Shear transfer in bolted precast concrete column connections

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- The current design method for shear transfer through bolted precast concrete column connections is based on the standards for steel structures.
- The method adjusted for design in the ultimate limit states may not be suitable for assessing performance in the serviceability limit states, where the maximum shear force is limited either based on unallowed deformations or displacement.
- This paper assesses the suitability of the current design method for evaluating the maximum shear force in the serviceability limit states and improvements for design are proposed.

Bolted connections are a practical solution for connecting the precast concrete structures. They require neither special skills nor special tools on site and reduce the need for temporary supports. **Figure 1** shows a detail for a typical bolted precast concrete column connection at the installation stage. As indicated in the figure, the precast concrete column includes column shoe inserts. Anchor bolts connecting the column are anchored to a foundation structure. The column rests on the lower nuts of the anchor bolts. By adjusting the lower nuts on site, one can adjust the vertical position and plumb of the column and then create a connection by tightening the upper nuts according to the instructions of the anchor bolt manufacturer. After assembly, the gap must be filled between the column and foundation with cement-based grout to secure the load transfer through the joint. This technique is not novel—a similar detail is provided in the *PCI Design Handbook: Precast and Prestressed Concrete*,[1] where a column is connected to a base structure through a bolted base plate. The main difference between the column-shoe and base-plate connections is the size of the bolt holes. In column shoes, oversized bolt holes are typically used to provide a generous amount of installation tolerances, which means that there may not be direct bearing on the bolts.

Usually, column connections transfer simultaneous bending

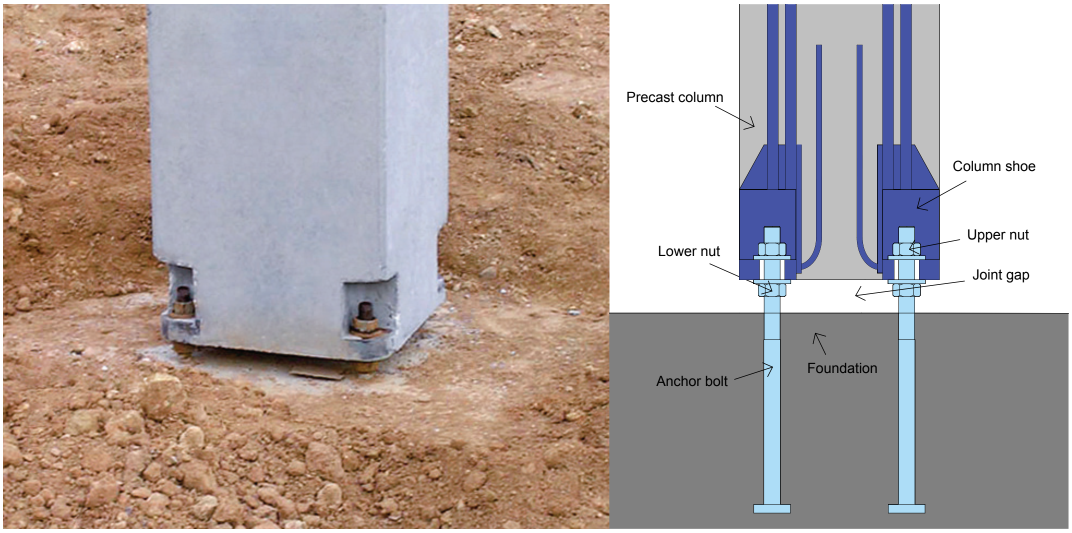

Figure 1. Photo and diagram of a typical bolted precast concrete column connection.

moment, compressive axial force, and shear force. Currently, there is no unified design method for the assessment of the shear resistance of bolted precast concrete column connections in harmonized design standards. In Europe, the lack of normative references is addressed through product-based technical approvals developed by the manufacturers of proprietary products.[2,3] The design method for shear resistance in such joints complies with methods commonly used for the design of steel structures[4] but the method is adjusted to conform with available test results.[5] The measured ultimate shear capacities (failure) of the bolted precast concrete column connections creates a basis for adjusting the method in the European standard EN 1993-1-8, *Eurocode 3: Design of Steel Structures—Design of Joints*.[4] The method in EN 1993-1-8, which was originally proposed to the drafting panel prEN 1993-1-8, was introduced in Gresnigt et al.[6] In that paper, the simplified model defined for the ultimate design strength of Grade 4.6 (400 MPa [58 ksi] nominal tensile strength and yield strength ratio of 0.6) and 8.8 (800 MPa [116 ksi] nominal tensile strength and yield strength ratio of 0.8) bolts is generalized for bolt materials with yield stresses ranging from 235 to 640 MPa (34.1 to 92.8 ksi).

