Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 60 (1990)

Artikel: Composite semi-rigid frame analysis and erection procedures
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DOI: https://doi.org/10.5169/seals-46506

Composite Semi-Rigid Frame Analysis and Erection Procedures

Dispositifs de montage d'un portique à connecteurs souples

Verbundrahmen mit nachgiebigen Rahmenknoten bei der Montage

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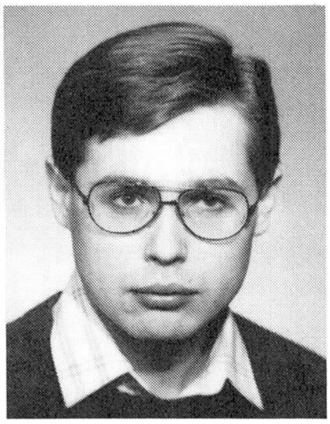

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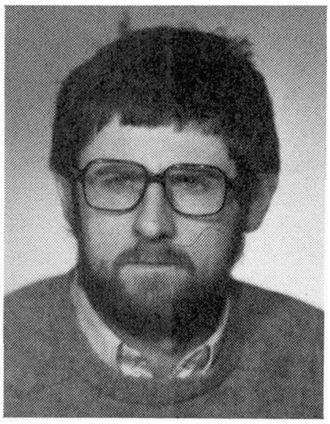

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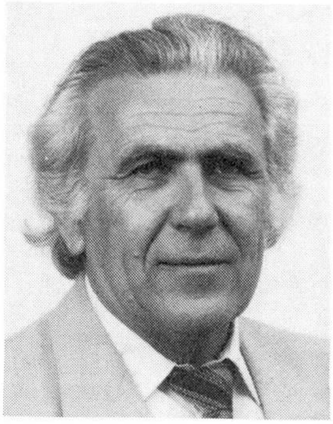

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SUMMARY
A numerical nonlinear model of composite steel-concrete frame with semi-rigid connections, slip in connectors and nonlinear material behavior has been developed. The result of a study taking account of different erection procedures is presented.

RÉSUMÉ
Pour le calcul des portiques mixtes acier-béton à connecteurs souples, on a créé un modèle numérique en admettant le glissement dans les connecteurs et le comportement non linéaire du matériau. Cette étude présente l'influence des séquences de montage sur l'état de contrainte de la construction.

ZUSAMMENFASSUNG
Im Programm für die Berechnung von Verbundrahmen wurden nachgiebige Rahmenknoten, Dübelverschiebung und das elastisch-plastische Verhalten des Materials berücksichtigt. Konkrete Beispiele des Einflusses auf den Montageablauf werden dargestellt.

1. INTRODUCTION

The widespread use of the composite steel-concrete construction in steel building frames has drawn the attention to semi-rigid composite connections which seemes to be an economically attractive solution for moments redistribution and for resistance to moderate horizontal forces. The major obstacle to the use of composite frames with semi-rigid connections is the lack of experimental verification and of analytical models both for the moment-rotation interaction in connections and for concrete-steel composite action. Joint and frame behaviour is very sensitive on erection procedures in steel-concrete composite structure.

The physical model consists of an I-shaped steel girder, concrete slab, steel reinforcement and steel shear studs. For each of these components a nonlinear material behaviour for monotonic loading is known. Therefore the method of analysis can be based on the calculation of moment-rotation behaviour of the cross-section and on the finite difference numerical integration along the span of the girder. The initial stress method is used for iteration on each load step in the frame analysis.

We use a simplified analytical spring prediction model to be compatible with the used frame in a plane design which would take full advantage of composite action in practice. A three-parameter power model is formed using the initial connection stiffness affected by slip in connectors, steel connection and contact between the concrete slab and the column and the ultimate moment carrying capacity. It is then the purpose of this paper to show erection problems when the semi-rigid steel frame connection changes from hinged model.

2. MATERIAL ASSUMPTION

Numerical model takes into account arbitrary nonlinear stress-strain interaction curve for concrete, steel and reinforcement. Steel-concrete connection is made by studs, slip is introduced in form of a function expressing the q - longitudinal shear force, u_c - slip). For practical calculation the corresponding values given in EC 4 were used.

3. COMPOSITE ACTION

Due to nonlinear behavior, mainly due to cracks in concrete in negative moment region the distribution of internal forces along a beam can be changed. This fact is taken into the account by such a computation procedure, in which the beam is divided in elements of finite length L and of the ideal stiffness EI corresponding to the curvature Φ . This curvature is taken from the interaction curve $M - \Phi$, $M - N - \Phi$ respectively. Equivalent stiffness of each element is computed in every iteration step repeatedly, since it must correspond anytime to adequate acting forces. The iteration is closed when proposed small difference of stiffness in two consequent iteration steps is reached.

