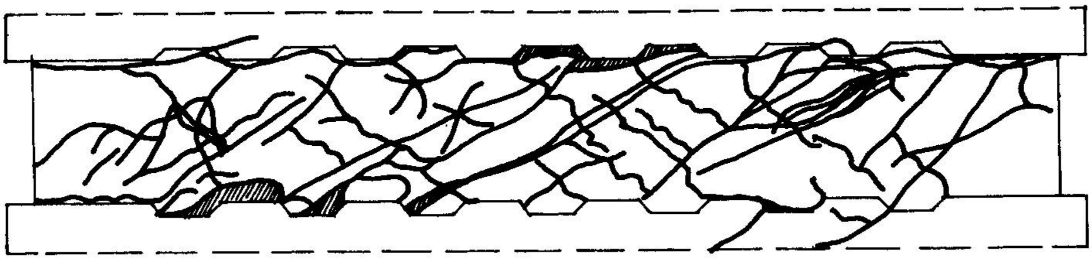

Fig.7 Cracking at collapse

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]

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

- the relation proposed by P101-78 [5]

$$R_{jv} = A_{crush} f_c + 0.8 A_s f_y \qquad (4)$$

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

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

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

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

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

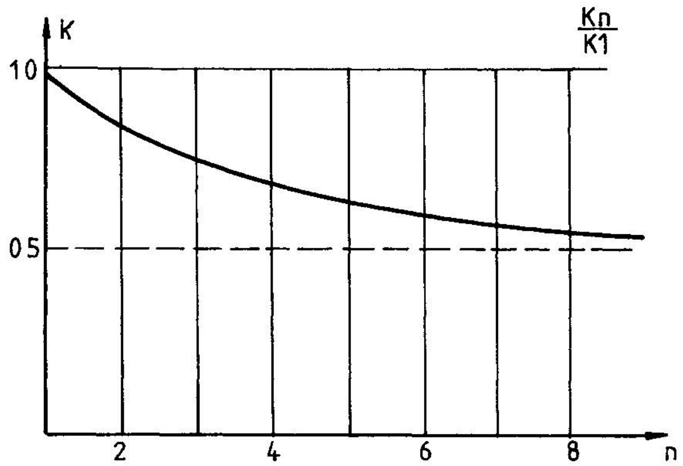

Fig.8 Stiffness decrease

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Influence of Contact Surface Problems on Design Practice

Influence de problème des joints de contact sur le dimensionnement

Einfluss von Kontaktflächenproblemen auf die Bemessungspraxis

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SUMMARY

Large buildings contain construction joints which may have a great influence on the loadbearing capacity of the structure. The design concepts of various standards for concrete structures will be presented and discussed briefly. Unfortunately, these design concepts treat the same problem in different ways. Based on a failure criteria of Mohr-Coulomb it will be shown how joints can be treated consistently in the design concept of strut-and-tie-models.

RÉSUMÉ

Les grands bâtiments comportent des joints qui influencent beaucoup la capacité portante de la structure; des concepts divers de dimensionnement sont brièvement présentés puis discutés. Le critère de rupture de Mohr-Coulomb constitue la base d'une démonstration décrivant comment les joints sont considérés dans le cadre d'un dimensionnement utilisant le modèle des bielles (analogie du treillis).

ZUSAMMENFASSUNG

Grössere Bauwerke beinhalten Fugen, die einen grossen Einfluss auf die Tragfähigkeit der Konstruktion ausüben können. Für Betonbauwerke werden Bemessungskonzepte verschiedener Normen vorgestellt und kurz diskutiert. Leider wird das gleiche Problem in diesen Bemessungskonzepten unterschiedlich behandelt. Auf der Grundlage eines Mohr'schen Versagenskriteriums wird aufgezeigt, wie Fugen in dem Bemessungskonzept der Stabwerkmodelle einheitlich behandelt werden.

1. DESCRIPTION OF THE PROBLEM

There are many structures where forces have to cross contact surfaces. In concrete structures this occurs for example at cracks or at construction joints between cast in situ or precast elements. The behaviour of the contact surfaces cannot solely be described by well-known material characteristics like the tensile and compressive strength, but also the surface condition (like smooth or rough) must be considered. Therefore an additional failure criterion for the contact surface must be defined.

In the following an attempt is made how the contact surface problems in connection with construction joints are treated in the consistent and translucent dimensioning concept of strut-and-tie-models according to Breen /B2/ and Schlaich /S1,S2/. This is desirable because contact surface problems may have an important effect on the structural behaviour. Up to now they are dealt with not systematically and partly unsufficiantly in the design practice.

2. PRESENT DESIGN PRACTICE AND EXISTING CODES FOR CONCRETE STRUCTURES

In codes for concrete structures these problems are so far treated either as dimensioning problems for shear and normal stress in the contact surface or/and in terms of construction requirements.