In the design of structures and their connections, designers should not only pay attention to ultimate failure loads but also ensure that requirements for service use are met.[6] The European design standards[7] imply that the appearance of the structure is required to remain adequate in the serviceability limit states. In EN 1990,[7] the term "appearance" is concerned with such criteria as unallowed displacements, deflections, and deformations. In bolted precast concrete column connections, key appearance-related issues may include avoiding the yielding of bolts, cracking of concrete, or excessive slip between the column and foundation structure.

This paper presents an assessment of the shear transfer of bolted precast concrete column connections in serviceability limit states. It also provides an analytical evaluation of shear tests published by the authors in a previous study[8] and shear tests published for the first time herein. The authors compare the experimental values with results from analytical models available in the literature and propose further development needs. The main focus is on the method based on European standards, but methods based on U.S. codes are also applied for comparison.

Research significance

Although the use of bolted connections for precast concrete structures is becoming more popular, little research has been done on the structural behavior of such structures. Shear transfer, in particular, is not a widely studied topic. Because research is lacking, there is no harmonized design standard for this type of structure. Typically, designers use proprietary design methods subject to approvals by local building authorities.[2] The current method, originally adjusted for assessing the ultimate failure load in shear, may not be suitable for considering the maximum shear force based on requirements for serviceability limit states. The method currently specified in the European Organisation for Technical Assessment's Technical Report 068, *Design of Structural Connections with Column Shoes*,[2] must be evaluated against new experimental evidence, and needed adjustments should be proposed.

Laboratory tests

The authors performed 10 full-scale load tests, including 4 shear tests and 6 bending tests. The target was to expand the knowledge about the behavior of bolted connections of precast concrete columns. In a previous paper,[8] the authors presented and discussed the details of the shear tests and their results. The authors performed all tests with identical precast concrete columns measuring 350 × 350 × 1500 mm (14 × 14 × 59 in.) and foundation structures measuring 450 × 700 × 1400 mm (18 × 28 × 55 in.). Precast concrete columns were connected to foundation structures (Fig. 1), but in a horizontal orientation, and a 50 mm (2 in.) gap between the structures was filled with cement-based grout with specified strength class C50/60 (nominal cubic strength 60 MPa [8.7 ksi] in compression). The top end of the column was supported on a roller support, and the foundation structure was laid on strong floor and constrained vertically by a hydraulic actuator. **Figure 2** shows the shear test S03, where the loading has not yet been started. **Table 1** summarizes parameters of the test setups. Shear tests are identified with letter S, and the letter B is applied for bending tests.

The tests involved two different types of anchor bolts. Test S03 was conducted with M16 (16 mm diameter [0.63 in.]) reinforcing bar anchor bolts protruding out from the foundation element and nine tests with anchoring couplers and M16 threaded bars. M16 bolts have an effective cross-sectional area of 156 mm^2 (0.242 in.^{2}). While the anchor bolts were manufactured from one solid piece, anchoring couplers were manufactured by fixing the coupler and anchor tail together. The threaded bar is a separate piece and must be inserted to an anchoring coupler before column assembly. All precast concrete columns had column shoes compatible with the bolt size M16. The center distance between the bolts was 250 mm (10 in.) in both directions. In five test setups, joints between the grout and both precast concrete structures were treated with a release agent to reduce the bond between parts: in setups S02-plate, B02-0, B02-50, and B02-100, thin steel plates with a thickness t of 5 mm (0.2 in.) separated the grout and precast concrete structures, whereas in setup S01-oil, demolding oil covered the surfaces of the precast concrete structures.

The external transverse load P was applied through a steel bar by a hydraulic actuator with a loading rate of 1 Hz. In the shear tests, the lever arm of the load was minimized by loading the column end just above the grout, which means that the real lever arm was at least 50 mm (2 in.). However, no significant bending moment was transferred through connections and the lever arm 0 has been input to **Table 1**.

In the bending tests, transverse load P loaded the specimens with a distance 500 mm (20 in.) from the surface of the end of the foundation. The simultaneous axial load F was constant throughout the test, with F equal to 0, 50, or 100 kN (0, 11, or 22 kip). A hydraulic jack applied the axial load F and an additional support restricted the horizontal movement of the specimens at the other end of the foundation. **Figure 3** shows a schematic presentation of the shear and bending tests.

