COMBINED EFFECTS OF DOWEL ACTION AND FRICTION IN BOLTED CONNECTIONS

Björn Engström
Chalmers University of Technology
Division of Concrete Structures
M.Sc., Research assistent

ABSTRACT

A series of detail tests on bolted connections between concrete elements was carried out. The aim was to study the shear-slip relationship, especially at large imposed deformations.

In the evaluation of the test results, it was found that friction effects at the joint interface often contributed to the observed capacity. In order to consider the combined effects of dowel action and friction, a theoretical approach was developed. Considering the state of deformation in the bolt at failure, it was possible to estimate the imposed elongation in the bolt and the resulting friction effect at the joint interface. There was good agreement between the calculated and observed capacities.

Key-words: Connection, bolt, precast elements, dowel action, shear-friction

1. INTRODUCTION

In 1985 a series of tests on bolted beam/column connections was carried out at Chalmers University of Technology, Division of Concrete Structures. The tests could be characterized as detail tests, as the test specimens only included the connection zones of a beam and a supporting member, respectively. However, the connections were built up in full scale using details which typically appear in prefabricated structures in practice. Two alternative load arrangements were used, see FIG. 1.

In the first alternative, a horizontal tensile force was applied on the beam element, while the support element was fixed to the laboratory floor. In this way the beam was forced to slide horizontally on the support and, as a result, the connection bolt was loaded in shear as a dowel pin. In the other alternative, the beam element was forced to rotate at the edge of the support by means of a vertical load.

After a first evaluation of measurements and observations, preliminary results from the study were presented in /1/. Among others, it was found that the yield load in the shear tests could be evaluated by means of the classical model for dowel action.

In this model the theory of plasticity is adopted, see for instance /2/. However, there was some scattter in the results, when the calculated values of the yield load were compared with the experimental ones.

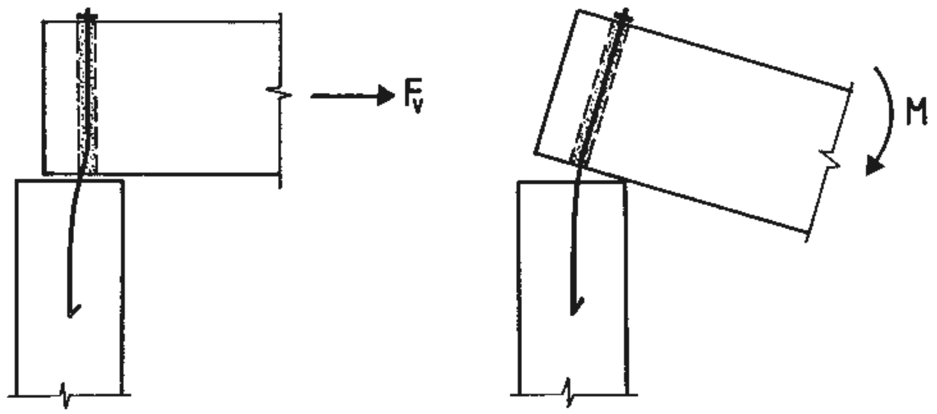

FIG. 1. Principal load arrangements used in the experimental study /3/, a) shear tests (sliding beam element), b) bending tests (rotating beam element)

A more careful evaluation of the test results was carried out in 1989-90. Then it was found that the calculated values of the yield load deviated from the experimental results in a systematic way. It was assumed that friction forces at the support joint contributed to the observed resistance of the connections.

This paper will be limited to the problem of how to estimate the yield capacity of bolted connections loaded in shear along the joint interface. It will be shown how the observed yield load from the experimental study varied, depending on the detailing, and how the classical theory for dowel action was insufficient to predict the real capacity. A modified theory, describing the mixed behaviour of dowel action and friction, will be presented and applied on the test series.

For the results from the bending tests, reference is given to the research report /3/. In the report also more extensive information about the shear tests is presented, for instance about the post-yield behaviour, splitting cracks, and the effectiveness of various arrangement of splitting reinforcement.

2. SHEAR TESTS ON BOLTED BEAM/COLUMN CONNECTIONS

2.1 Description of the tests

In prefabricated beam column systems, bolted connections have been and are still widely used. A typical solution of a beam to column connection is shown in FIG. 2 a.

The aim of the project was primarily to study the behaviour of bolted beam/column connections when exposed to large relative displacements along the joint interface.

It was of special interest to study the ductility and deformation capacity of the connections and in this respect also examine the effect of various detailing.

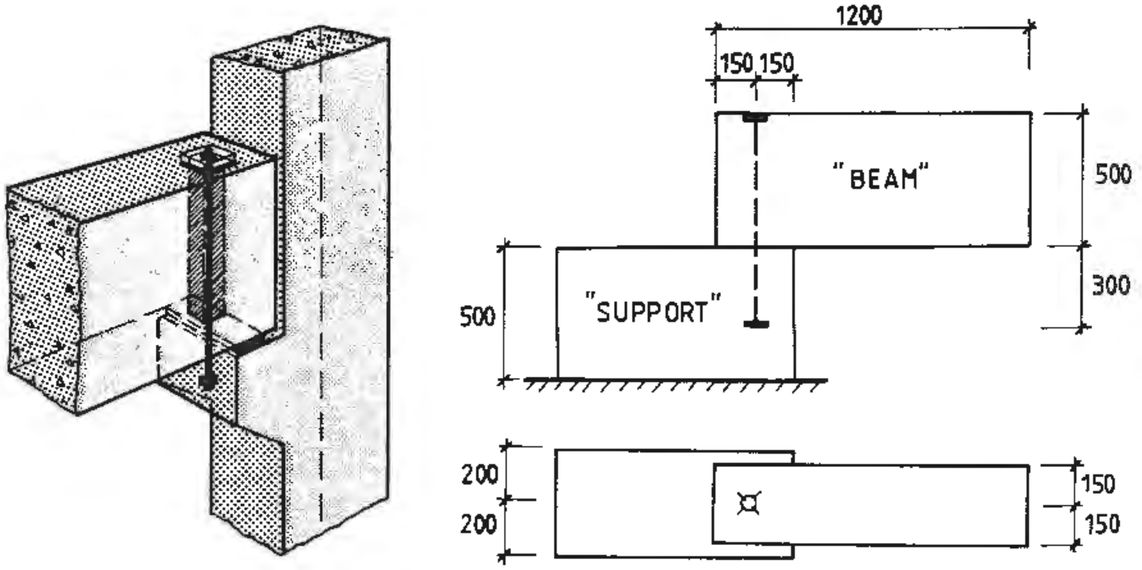

FIG. 2. Bolted beam/column connections
a) typical solution in a precast structure
b) test specimen consisting of a "support element" and a "beam element" which are connected by a connection bolt