2.1 German Standards

The German Standards /D2/ require a sufficient roughness for longitudinal joints between prefabricated parts and concrete cast in situ. The description of the appropriate surface condition is only given in a comment upon the Standards /E1/. An increased transverse reinforcement is needed, apart from a few exceptions, and the permissible shear stress is limited to 60 % of the regular value. The specialities of construction joints transverse to the loadbearing direction are specified only for prestressed structural members. Thereby a rough (or keyed) surface is a precondition, too. Additionally a value for the compression strength for the joint section is given.

2.2 ACI Standard 318-77

In the ACI Standard 318-77, section 11.7 /A1/ the dimensioning of construction joints is described on the basis of the shear-friction-theory which was originally developed by Birkeland /B2/ and Mast /M1/. An ultimate shear force V_u is defined for rough surfaces.

$$V_u = 1.0\, A_s f_y < \begin{cases} 0.2 f'_c A_c \text{ [N]} \\ \text{or } 5.5\, A_c \text{ [N]} \end{cases}$$

A_s : total crossectional area of reinforcement across interface
f_y : yield strength of reinforcement < 420 N/mm^2

Thereby rough means a clean interface which is free of laitance and roughened to a full amplitude of approximately 5 mm.

2.3 SIA-Standard 162

The SIA-Standard 162 /S3/ section 4 45 limits the concrete strength and steel strength of the prefabricated and connected parts.

Concrete: $f_{c,red} = 0.35\, f_{cw,min}$ Steel: $f_{y,red} = 0.80\, f_y$

According to section 6 06 2 only rough or keyed joints are permissible preferably perpendicular to the direction of the compression field. The rough joint should be realized by removing the cement-sand grout from the concrete surface.

3. A CONSISTENT DIMENSIONING-CONCEPT USING STRUT-AND-TIE-MODELS

3.1 Mathematical Description of the Loadbearing Capacity of Joints

The mathematical description of the loadbearing capacity of joints is based on the extended shear-friction-theory of Mattock /M1,M2/. It is drawn from a proposal by a FIP-Commission /F1/ supplemented by Walraven /W1/. The results are shown graphically in fig. 1.

$$\tau_u = K1\,(f_y + \sigma_N) + K2\, f_{ctk} \le 0.25\, f_{ck}$$

μ : A_s/A_c
f_y : yield strength of reinforcement <400 N/mm^2
σ_N : normal stress
f_{ctk} = $0.25\,\sqrt{f_{ck}}$

surf.cond.	K1	K2
very smooth	0.6	0.1
smooth	0.6	0.2
rough	0.9	0.4

Table I

The coefficient K1 is equivalent to a frictional coefficient and the term K2 f_{ctk} corresponds to the so-called cohesion, which here is proportional to the tensile strength of the concrete. Both coefficients depend on exactly described surface conditions which are subdivided in three categories (Tab. I). By comparison with an extensive series of experiments carried out by Daschner /D1/ Walraven found out that the concept is on the safe side, if the steel strength is limited to $f_y < 400\ N/mm^2$. For comparison, the parabolic description of the joint behaviour according to Tassios /T1/ is shown in fig. 2.

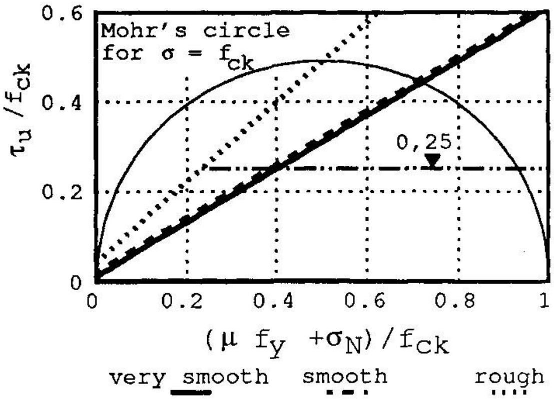

Fig. 1: acc. to FIP

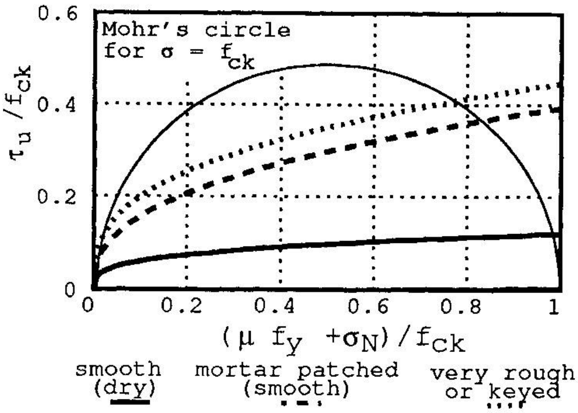

Fig. 2: acc. to Tassios

3.2 The Influence of a Joint on the effective Strength of a Prism

With the above given proposals the loadbearing capacity of a prism with a joint (fig. 3) is given according to Zelger and Rüsch /Z1/ and can be seen in fig. 4 and fig. 5. The diagrams clearly show that the capacity of the prism is only reduced by the joint, if the inclination of the joint exceeds a critical angle α_{crit}. If the inclination is smaller than α_{crit} the prism fails in concrete-compression. These dependencies can also be shown in a different way using Mohr's circle (see: Guckenberger /G1/ and Basler /B1/).