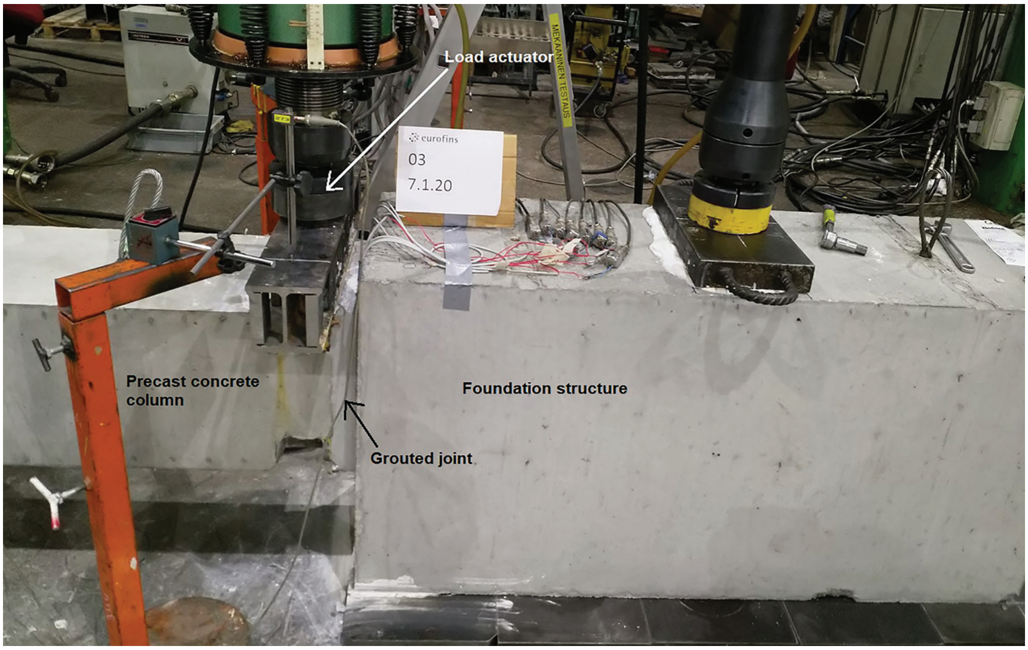

Figure 2. Test setup for shear testing.

Table 1. Characteristics of test setups

Test setup	Bolt type	Thread material	Release agent	Lever arm, mm	Axial compression, kN
S03	Anchor bolt	B500B reinforcing bar	n/a	0	0
S01	Anchoring coupler	Grade 8.8 (800 MPa nominal tensile strength and yield strength ratio of 0.8)	n/a	0	0
S01-oil			Oil	0	0
S02-plate			Thin steel plates	0	0
B01-0			n/a	500	0
B01-50			n/a	500	50
B01-100			n/a	500	100
B02-0			Thin steel plates	500	0
B02-50			Thin steel plates	500	50
B02-100			Thin steel plates	500	100

Note: n/a = not applicable. B500B = 73 ksi; 1 mm = 0.0394 in.; 1 kN = 0.225 kip; 1 MPa = 0.145 ksi.

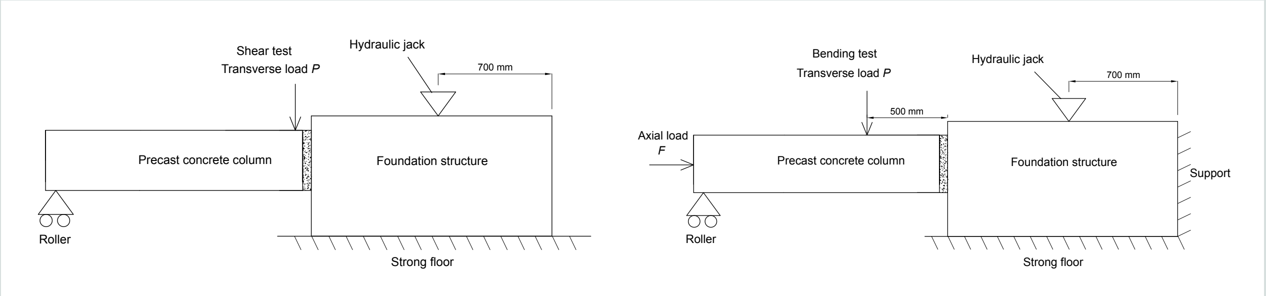

Figure 3. Shear and bending tests. Note: 1 mm = 0.394 in.