The test specimens consisted of two concrete elements which simulated the connection zones in a prefabricated beam and a supporting structure, respectively. The elements were idealized, but the detailing and the type of concrete were in accordance with typical solutions which appear in practice. The "beam element" was provided with a vertical bolt hole. This was formed by a rectangular 2 mm plastic tube with outer dimensions 40x80 mm. The plastic tube was fixed in the mould and left in the concrete.

The connection bolt was normally cast into the "support element" and anchored by an end-anchor, consisting of a nut placed on the threaded end of the bolt. In one of the tests, as an alternative, the bolt was anchored in the support by means of a threaded insert of type "Vemo no. 1981".

At an age of 7 days the concrete elements were mounted together. The "beam element" was fixed to the "support element" by means of the connection bolt. Then a nut and a steel plate were placed on the bolt in a recess at the upper side of the "beam element" and the nut was tightened with a torque of 150 Nm.

Immediately after the mounting, the bolt hole was with one exception filled with grout. The dimensions of the test specimen and the placing of the bolt appear from FIG. 2b. The bolts, with a dimension of $\phi 24$ mm, were made from steel of type SIS 2172 according to Swedish Standards. Two types of bolts were used.

The bolts of type 1 were made from smooth bars with the ends threaded along a length of 80 mm. The bolts of type 2 were made from threaded bars. The threads were in both cases of type M24. For each type of bolts, a series of 5 standard tensile tests was carried out. The strength data are presented in TABLE 1.

TABLE 1. Strength data for the bolts (mean values)

Type	Dimension ϕ [mm]	Strength f_{sy} [MPa]	Strength f_{su} [MPa]	Limit strain $\epsilon_{s,lim}$	Remarks
1	24	476	674	0.212	threaded ends
2	"	510	653	0.177	threaded bar

The concrete in the test specimens was in all cases of grade K40 according to Swedish Standards and with an age of 15 days, when the test was carried out. At the same time as the main test, the concrete compressive strength was determined by tests on 150 mm concrete cubes, cured as the main test specimen. Three concrete cubes were taken from each concrete mix. The mean strength values are presented in TABLE 2.

TABLE 2. Strength data for the concrete and the grout fill used for the test specimens

Test No	"support element"	" beam element"			
	concrete	concrete	grout		
	f_{cc} [MPa]	f_{cc} [MPa]	type	ρ[kg/m^3]	f_{cc} [MPa]
1	56.3	53.0	A	2.17	50.7
2	53.5	60.9	-		
3	56.1	55.8	A	2.15	48.1
5	60.3	55.1	A	2.13	47.1
6	59.1	62.4	A	2.13	47.1
8	55.3	60.4	B	2.03	36.9
9	59.2	57.2	A	2.14	51.4
10	57.5	55.3	A	2.14	51.4
11	46.4	58.1	A	2.16	33.2
12	54.9	51.9	A	2.15	48.1
13	58.3	59.3	A	2.12	46.1

Two types of grout were used as fill in the bolt holes. Type A was an ordinary mix and type B was a more porous one. The compressive strength of the grout was also measured on 150 mm cubes and the mean values from series of three cubes are presented in TABLE 2.

The beam elements were provided with splitting reinforcement which was designed in order to balance the splitting effects from the bolt. Two different solutions were used.

Both the solutions were capable of preventing splitting failures until the yield load was reached. The yield capacity of the connections seemed not to be affected by the type of detailing. However, in the post-yield state, splitting failures occurred in all the tests but for considerable imposed relative displacements, in the range of 11 - 46 mm. The effect of the various arrangements is further discussed in /3/.

In the shear tests, the "beam element" was subjected to a tensile force by means of a horizontally oriented hydraulic jack. The tensile force was transferred to the "beam element" by two reinforcement bars ϕ 16mm Ks600 which were anchored in the concrete by bond along a length of 500 mm. The "support element" was fixed to the laboratory floor and, by means of the horizontal load, the bolted connection was sheared along the support joint. During the test, the horizontal load and the relative horizontal slip between the two concrete units were measured and recorded continuously. The technique of measurements and the instrumentation are more extensively described in /3/. The test programme for the shear tests are presented in TABLE 3.

TABLE 3. Test programme for the shear tests

Test No	Bolt hole	Grout type	Bolt type	Bolt anchorage at top	Bolt anchorage at bottom	Bearing
1P	grouted	A	1	nut	nut	-
2	open	-	1	nut	nut	-
3	grouted	A	1	-	nut	-
5	grouted	A	1	nut	insert	-
6	grouted	A	2	nut/bond (450mm)	bond (300mm)	-
8	grouted	B	2	nut/bond (450mm)	bond (300mm)	-
9	grouted	A	1	-	nut	-
10	grouted	A	1	nut	nut	-
11	grouted	A	1	nut	nut	steel (10mm)
12R	grouted	A	1	nut	nut	-
13	grouted	A	1	nut	nut	rubber (10mm)

P = Pilot test
R = Reference test

2.2 Test results

All the shear tests had the same general behaviour. A typical relationship between applied tensile force (shear force) and the relative displacement at the support joint is presented in FIG.3.