according to FIP /F1/:

$$\frac{\sigma_u}{f_c} = \frac{K2\ f_{ctk}}{f_c}\ \frac{1 + \tan^2 \alpha}{\tan \alpha - K1} \leq 1$$

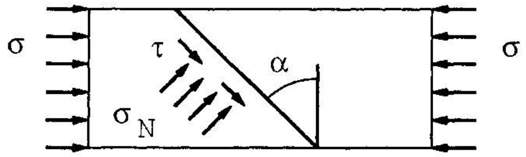

Fig. 3: prism with a joint

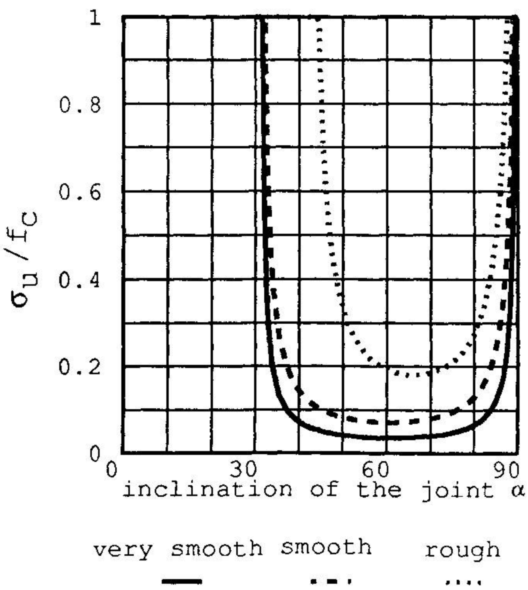

Fig. 4: acc, to FIP

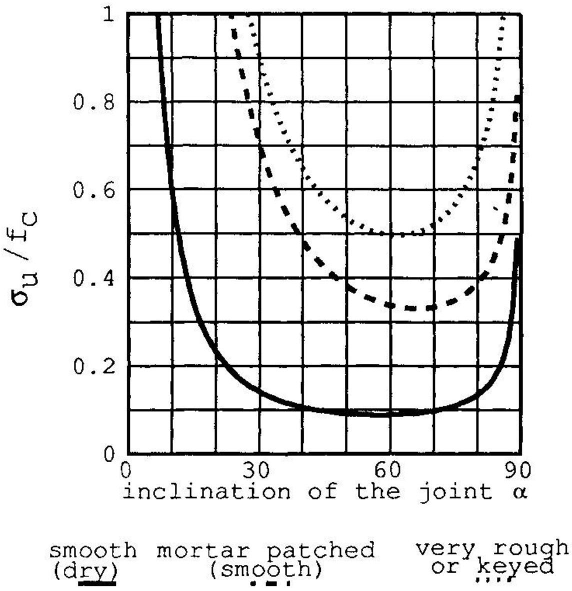

Fig. 5: acc. to Tassios

The following diagrams show in a different way some characteristics of the loadbearing behaviour of the prism. They are based on the FIP proposal, because it satisfies the demands of the structural engineer better than other proposals. Fig. 6 shows the dependence of the critical inclination on K1 and K2 f_{ckt}. It is remarkable that the critical inclination of the joint is almost totaly independent of the term K2 f_{ckt} (cohesion). Fig. 7 shows the dependence of the minimal effective strength on K1 and K2 f_{ckt}.