Linear variable differential transducers (LVDTs) recorded displacement in the transverse load direction from both sides of the bolted column end with a data sampling rate of 1 Hz. **Figure 4** presents a section view of the joint with placement and identification of loading actuator, LVDTs and strain gauges. LVDTs were placed on the steel bar (points 9 and 10 in Fig. 4) and the transverse load was applied to the middle (point 0). The derived mean displacement d is an average from measured values for points 9 and 10. Strain gauges (marked 1–8 in Fig. 4) were used in the shear tests to record tensile and compressive strains of the threads; these gauges were glued on threads at the level of foundation surface. Four LVDTs (marked 11–14 in Fig. 4) were used in the bending tests to record relative slip between the bolts and column shoes.

The authors tested the concrete strength of the precast concrete structures and hardened grout pads by crushing concrete test cubes measuring 150 × 150 × 150 mm (5.9 × 5.9 × 5.9 in.) in accordance with EN 12390, *Testing Hardened Concrete: Determination of the Carbonation Resistance of Concrete—Accelerated Carbonation Method*.[9] The age of the cubes varied between 12 to 15 days, and their strengths were tested on the same day that the related load tests took place. The measured cubic strengths of grout varied slightly, ranging from 59.0 to 61.9 MPa (8.56 to 8.98 ksi). The average cubic strengths of precast concrete column and foundation structure were 74.3 MPa (10.8 ksi) and 70.9 MPa (10.3 ksi), respectively. The authors did not measure the material properties of the thread parts. Yielding of steel was not considered to occur under service state loads, and nominal material properties of B500B and Grade 8.8 steel are used in practical design.[10,11]

Before loading the specimens to failure, two loading cycles were applied by raising the actuator load to a maximum of 30 kN (6.7 kip) and then unloading back to zero to study and eliminate the irreversible part of initial deformations. It is assumed that these small deformations were caused by the settlement of the foundation structure against the strong floor. That most likely happened because of the horizontal placement of the structures and would not have happened when precast concrete columns were placed vertically. This was determined by observing the remaining displacement when the load had been removed, and these displacements have been taken into account in defined load-displacement and shear force-displacement patterns. Diagram on the left in **Fig. 5** presents the load-displacement patterns from all 10 tests. These patterns have been limited to the ultimate load corresponding to failure of connection. The rupture of the bottom bolts was the governing failure mode in all tests.

Structural behavior

In the shear tests, the applied load equaled the shear force transferred through the joint. In the bending tests, part of the applied load went to the other support (roller), and shear force through the bolted precast concrete column connection was proportional to the applied load (Eq. [1]).

$$V = \frac{P}{2}\left(\frac{3a}{L} - \frac{a^3}{L^3}\right) \tag{1}$$

where

V = shear force through the bolted connection in the bending tests

P = applied load

a = distance of the load from the further support (roller)

L = span length (sum of the length of the column and the thickness of the grout)

The simplified static model in Eq. (1) is based on the assumption that one end of the column is freely supported (roller) and the other end is fixed (bolted connection). Even if bolted connections between the column and foundation are often designed as hinge, a connection with four bolts is capable of transferring bending moment. A moment-resistant connection

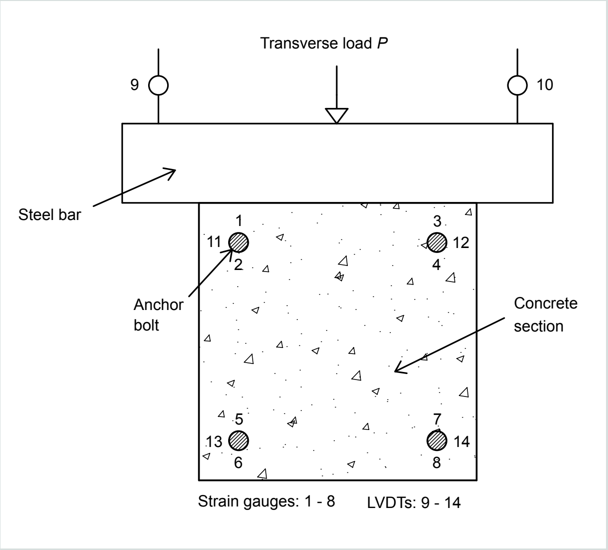

Figure 4. Section view of the grouted connection. Note: LVDT = linear variable differential transducer.

is needed when columns are designed as cantilevers for stiffening the building frame or when columns are erected without additional propping. Many engineering textbooks provide such simplified relationships between the applied force and shear force for single-span structures. Equation (1) results in V equal to 0.861P with the lever arm of 500 mm (20 in.).