In all the tests where the bolt holes were filled with grout, the bolted connections had initially a stiff elastic behaviour. The elastic phase was ended by a marked change to a more plastic behaviour.

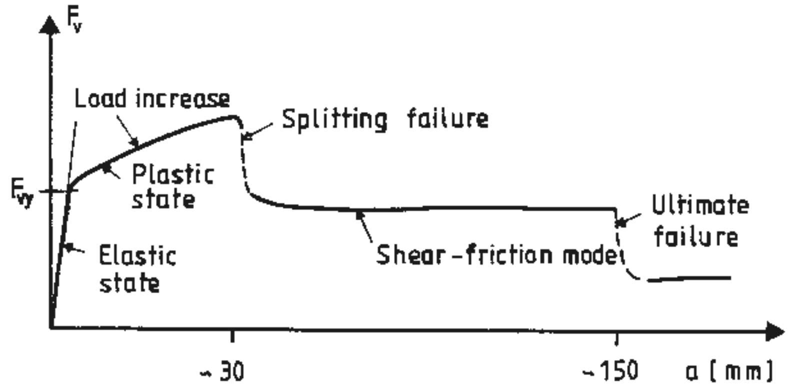

FIG. 3. Typical relationship between tensile force and horizontal relative displacement from the shear tests

The yield load is here denoted F_{vy} and the corresponding value of the relative displacement a_{vy}. The yield load was evaluated from the recorded load-displacement relationships as the point where the curve deviated from a straight line, as indicated in FIG. 3. It was always possible to note a slight but marked decrease of stiffness when the shear force was about 80 - 90 % of the yield load. This load level will in the following be denoted $F_{vy,min}$ and the corresponding relative displacement $a_{vy,min}$.

When the bolts were anchored by a nut in the top of the beam, it was always possible to increase the load in the plastic state. The load then increased constantly with increased imposed displacement at the joint interface. It was normally possible to increase the load with about 30 % of the yield load.

TABLE 4. Test results from the shear tests

Test No	$F_{vy,min}$ [kN]	$a_{vy,min}$ [mm]	F_{vy} [kN]	a_{vy} [mm]	F_{max} [kN]	a_{split} [mm]	F_{vr} [kN]
1	114	1.7	120	3.5	150	18	43- 74
2	88	28.5	102	30.5	176	101	120-136
3	92	4.2	95	5.0	96	11	32- 70
5	90	2.5	98	4.0	128	17	76-147
6	100	2.5	108	3.5	140	31	108-162
8	91	2.7	100	4.2	164	40	106-162
9	89	1.5	101	3.5	124	120	100-123
10	105	2.7	121	5.5	172	46	94-174
11	78	2.7	99	8.0	131	27	94-141
12	100	3.0	110	4.0	146	28	82-115
13	81	3.0	90	5.0	135	33	65-107

In the plastic state, considerable cracking occurred in the beam as well as in the support element. The maximum load F_{max} was always determined by a brittle splitting failure. When splitting failures occurred, the corresponding relative displacement a_{split} was normally about 30 mm.

However, as the bolt was anchored on both sides of the support joint, it was still possible to transfer forces in the connection by shear friction effect at the support joint. It was possible to reach ultimate displacements of about 120 - 200 mm. During this residual state, a considerable shear transfer capacity was still remaining. However, the shear force F_{vr} transferred in the residual state varied with increased displacement. The range of variation of F_{vr} is presented in TABLE 4 where all the important test results are summerized.

The values of the relative displacement $a_{vy,min}$ and a_{vy} are very high in test No 2 compared with the other tests. In this test the bolt hole was left open and the measured displacements therefore include also the clearance in the bolt hole.

In the tests No 3 and 9, the bolt was not anchored in the top of the beam. As a result, there was no or only a slight increase of the load in the plastic state, compare F_{max} with F_{vy}. In the tests No 2 and 8, this increase is more pronounced. Here, the bolt hole was either left open or filled with the porous grout of type B. Appearently, stress concentrations were prevented in those two cases.

3. APPLICATION OF THE CLASSICAL THEORY OF DOWEL ACTION

3.1 The classical theory of dowel action

When bolts are used as dowels, the shear capacity can be estimated by means of the theory of plasticity. The model is illustrated in FIG. 4, for a case when a steel dowel is embedded in concrete at one end and loaded by a transverse load acting along the face of the concrete body.

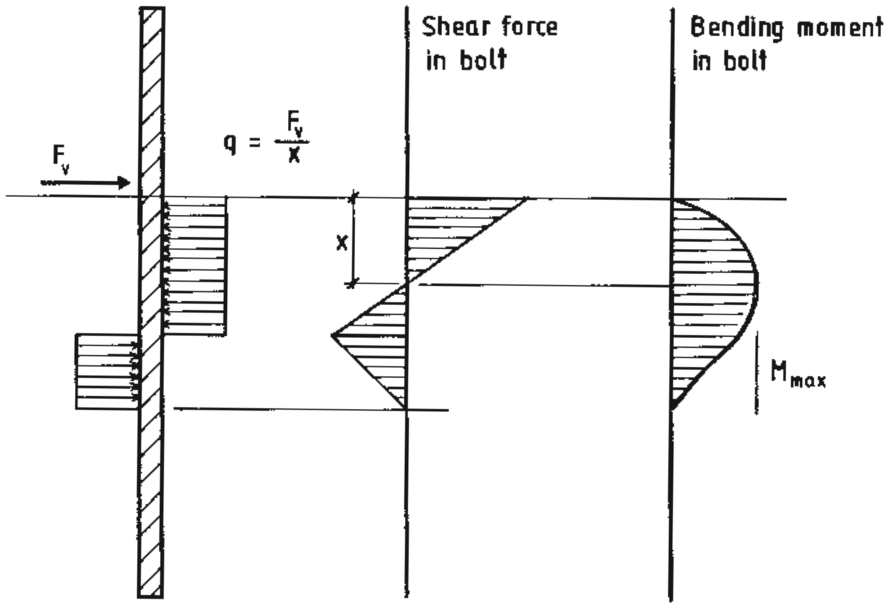

FIG.4. Model for calculation of the shear force capacity of a steel dowel embedded in concrete at one end

This model was presented by B Höjlund-Rasmussen /2/ who used the theory for evaluation of his own tests on steel bars and bolts partly embedded in concrete cubes. The model was improved by H Dulácsca /4/ who treated the case when reinforcement bars crossed a joint at an angle.