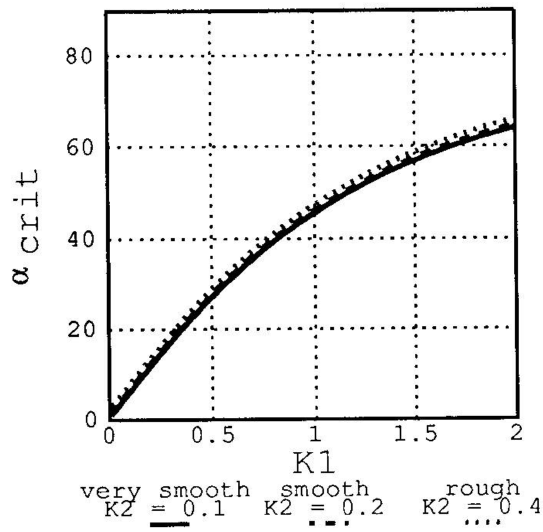

Fig. 6: crit. inclination

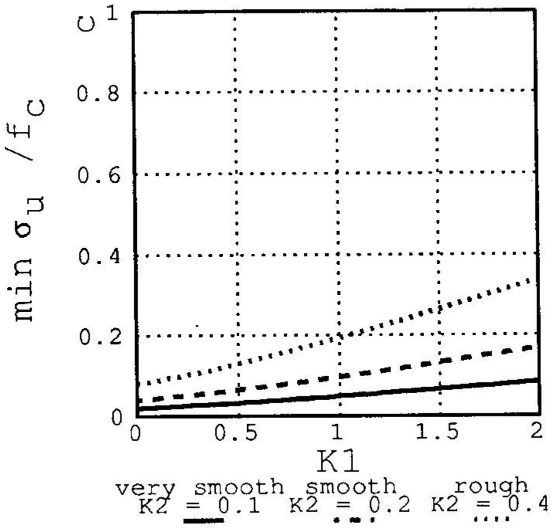

Fig. 7: minimal effective prism strength

With the above diagrams some problems of dimensioning structural members with joints can be solved directly, as e.g. for a joint in an arch bridge between the bottom edge of a column and the supporting arch.

3.3 Joints in the web of a Beam

In the following the influence of the joint on the ultimate shear force of a beam is explained. Two cases are considered either a longitudinal joint between a precast part and a layer of cast in situ concrete or a transverse construction joints between two stages of construction or precast elements (fig. 8). The loadbearing behaviour of the beam is described with a strut-and-tie-model. Thereby the joint influences essentially the effective strength of the compression strut. It is assumed that the effective strengths of the chords are not reduced. This leads to the conclusion that the reduction of the ultimate shear force depends mainly on the inclination of the strut - and accordingly on the transverse reinforcement - and the roughness of the joint.

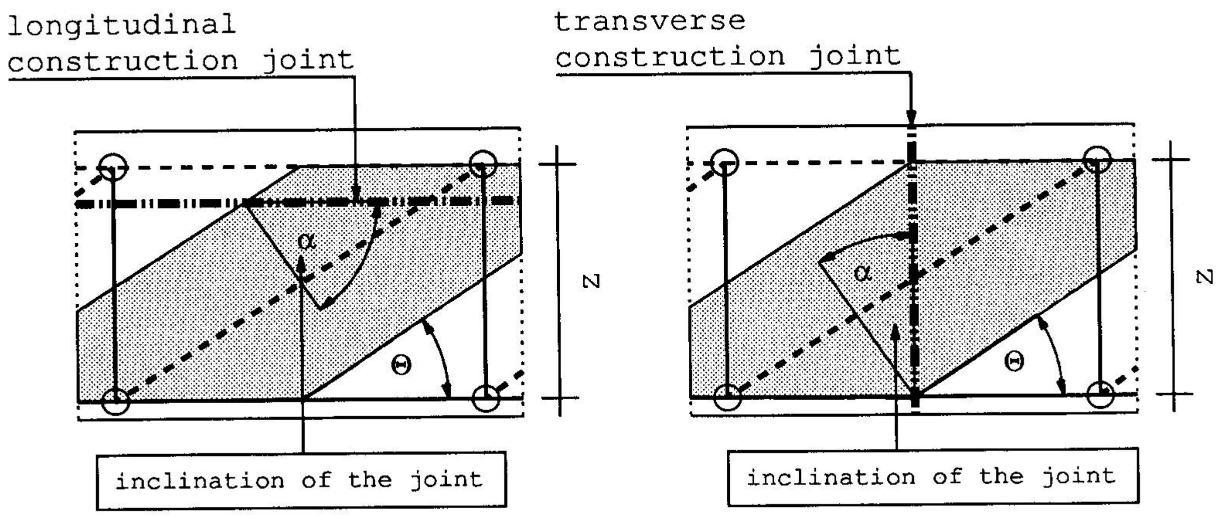

Fig. 8: strut-and-tie-model of a beam

The reduced ultimate shear force due to joint failure can be calculated as follows:

$$\frac{V_u}{b\,z\,\sigma_{cw,u}} = \frac{K2 \sin\Theta \cos\Theta(1+\tan^2\alpha)}{\tan\alpha - K1}\,\frac{f_{ctk}}{\sigma_{cw,u}} \leq 1 \qquad \text{for } \alpha > \alpha_{crit}$$

Considering $\alpha = 90° - \Theta$ in case of a longitudinal joint and $\alpha = \Theta$ in case of a transverse joint yields the curves shown in fig. 9 and fig. 10. In both cases the ultimate shear force of a beam without joints can be reached nearly if the surface is rough. However for a longitudinal joint an inclination 45° for the compression field and an accordingly increased tie reinforcement is required. For a transverse joint the inclination of the compression field varies between 30° and 45° as usually.