3.1 Moment-curvature

Accepting the Navier´s hypothesis, the strain ε_z in the distance z from the neutral axis is

$$\varepsilon_z = \Phi \cdot Z \qquad (1)$$

where Φ is the angle of curvature.

The slip strain ε_d |5| can be introduced as

$$\varepsilon_d = \varepsilon_s - \varepsilon_c \qquad (2)$$

where ε_s is strain in top fibers of steel girder flange
ε_c is strain in bottom fibers of concrete slab.

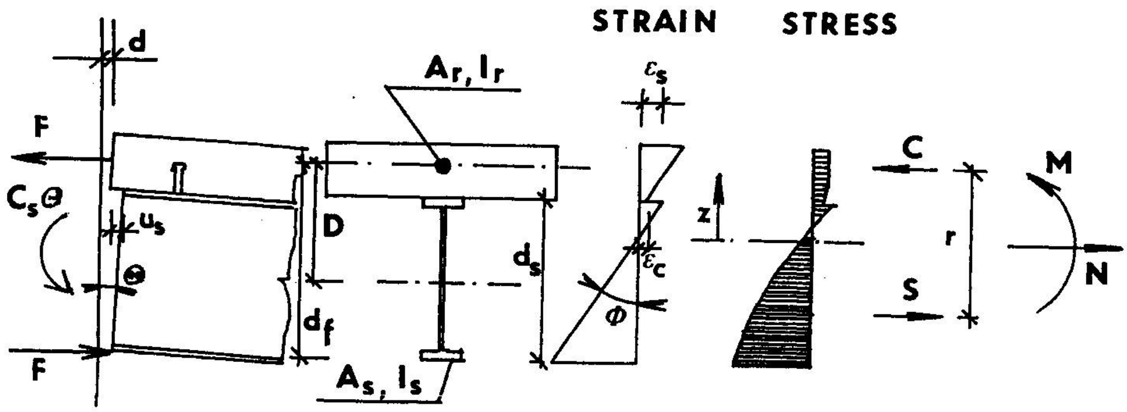


Fig. 1 The Composite Beam Element Model

Then the steel girder strain is $\varepsilon_{z,s} = \Phi \cdot Z$ (3)

and that one in concrete slab is $\varepsilon_{z,c} = \Phi \cdot Z - \varepsilon_d$ (3a)

The external load produces in concrete slab a force C, which can be calculated by integration over concrete slab area

$$C = \int \sigma_c \cdot dA_c \tag{4}$$

Similarly, the force S in steel section and R in concrete reinforcement has to be calculated.

Normal stress in all sections is

$$\sigma = f(\varepsilon) \tag{5}$$

Of course, the equilibrium condition for inter nal and external forces in the form

$$\begin{aligned} C + S + R + N &= 0 \\ C \cdot r = S \cdot r &= M \end{aligned} \tag{6}$$

must be fulfilled.

If the length L of an element is small enough, the uniform distribution of longitudinal shear force along the element can be assumed

$$q = - C/L \tag{7}$$

from $q = f(u_c)$ (8)

where u_c - slip

and from slip strain correlation $\varepsilon_d = u_c/L$ (9)

the formula (10) can be derived

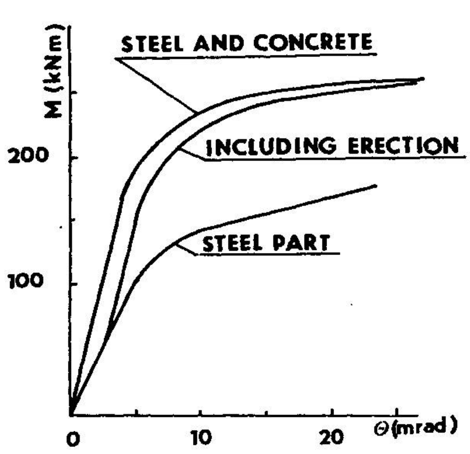
$$C = - L \cdot f(u_c) \tag{10}$$

Introducing (3), (3a) and (5) in (4), using numerical integration and iteration for searching the actual position of neutral axis (with help of eq. (6))it is possible to find out for the given value of slip u_c and that of curvature Φ the correspondent value of bending moment M. By numerical solution of eq. (10) for the known value C the unknown value of slip u_c for following iteration step can be calculated.

4. COMPOSITE JOINT BEHAVIOR

4.1 The Moment-rotation Relationship

The moment-rotation relationship of a composite joint is the end product of a complex interaction between the composite beam and the concrete slab. Using the initial connection stiffness C_i and the ultimate moment carrying capacity



M_u a three parameter power model is found to be adequate for representing the moment-rotation stiffness. Therefore we adopt this model for the presented composite connections.