The steel dowel is fixed in the concrete which gives a distributed reaction q along the bolt. Locally, the concrete is assumed to get plastic deformations. In the model, the reaction q from the concrete is assumed to be uniformly distributed and proportional to the product ϕf_{cc}, where ϕ = diameter of the dowel and f_{cc} = the compressive strength of the concrete. An equilibrium condition for the state when the bar is fully plastified in the section for the maximum bending moment gives the shear capacity

$$F_{vy} = c_1 \phi^2 \sqrt{f_{cc} f_{st}} \qquad (1)$$

where $c_1 = \sqrt{k/3}$
k = constant of proportionality ($q = k\phi f_{cc}$)
f_{st} = steel strength

By means of a large amount of tests on steel dowels embedded at one end, Höjlund-Rasmussen found that the constant in (1) should have the value $c_1 = 1.29$. The concrete strength was then determined on cylinders ϕ150 mm with the height 300 mm. The diameter of the dowels varied between 16 - 26 mm and the concrete compressive strength between 10 - 45 MPa.

The ratio between cylinder strength and cube strength varies between 0.80 - 0.97 for cube strength values in the interval 20 - 60 Mpa. This means that the constant c_1 should have values between 1.15 - 1.25 in (1) when the concrete strength is determined by tests on cubes. For the concrete strength in the present test series, an appropriate value of the constant is $c_1 = 1.23$.

In the present study on bolted beam/column connections, the bolts were embedded at both ends on each side of the support joint. This case has been treated theoretically as well as experimentally by H Dulácsca /4/. In the tests by Dulácsca, the joint faces were treated in order to eliminate the friction effects. The concrete compressive strength was determined by tests on 150 mm cubes and was in the range of 10 - 35 MPa.

Using the formula proposed in /4/, for the special case that the steel dowel is oriented perpendicular to the joint, the following expression is received

$$F_{vy} = 1.15 \phi^2 \sqrt{f_{cc} f_{st}}$$

This expression is identical with (1), in the case when the constant c_1 is tranformed to account for cube values of f_{cc} in the order about 20 MPa.

Thus, the model in FIG. 4 is applicable also for dowels which are embedded in concrete at both sides of the loaded section. The condition of equilibrium at one end of the dowel will determine the shear capacity also if both the ends are embedded in concrete.

In the present test specimens, the concrete strength was normally not the same in the beam element as in the support element. When (1) is used for estimation of the capacity, it is assumed that the concrete around the bolt is confined and that plastic deformations can be reached locally in the concrete. For the bolt which is embedded by grout in the bolt hole, it is the strength of the grout which represents the local resistance of the surrounding concrete.

The formula (1) gives an estimate of the shear force which produces plastification in the section of maximum bending moment in the bolt. For a bolt which is embedded at both ends, the yield load F_{vy} is reached when a mechanism is formed. This will be the case when plastic hinges are produced in the bolt on both sides of the loaded section (the joint). When the strength of the concrete or the grout is not the same at both ends of the bolt, the two plastic hinges will be developed for different values of the shear force. In the element with the weaker concrete, a plastic hinge will be formed first. When, for an increased shear force, a plastic hinge is formed also at the other end, a mechanism is formed. This mechanism determines the capacity (yield load) of the bolted connection.

Therefore, for estimation of the capacity of the connection, it is the higher strength value of the two which should be introduced in (1).

In normal applications, the effects of friction in the joint section can not always be neglected. The contribution by friction to the shear capacity must be proportional to the normal force in the bolt or, as it is expressed in /2/

$$\Delta F_v = c_f \phi^2 \sigma_s \qquad (2)$$

As this contribution to the capacity is proportional to ϕ^2, the total capacity can still be estimated by means of (1). The value of c_1 must than be increased, in order to account for the friction effects.

In some applications, the steel dowel may be prevented from free rotation at the concrete face, for instance by a threaded insert. Then the original model in FIG. 4 must be modified, see FIG. 5a. Bending moments with different signs will appear in the bolt. A mechanism is formed when plastic hinges appear both at the restrained section and at a section at some distance away. An equilibrium condition for the mechanism gives the yield load

$$F_{vy} = c_r \cdot c_1 \ \phi^2 \ \sqrt{f_{cc} \ f_{st}} \qquad (3)$$

where c_r = restraint factor

In this case, friction effects can not be avoided. The friction effects can be considered by an increased value of c_1 according to the principle in (2). Under the assumption of full fixation at the concrete face, the restraint factor has the value $c_r =$ $= \sqrt{2}$. In case of partially fixation, the section for the maximum bending moment will be moved and values between 1 and $\sqrt{2}$ can be expected depending on the degree of fixation.