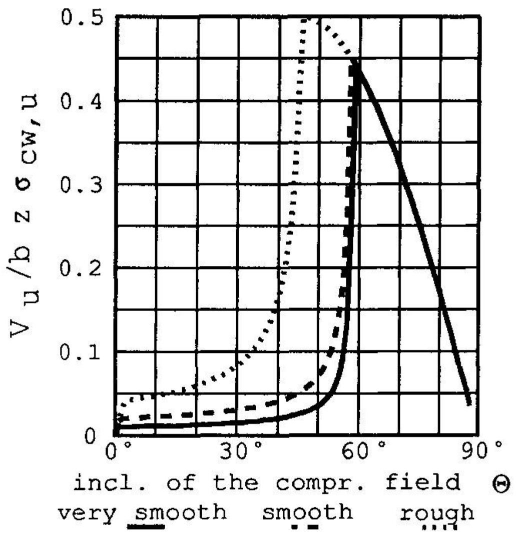

Fig. 9: Longitudinal joint

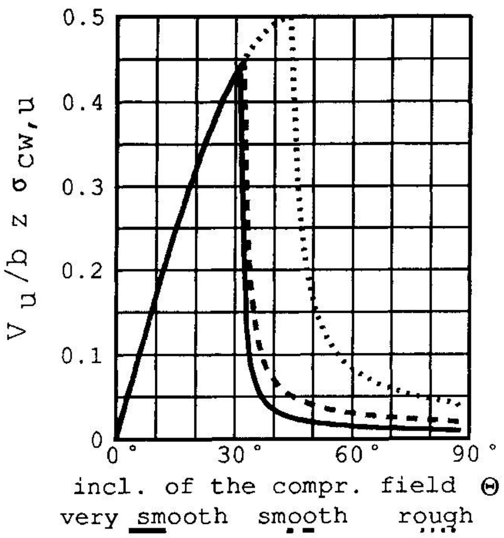

Fig. 10: Transverse joint

Fig. 12 and fig. 13 show diagrams for the necessary vertical shear reinforcement. For longitudinal joints highly increased reinforcement is required even if the joint is rough. For rough transverse joints only the maximum value for the ultimate shear force is reduced by the joint.