The moment-rotation relationship can be represented adequately by the power model in the form

$$M = C_i . \theta (1+(\theta/(M_u/C_i))^n)^{-1/n} \tag{11}$$

in which is n the shape parameter.

Fig. 2 Moment-rotation Behavior Prediction Model of Flush End Plate Connection

4.2 Modeling of Steel Connection Behavior

Several analytical prediction models have been developed to represent steel connection flexibility. These models are generally either sophisticated numerical simulation or an approximation based on test data. We used prediction models, which are determined by using simple analytical procedures |3|, |6|.

4.3 Initial Stiffness

The initial stiffness of cracked composite connection is influenced by the concrete slab action, steel part connection and slab-column action. The interface slip substantially affects the response of the connection.

For the simple composite joint shown in Fig. 1. the equilibrium and compatibility conditions at the column face are

$$M = F \; . \; h_F + C_s \; . \; \theta \tag{12}$$

$$\theta \; . \; h_s = (d_1 + d_2).h_s/h_F + u_s \tag{13}$$

C is the secant stiffness of the steel part of connection.

For the slab-column action we divide the deflection in steel bars of the slab d_1 and in the connection d_2 . The deflection d_1 we can determine

$$d_2 = F(1+(M - M_2)/M)/C_r \tag{14}$$

for C_r as secant stiffness of steel bar connection to the column and M_2 a moment on the opposite side of the column.

The contribution of slip action is accepted from composite beam behavior on each load step.

4.4 The Ultimate Capacity of the Composite Joint

The ultimate moment capacity of the joint can be determined simply by adding up the moment capacity of the steel connection M_s to the moment of resistance given by the yield strenth of the bars |3|

$$M_u = M_s + A_r . \; f_r . \; d_f \tag{15}$$

5. THE DISCUSSION OF ERECTION PROBLEM

The frame in Fig. 3 was analyzed using the composite girder and steel columns for both noncomposite and composite design. Beam-to-column connections were constructed in form of different types of steel connections. The survey of steel connection constants is in Table 1. The ultimate limit was established by limit deformations of beam and collapse load of whole frame. The proportional limit load with real connections to the rigid ones summaries the results of parametrical study in Table 1 (column 3). The same results are for frames with flexible connections with only composite action (column 4).

Connections	1 C_i |kNmmrad-1|	2 M_u |kNm|	3 P_{ul}	4 P_{ulcom}
1. Double Web-Angle	20	20	0,45	0,70
2. Top and Seat-Angle	25	40	0,48	0,70
3. Top and Seat-Angle with Double Web-Angle	30	80	0,80	0,95
4. Header Plate	50	60	0,45	0,80
5. Flush End-Plate	90	110	0,90	0,98

Table 1

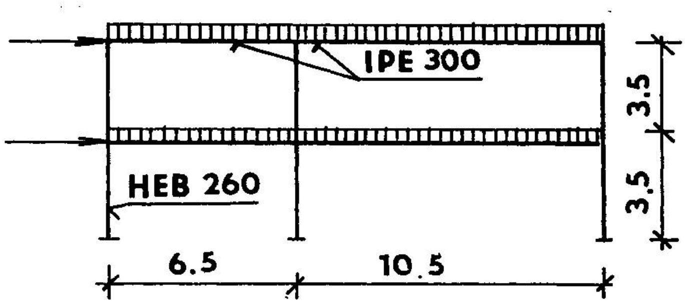


Fig. 3 Frame Configuration

6. CONCLUSIONS

1. The proposed power model is in a good agreement with the available results of cantilever tests. The power model can be implemented in a second-order analysis more easily than a piece-wise linear model.

2. Due to the longitudinal interface slip between the steel beam and concrete slab in composite members the moment-rotation characteristic does not depend solely on joint parameters and it is necessary to take into account the behavior of the whole frame. For a beam element the slip strained calculation and numerical integration along the beam element give reasonable accurate results conform ing with the slip in initial stiffness for some known cantilever tests.

3. The slab-column interaction is of great importance and affects the moment-rotation characteristics, first of all the initial stiffness, when the node is subject to asymmertical loading. Establishing the value of major parameters is the problem of the prepared tests and numerical study.

4. The limited knowledge of the behavioral mechanisms of this type of joints hampers their practical use. There is great need for experimental verification of numerical model.

5. The steel should act as simply supported under dead and constructions loads. The use of semi-rigid steel connections under erection could show reasonable advantages. The reduction of ductility is under required limits.

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