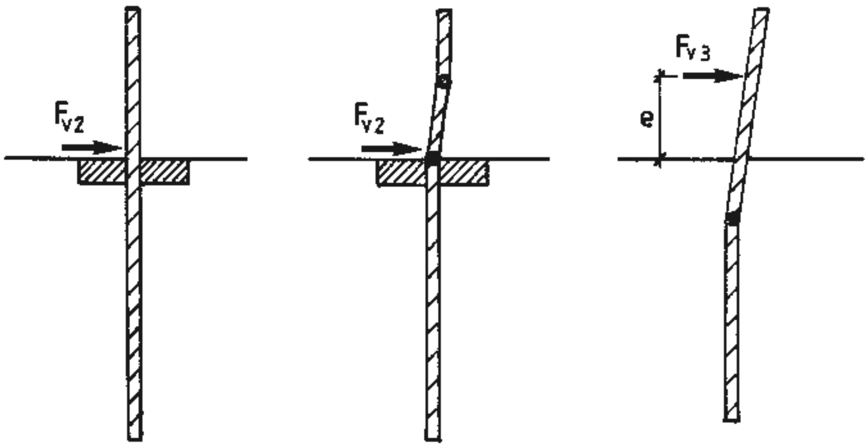

FIG.5. Modified models for dowel action in case of
a) restraint at the loaded section
b) eccentric loading

For a steel dowel which is loaded by a transverse load applied at some distance e from the concrete face, the bending moment will increase in the bolt. As a result, the shear capacity by dowel action will decrease. The original model must be modified according to FIG. 5b. The yield load is than received as

$$F_{vy} = c_e \cdot c_1 \ \phi^2 \sqrt{f_{cc} \ f_{st}} \tag{4}$$

where $c_e = \sqrt{1+(\epsilon c_1)^2} - \epsilon c_1$ (5)
(eccentricity factor)

$$\epsilon = \frac{3e}{\phi} \sqrt{f_{cc}/f_{st}}$$

Eccentric loading will for instance be the case, when there is a joint gap between the two connected concrete elements. Then the eccentricity e should be taken equal to half the joint gap, when the eccentricity factor (5) is calculated.

3.2 Comparison with test results

The classical theory for dowel action, as presented in the previous section, was applied on the tested bolted connections. In most of the tests, the bolts were embedded on both sides of the loaded section.

It was than assumed that the yield load could be calculated by (1), if the higher strength value of the two embedding materials was introduced in (1). This strength value will be denoted $f_{cc,max}$ in the following.

When the bolt hole was open (test No 2), the bolt was considered as embedded at one end only and with the other end free. Then, the concrete strength of the support element was introduced in (1).

In test No 5, the bolt was anchored to the support by means of a threaded insert. The restraint which followed this solution was then considered by a restraint factor c_r according to (3). Two alternative values of c_r was used in the calculations, either $c_r = \sqrt{2}$, assuming full fixation, or $c_r = (1 + \sqrt{2})/2$ assuming partial fixation. When the test specimen was examined after the test, it was found that the insert had got a small rotation, which means that the bolt was not fully fixed. The plastic hinges appeared in the bolt within the threaded end zone. Therefore, the net diameter ϕ_{net} = 21 mm was used in the calculations in this case.

In tests No 11 and 13, the support joint was provided with a 10 mm bearing of steel or neoprene with dimensions 100x300 mm. The bearings were provided with a hole ϕ40 mm for the bolt.

When the bolt hole in the beam was grouted, also the hole in the bearing was filled with grout. Thus, the bolt was embedded with grout or concrete along the full length. In the case with the steel bearing, the grout was effectively confined by the bearing. It was probably possible to reach plastification in the concrete also at the level of the joint gap.

In the case with the neoprene bearing, it is more doubtful whether the grout was effectively confined at the level of the joint gap. Therefore, an eccentricity factor should be introduced in the calculation. Here, two alternative calculations were made for both the tests, with or without an eccentricity factor c_e according to (5). The eccentricity e was then taken equal to half the thickness of the bearing.

For threaded bars, the net diameter ϕ_{net} = 21 mm was used in the calculations.

Using the measured value of the yield load F_{vy}, a value of the constant c_1 was evaluated, see (1)

$$c_1 = \frac{F_{vy}}{\phi^2 \sqrt{f_{cc,max} \; f_{st}}} \tag{6}$$

When topical, the yield load in (6) was also divided by the correction factors c_r or c_e according to (3) and (5). The results of the calculations are presented in TABLE 5, where the tests are arranged in order of restraint.

TABLE 5. Values of the coefficient c_1 estimated from the test results according to (6)

Test No	Bolt hole	Bolt type	Bolt anchorage at top	at bottom	c_r	c_e	c_1
3	grouted	1	-	nut	1	1	1.01
9	"	1	-	"	1	1	1.04
2	open	1	nut	nut	1	1	1.11
1	grouted	1	nut	nut	1	1	1.27
5	"	1	"	insert	(1.41)	1	(1.05)
					1.21	1	1.22
10	"	1	"	nut	1	1	1.27
11	"	1	"	"	1	(0.82)	(1.41)
						1	1.16
12	"	1	"	"	1	1	1.18
13	"	1	"	"	1	0.80	1.17
					1	(1)	(0.94)
6	grouted	2	nut/bond	bond	1	1	1.41
8	"	2	"	"	1	1	1.35

For some of the tests, the calculations were carried out with alternative values of the correction factors. The most unlikely result is put within brackets in TABLE 5. As appears from the table, the remaining alternative always gave results which is in good agreement with the other results in the group.

In TABLE 5, it is possible to distinguish four groups in order of restraint. In the first group, the bolt was not anchored in the top of the beam. When the connection was loaded in shear, the bolt could slip in the bolt hole. The effect of friction was probably negligible in those cases. The mean value of the coefficient c_1 was 1.03 in the first group. This can be considered as the "basic value" with no effect of friction included.

From the tests in /2/, the corresponding value was evaluated to c_1 = 1.23. Relationships between shear force and displacement were not presented in /2/. It was probably the maximum value of the load which was taken as failure load, and not the yield load as in the present study.

Values of the bolt displacement at failure are reported in /2/ and are in the same order as the dimension of the dowel. That is much more than was measured in this test series when the yield load was reached. Therefore, in the test reported in /2/, it is likely that the strain hardening in the steel has contributed to the higher values of the capacity. With the big shear displacements at failure, it is probable that the maximum load was determined by cracking of the concrete element and not by the steel strength.

In the second group, only the test with the open bolt hole is placed. Here the bolt was anchored in the top and could not slip freely in the bolt hole. An imposed displacement along the support must have resulted in tensile strain in the bolt. However, due to the clearance in the bolt hole, the system had a considerable flexibility.