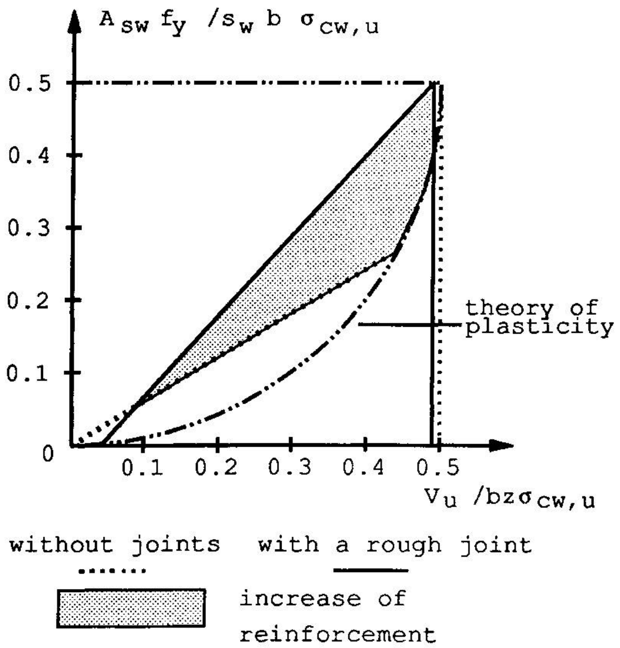

Fig. 11: Longitudinal joint

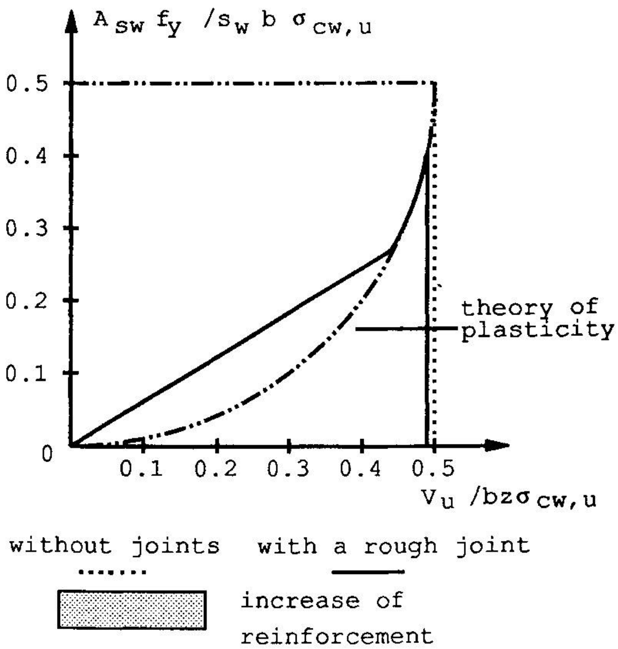

Fig. 12: Transverse joint

4. SUMMARY AND PROSPECTS

It was shown that the influence of a joint on a structural concrete member can be expressed by an reduced effective strength of the struts in a strut-and-tie-model. Therefore the normally given relation between the normal stress and the ultimate shear stress of a joint must be transformed into a relation between the inclination of the joint in a strut and of its effective compressive strength. This relation depends on the roughness of the contact surface, which has to be described exactly. Reineck shows a similar approach in /R1/ concerning the influence of friction mechanisms in cracks on the ultimate strength of the web compression field. The advantage of these methods is that it becomes more obvious how a structural member with a joint or cracks works and which element will fail under ultimate load.

Prospective efforts should aim in the direction of integrating composite structures with contact surface problems between their different materials in a consistent dimensioning concept. At the moment this is unfortunately complicated by the splitting of codes according to materials which leads to different approaches for the same problem. Further types of contact surface problems will arise in future, when the advantages of combining new or unusual materials will be used to realize new kinds of structures with different and novel qualities.

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Reliance on Tensile Strength for Cast-in-Situ Wall-to-Wall Joints

Résistance à la traction des joints clavés de parois coulées sur place

Tragfähigkeit von Fugen in Ortbetonwänden

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SUMMARY
The paper considers theoretical and practical problems concerning an estimation of the complete concrete tensile deformation diagram in the analysis of the load bearing capacity of keyed joints of cast-in-situ exterior and interior walls.

RÉSUMÉ
Cet article prend en considération les problèmes théoriques et pratiques d'une appréciation concernant le diagramme complet traction-déformation du béton, et ceci, dans le contexte de l'analyse de la capacité portante de joints clavés, appartenant à des parois intérieures et extérieures coulées sur place.

ZUSAMMENFASSUNG
Im Aufsatz wird das theoretische und praktische Problem der Bewertung des vollständigen Dehnungsdiagramms des Betons in der Tragfähigkeitsberechnung der Monolithverzahnung der Aussen- und Innenwände besprochen.

The connections of intersection walls constructed both of cast--in-situ concrete of different nature and class (Fig. 1) and of precast concrete members can be provided by using keyed joints. The cross-shaped and wavy dividing closures of asbestos cement sheets provided a rational solution ensuring technologically simple connections between walls of buildings and constructions. A remarkable cohesion of these sheets with concrete is characteristic. Therefore it is expedient to use new types of joints for connections of cast-in-situ exterior lightweigt and interior heavy concrete walls of buildings.

The experimental laboratory and site data indicate the necessity of new types joints with the division closures. The keyed joints of concrete walls lead to the economy and increase in both tolerable rigidity of the connections and stiffness of multi-storey buildings subjected to shear forces. Such forces are generated by horizontal wind or seismic actions, non-symmetrical vertical loading, shrinkage deformations, temperature gradient etc.

Shear carrying capacity of keyed joints depends on mechanical properties of wall concrete in tension. The shear strength of a key system belonging to one of wall connection members is closely related to the possibility of a redistribution of shear stresses both in a single key and among the connection keys.

Twenty four full-scale specimens of exterior and interior wall connections were tested to investigate the behavior of concrete keys and to establish a cracking resistance (a load rating) and a failure strength of keyed joints.

The shear capacity of keyed wall-to-wall joints can be determined by the following equation derived from a test analysis and modelling data

$$R = nR_1\,(1 - tg\alpha/tg\beta) + A_s f_y\, tg\alpha. \qquad (1)$$

Here n - the number of keys in weaker wall member; R_1 - shear strength of a single key; $tg\alpha$ - coefficient taking into account the shape of a crack surface and the position of a crack direction angle; $tg\beta$ - coefficient characterizing the key slope angle and the contract surface between asbestos cement sheet and concrete; A_s - cross-sectional area of horizontal reinforcing bars; f_y - steel yield strength.

The shear strength of a concrete key

$$R_1 = æ A_c f_v\,, \qquad (2)$$

where $æ$ - coefficient which helps to evaluate the influence of crack location on the cross-sectional area A_c of shear key; f_v - concrete shear strength.