Most of the tests are placed in the third group. Here the bolt hole was filled and the bolt was anchored by end anchors at both ends. The mean value of the coefficient c_1 in this group was $c_1 = 1.21$. This is about 18 % higher value than the basic value $c_1 = 1.03$ in case of no friction effect. In the third group the bolts were always made of smooth bars and local slip along the bolt was possible.

In the forth group, threaded bars were used. Now also local slip was resisted by the bond capacity of the bar. As a result, the friction effect increased. The mean value of the coefficient c_1 was $c_1 = 1.38$ in this group.

The values of c_1, which was evaluated in TABLE 5 from the measured values of the yield load, will now be used to estimate the load $F_{vy,min}$ for which the stiffness decreased. It is then assumed that the decrease of stiffness will occur, when the first plastic hinge is formed in the bolt. The corresponding load can be calculated as

$$F_{vy,min} = c_1 \, \phi^2 \sqrt{f_{cc,min} \, f_{st}} \qquad (7)$$

The results of the calculations are summerized in TABLE 6. The calculated values are compared with the measured values for the observed decrease of stiffness.

TABLE 6. Comparison between calculated values of $F_{vy,min}$ according to (7) and observed values of the shear force for which a decrese of the stiffness was observed

Test no.	c_1	$F_{vy,min}$ calculated [kN]	$F_{vy,min}$ observed [kN]	$\frac{F_{vy,min} \text{ (calc.)}}{F_{vy,min} \text{ (obs.)}}$
1	1.27	114	114	1.00
3	1.01	88	92	0.96
6	1.41	96	100	0.96
8	1.35	82	91	0.90
9	1.04	94	89	1.06
10	1.27	114	105	1.09
11	1.16	84	78	1.08
12	1.18	103	100	1.03
13	0.94*	80	81	0.99
			mean value	1.02

* $c_e \cdot c_1 = 0.80 \cdot 1.17 = 0.94$

From TABLE 6, it appears that the proposed expression (7) gave rather good estimates of the load $F_{vy,min}$ for which the decrease of stiffness was observed.

3.3 Conclusions

It is obvious that the friction at the support has contributed to the resistance of the connections. The effect of friction has varied depending on the actual detailing.

It is possible to use the classical theory for dowel action to estimate the capacity of bolted connections, eqs. (1), (3) and (4). However, the coefficient c_1 in the formulas must be adjusted in order to include the friction effects. The actual values of the coefficient can be determined by tests.

It appears from the present test series, that the detailing will affect the friction effect to a considerable degree. Therefore, it is desirable to have a more general model for the analysis of bolted connections. A model, which can be used to estimate the combined effects of dowel action and friction of bolted or stud connections, will be proposed in the next chapter.

4. MODIFIED THEORY TAKING INTO ACCOUNT THE FRICTION EFFECTS

4.1 Evaluation of the friction effects in the test series

In the cases when friction contributed to the capacity of the bolted connections, it is now assumed that the observed yield load is composed of one part from dowel action and one part from friction. The friction depends on a normal force N_s which is received in the bolt. The corresponding normal stress σ_{sm} in the bolt depends partly on the prestress σ_p which is a result of the tightening of the top nut, partly on imposed elongations which follow the relative displacement along the support joint. The normal stress σ_{sm} can be calculated as

$$\sigma_{sm} = \sigma_p + \sigma_{se} \tag{8}$$

where $\sigma_{se} = \Delta\epsilon_s \cdot E_s$

As a result, part of the tensile capacity of the bolt is used to balance the normal force. Hence, only the remaining capacity $f_{st,red}$ can be used for dowel action.

The total shear capacity of the bolted connection can be calculated as

$$F_{v,tot} = c_1\ \phi^2\ \sqrt{f_{cc,max} \cdot f_{st,red}} + \mu\sigma_{sm}A_s \tag{9}$$

where $f_{st,red} = f_{st} - \sigma_{sm}$

μ = frition coefficient

Restraint and eccentric loading should be considered by correction factors c_r and c_e as presented in the previous sections.

The expression (9) can now be used to estimate the contribution by friction in the tests. The total capacity $F_{v,tot}$ is put equal to the observed yield load F_{vy} and the imposed steel stress σ_{se} in the friction term is replaced by (f_{st} - $f_{st,red}$). Then the reduced steel strength $f_{st,red}$ can be solved from (9).

In /5/ low and high values for the friction coefficient between various materials are given. Here the friction coefficient is estimated by the average of those low and high values. The friction coefficients used in the calculations are μ = 0.6 for slip between concrete to concrete, μ = 0.4 for concrete to steel, and μ = 0.3 - 0.4 for concrete to rubber. Using (9), the coefficient c_1 should not include any friction effects. Here the assumed "basic value" c_1 = 1.03, evaluated from the present test series, is used. The results from the calculations are summerized in TABLE 7. Also in this table, the tests are arranged in order of restraint.

As appears in TABLE 7, the contribution by friction to the observed yield load was considerable in the usual cases when the bolt was anchored at both ends and the bolt hole was grouted. The friction effect increased with increased restraint of the bolt, as could be expected.

For test No 13, with a neoprene bearing in the joint, no solution was obtained when the value μ = 0.3 was assumed for the friction coefficient. The assumption of μ = 0.4 resulted in a reasonable result.

TABLE 7 The shear friction component in relation to the observed yield load. Estimation by (9) with c_1 = 1.03

Test no.	Bolt hole	Bolt type	Anchor at top	μ	$f_{st,red}$ [MPa]	$\mu\sigma_s A_s$ [kN]	$\frac{\mu\sigma_s A_s}{F_{vy}}$
2	open	1	nut	0.6	433	12	0.12
1	grouted	1	"	0.6	333	39	0.32
5	"	1	"	"	348	27	0.27
10	"	1	"	"	332	39	0.32
12	"	1	"	"	391	23	0.21
11*	"	1	"	0.4	342	24	0.24
13*	"	1	"	0.3	-	-	-
				0.4	327	27	0.30
6	grouted	2	bond/nut	0.6	262	52	0.48
8	"	2	"	"	319	40	0.40

* with bearing in the support joint

4.2 Introduction of a deformation criterion

For a certain value of the mean normal stress σ_{sm} in the bolt, the shear capacity of the bolted connection can be calculated by (9), assuming an appropriate friction coefficient for the joint interface. The steel stress σ_{sm} refers to the normal stress in the bolt in the moment when a plastic hinge mechanism is fully developed. It is now necessary to introduce a deformation criterion which can be used to determine the state of deformation in the bolt when the mechanism is formed.