The value of the strength f_v can be calculated on the basis of the empirical equation

$$f_v = \xi_v \sqrt{f_c f_{ct}}\,, \qquad (3)$$

where ξ_v = 0,7 for heavy and ξ_v = 0,5 for lightweight concrete;

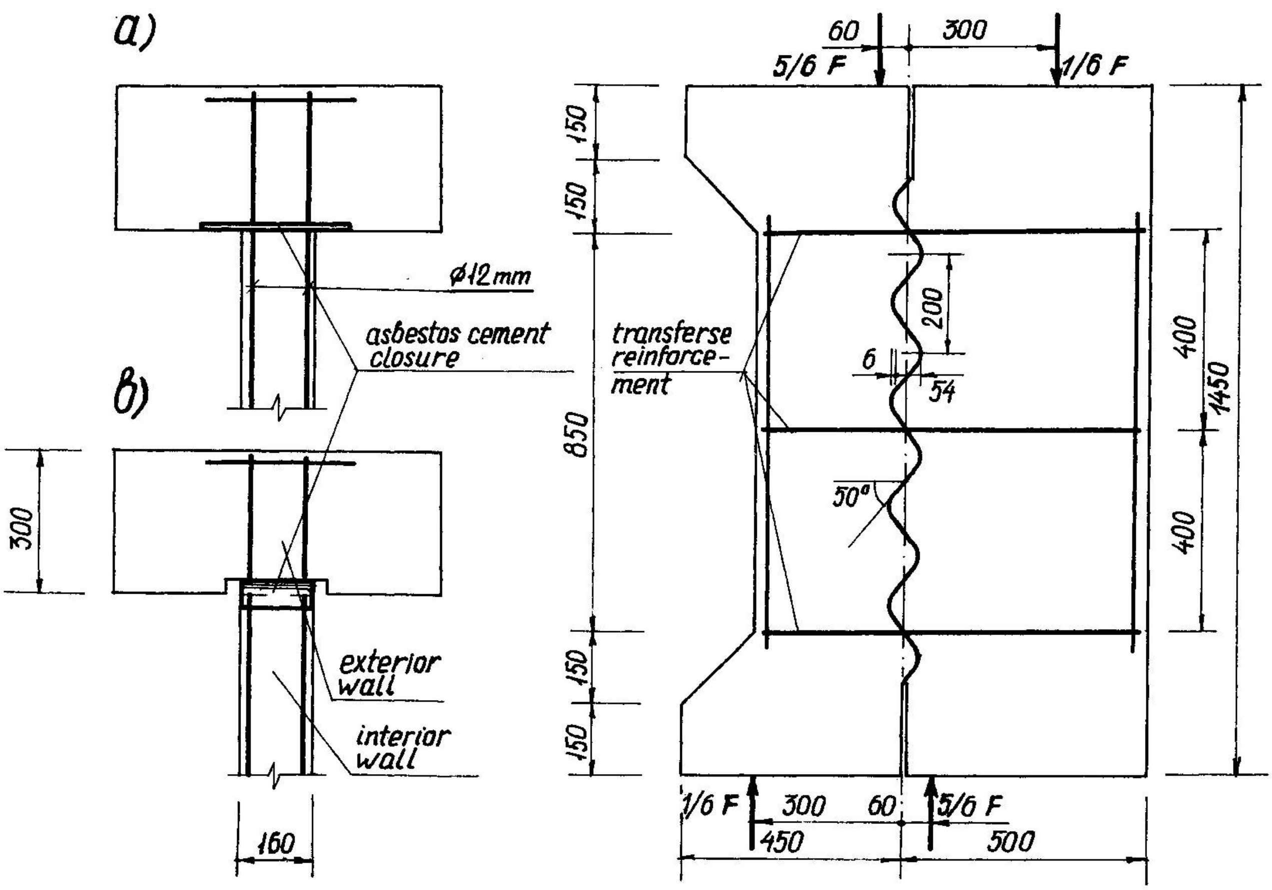


Fig. 1. Test specimens and test loading diagram of plain (a) and keyed (b) joints (dimensions in mm)

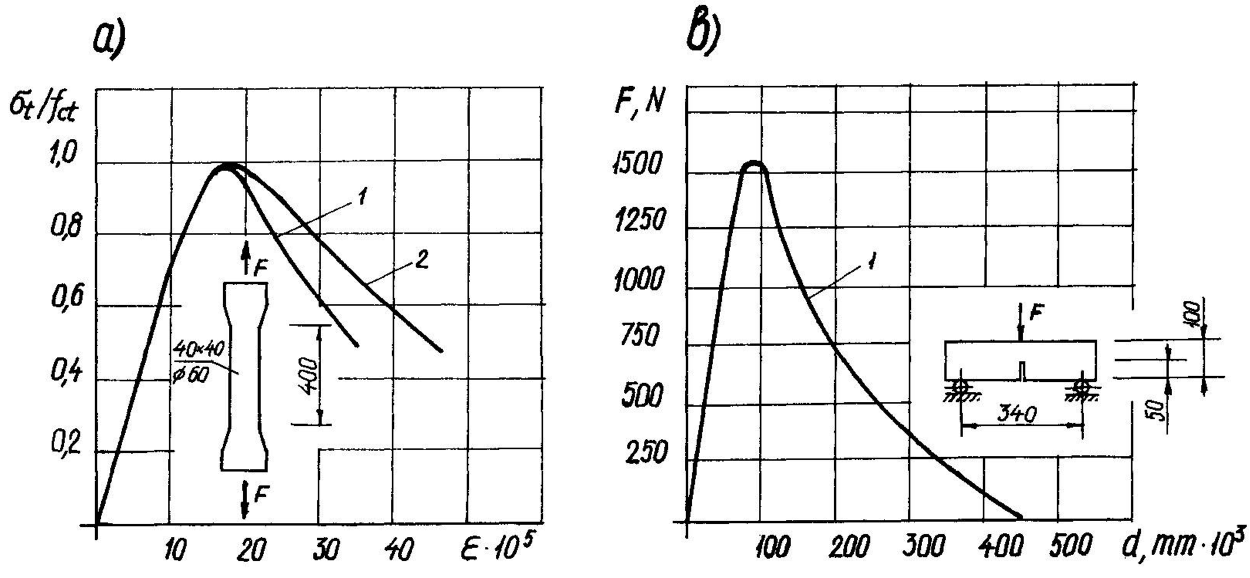


Fig. 2. A tensile (a) and a stable bending (b) tests of concrete (1) and reinforced concrete (2) specimens

f_c and f_{ct} - concrete compressive and tensile strength, respectively.

According to the theory of plasticity [1] the shear strength of concrete can be assumed as

$$f_v = \sqrt{\frac{\nu_{ct}}{\nu_c} f_c f_{ct} \left(\nu_c - 4\frac{\nu_{ct}}{\nu_c}\frac{f_{ct}}{f_c}\right)} , \tag{4}$$

where ν_{ct} and ν_c - are the so called effectiveness factors for concrete in tension and compression, respectively.

The values of factors ν_{ct} and ν_c can be obtained by using the stress-deformation curves not only with ascending but also with descending branches, i.e. with the help of intensity of released energy of tensile concrete in fracture G_1 .

A non-linearity of the post-peak curve contributes substantially to the toughness and ductility of tensile concrete. An investigation of fracture mechanism of tensile concrete gives a possibility to understand the behavior of keyed joints in shear. Moreover, it helps to conceive the indispensable conditions leading to instantaneous cracking and collapsing of the keyed joints.

A tensile stress concentration in concrete due to internal cracking is noticed. If concrete is a sufficiently tough material the stress concentration does not lead to a sudden brittle failure of a single key and to a decrease of carrying capacity of wall-to--wall joints. It may be explained by the fact that the toughness property helps to absorb the released energy of cracking concrete. Owing to it a distribution of load effects occurs in the key system.

Twenty test sets were used to investigate the main mechanical and energetical properties of tensile loaded concrete C15...C20. Both notched and un-notched tensile and bending specimens were tested to study the tension behavior of concrete (Fig. 2).

According to the recommendation [2] the failure mechanism of tensile concrete may be described by two parameters (Table 1). The critical coefficient of stress intensity

$$k_1 = \sqrt{\pi l}\; f(\varsigma) , \tag{5}$$

where $f(\varsigma)$ - function, characterizing a specimen configuration and loading technique.

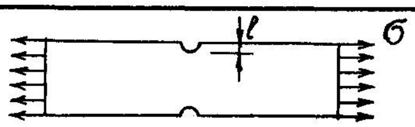
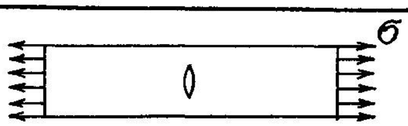
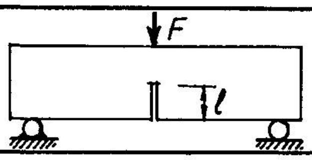
The intensity of released energy may be evaluated by the equation

$$G_1 = k_1^2 / E_c , \tag{6}$$

where k_1 - coefficient by (5); E_c - modulus of elasticity of concrete. The intensity G_1 by (6) allows to calculate the values of factors ν_{ct} , ν_c and concrete shear strength f_v by (4) more accurately.

Table 1

Parameters of fracture criterion of concrete in tension

Loading technique	Loading diagram	k_1, $MN/m^{3/2}$	G_1, N/m
Centric tension		1,02	52
Centric tension		1,19	62,8
Three point bending		1,08	51,6

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