It is assumed that the plastic hinge mechanism is formed when the bolt has reached a critical angular deformation α_{crit} expressed as

$$\alpha_{crit} = k \frac{f_{st,red}}{\phi \cdot E_s} \tag{10}$$

The constant k can be determined by means of measurements from the tests. As a simplification, the angular deformation of the bolt is assumed to be concentrated to the sections of maximum moment as illustrated in FIG. 6.

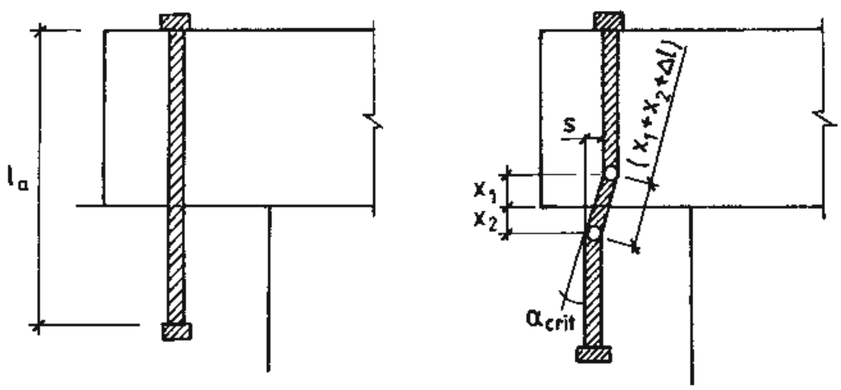

FIG. 6. Simplified state of deformation for a bolt embedded on both sides of the loaded section

The distance x from the concrete face to the section for the maximum bending moment in the bolt can be calculated by means of an equilibrium condition. Only that part of the total yield load $F_{v,tot}$ which is resisted by dowel action should be introduced in this condition according to

$$x = \frac{F_{v,red}}{q} = \frac{\sqrt{f_{st,red}/f_{cc}}}{3\ c_1} \phi \tag{11}$$

where q = the equally distributed reaction in the concrete

$q = 3(c_1)^2\ f_{cc}\ \phi$, see (1)

When the bolt is restrained or loaded by an eccentric load, the value of x should be adjusted by multiplying it with the correction coefficients c_r or c_e, see (3) and (5). For bolts which are embedded on both sides of the loaded section, the distance x should be calculated separately for each side of the joint, using the actual values of the concrete strength. The critical angle α_{crit} can now be calculated for each shear test in the test series using the observed value of the relative displacement a_{vy}, when the yield load was reached, see TABLE 4.

$$\alpha_{crit} = \frac{a_{vy}}{l_p} \quad \text{(for bolts embedded at both ends)}$$

$$\alpha_{crit} = \frac{a_{vy}}{x} \quad \text{(for bolts embedded at one end)}$$

where $l_p = x_1 + x_2$ for connections without joint gap
$l_p = x_1 + t + x_2$ for connections with a joint gap t
x = distance between the concrete face to the section of the maximum moment according to (11)

The estimated values of the reduced steel strength $f_{st,red}$ in TABLE 7 are then introduced in the calculations of of x and l_p.

The calculated distance x, between the concrete face and the section of the maximum bending moment, varied in the test series within a rather narrow interval, from $x = 0.81\phi$ to $x = 1.03\phi$. The mean value for the whole test series was $x_m = 0{,}88\phi$. For the measurement of the relative displacement a_{vy} at the yield load, the bolts with both the ends embedded seemed to be most reliable. For the bolts of threaded bars $\phi 21$ mm the critical angle was evaluated to $\alpha_{crit} = 0.116$ rad (6.65°) and for the bolts of smooth bars $\phi 24$ mm to $\alpha_{crit} = 0{,}118$ rad (6.76°) (average values). Those values are now used to determine the constant k in the deformation criterion (10). The average value of $f_{st,red}$ for each of the two groups is then used in (10). The calculations give

for bolts $\phi 24$ $\quad k = 0.118 \cdot 0.024 \cdot 210 \cdot 10^9 / 345 \cdot 10^6 = 1.72$ m

for bolts $\phi 21$ $\quad k = 0.116 \cdot 0.021 \cdot 210 \cdot 10^9 / 291 \cdot 10^6 = 1.76$ m

There is a good agreement between the two evaluated values of k. In spite of the low number of tests and the limited variation of the bolt diameter, it is now proposed that the deformation criterion for the formation of a plastic hinge mechanism can be expressed by (10) with k = 1.75 m.

4.3 Calculation of the normal stress in the bolt

For certain values of α_{crit} and x, the imposed elongation of the bolt can be calculated by means of FIG. 6. For bolts which are embedded at both ends, the imposed elongation Δl is determined as

$$\Delta l = \sqrt{(l_p)^2 + (\alpha_{crit}\ l_p)^2} - l_p \qquad (12)$$

For bolts embedded at one end only, the imposed elongation can be calculated as follows

$$\Delta l = \sqrt{x^2 + (x\alpha_{crit})^2} - x \qquad (13)$$

For the bolts of smooth bars with end anchors, it is now assumed that the imposed elongation is equally distributed along the bolt length. The increase of steel strain $\Delta\epsilon_s$ due to the imposed elongation will then be

$$\Delta\epsilon_s = \Delta l / l_a \qquad (14)$$

where l_a = distance between end anchors

When the mean normal stress σ_{sm} in the bolt is calculated, also possible prestress σ_p is considered according to (8). For bolts of threaded or ribbed bars anchored by bond, the imposed steel stress σ_{se} can be evaluated by means of a known relationship between steel stress and end-slip for the actual bar. When the imposed elongation is determined by (12), it corresponds to double endslip. When (13) is applicable, the imposed elongation equals the end slip.

4.4 Total shear capacity due to dowel action and friction

The total shear capacity of bolted connections can be calculated by an iterative procedure including the following steps. The method is applicable on connections with steel dowels from smooth bars which are anchored at both ends. When the steel dowel are made from ribbed or threaded bars, the imposed steel stress σ_{se} must be determined by means of a known relationship between steel stress and end-slip.

1) assume a value of $f_{st,red}$
2) calculate the distance x between the concrete face and the section for the maximum moment according to (11). In case of restraint or eccentric load, correction factors c_r or c_e according to (3) and (5) should be introduced in (11)
3) calculate critical angle α_{crit} according to (10) with $k = 1.75$ m
4) calculate the imposed elongation Δl according to (12) or (13)
5) calculate the increase in steel strain $\Delta\epsilon_s$ and the normal steel stress σ_{sm} according to (14) and (8)
6) calculate the reduced steel strength $f_{st,red}$ according (9). If the calculated value equals the assumed value go to step 7), otherwise go back to 1)
7) calculate the total shear capacity $F_{v,tot}$ according to (9) with $c_1 = 1.03$

The test specimens in the present study were analysed with the method above. The results from the calculations and comparisons with the test results are presented in TABLE 8. Considering the torque of 150 Nm which was used for tightening the nut at the top of the bolt, the prestress in the bolt was estimated to σ_p = 20 MPa. For tests No 3 and 9, where the bolt was not anchored in the beam, it was assumed that no contribution by friction was possible. For the tests with bolts from threaded bars, it was not possible to calculate the capacity as the relationship between steel stress and end-slip was not known.

As appears in TABLE 8, the proposed method gave good estimates of the observed yield loads. However, the adopted values of the friction coefficient are very uncertain. Futhermore, the deformation criterion according to (10) with k = 1.75 m has been evaluated from a rather small series of test where the bolts mainly had the dimension ϕ = 24 mm. It is uncertain whether it is possible to apply the same criterion also on bolts of other dimensions.

TABLE 8. Comparison between calculated values of the shear capacity according to (9) and the observed yield load F_{vy} for the tests with smooth bars

Test No	c_r	c_e	μ	$\mu\sigma_s A_s$ [kN]	$F_{v,tot}$ [kN]	$\frac{\mu\sigma_s A_s}{F_{v,tot}}$	F_{vy} [kN]	$\frac{F_{v,tot}}{F_{vy}}$
1	1	1	0.6	30.9	116	0.27	120	0.97
2	1	1	0.6	22.0	108	0.20	102	1.06
3	1	1			97	0	95	1.02
5	1.21		0.6	26.2	98	0.27	98	1.00
9	1	1			100	0	101	0.99
10	1	1	0.6	30.6	116	0.26	121	0.96
11	1	1	0.4	22.2	98	0.23	99	0.99
12	1	1	0.6	31.5	115	0.27	110	1.05
13	1	0.77	0.4	21.2	88	0.24	90	0.97
							mean value:	1.00

5. CONCLUSIONS

In the present tests on bolted connections, a marked yield point could always be recognized in the load-displacement relationsship. The maximum load was always determined by splitting failure after considerable plastic deformations. It was assumed that the practical capacity of bolted connections should be determined by the yield load.

In the cases when the bolt was anchored on both sides of the loaded section, it was found that friction effects in the joint contributed in a considerable way to the total shear capacity. This effect is not appropriately considered in the classical theory for dowel action.

Therefore, the model was improved to account for the combined effects of dowel action and friction, see (9). In order to determine the bolt deformation related to the formation of a plastic hinge mechanism, a deformation criterion (12) was suggested. With the state of deformation given, it was then possible to separate the effects from dowel action and friction.

In spite of uncertainties and simplifications in the modelling, the real capacities of the tests could be predicted with great accuracy by the proposed theoretical method. The roughness of the joint interface may lead to transverse joint separation, when the bolted connection is loaded in shear. This effect may produce additional normal stresses in the bolts, but was not considered in the present analysis. However, the theoretical approach can easily be modified to include this effect.

The proposed method can be adapted also on other types of connections where steel dowels are used, for instance stud connections in composite structures.

For bolts anchored by bond, the imposed elongation of the bolt will result in higher steel strain. It may even be the case, that the bolt reaches the yield point before the plastic hinge mechanism is fully developed. In that case, the behaviour can be described as a pure shear-friction mode. For most of the tests, there was a combined effect of dowel action and friction, but the pure modes of dowel action and shear-friction can be regarded as extreme behaviours.

6. REFERENCES

/1/ Engström, B: Bolted beam-column connections for precast structures, VTT Symposium 62 "Connections between precast concrete elements", Statens Tekniska Forskningscentral, Espoo 1985, pp 71-87

/2/ Höjlund-Rasmussen, B: Betonindstöbte, tvaerbelastede boltes og dornes baereevne (The bearing capacity of transversally loaded bolts and studs embedded in concrete), Bygningsstatiske Meddelelser 34, 1963.

/3/ Engström, B: Bultade anslutningar i betongelementstommar - verkningssätt vid stora påtvingade deformationer (Bolted connections in precast structures - behaviour at large imposed deformations), Chalmers University of Technology, Division of Concrete Structures, Report 90:1, Göteborg 1990.

/4/ Dulácsca, H: Dowel Action of Reinforcement Crossing Cracks in Concrete, ACI Journal, December 1972.

/5/ Bestämmelser för betongkonstruktioner (Regulations for concrete structures), BBK 79 utgåva 2, Band 1, Konstruktion, Statens betongkommitté, AB Svensk Byggtjänst, Stockholm 1988