While executing the shear tests, the first visible response in the joints appeared as cracking of the grout (**Fig. 6**). The shear force level at which these cracks appeared on the outer surface of the grout was not recorded. However, it is assumed that the stiffness reduction (end of linearity) observed in the shear force–displacement patterns is associated with the irreversible deformations of the grout, which eventually led to cracks visible on the outer surfaces (Fig. 5). In the shear tests, the measured strain development of the threads further supports this assumption because the stress levels of threads were well below the nominal yield strengths. Diagram on the right in

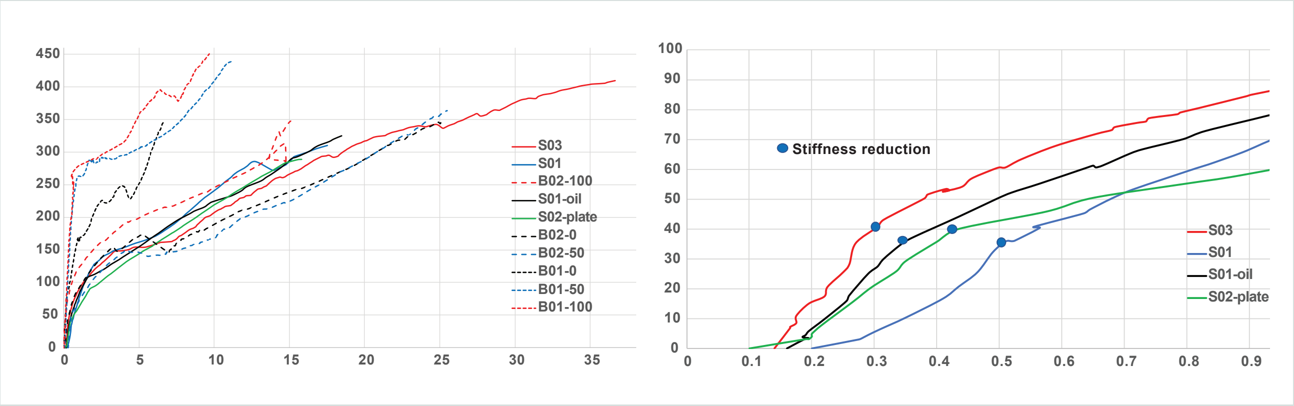

Figure 5. The measured load-displacement patters for all test setups and shear force-displacement patterns from the shear tests for displacement range 0 to 1 mm. Note: 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

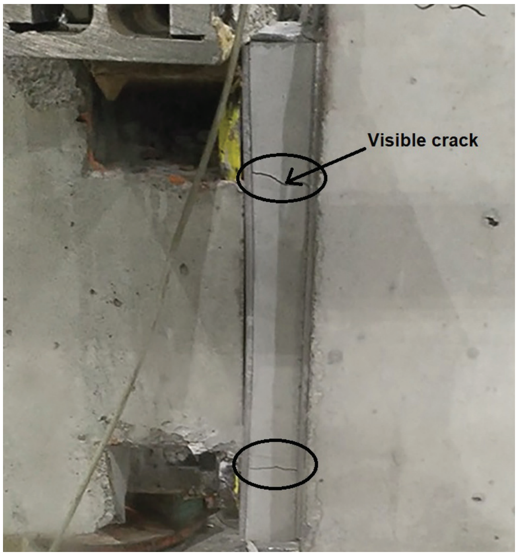

Figure 6. Cracks in grout pad during loading.

Table 2. Measured maximum shear forces and shear forces associated with the major slip

Test setup	Maximum measured shear force V_{meas}, kN	Shear force related to major slip, kN
S03	40	n.d.
S01	36	n.d.
S01-oil	36	n.d.
S02-plate	40	n.d.
B01-0	65	150
B01-50	140	225
B01-100	150	200
B02-0	36	120
B02-50	40	125
B02-100	40	130

Note: n.d. = no data. 1 kN = 0.225 kip.

Figure 5 illustrates how the maximum shear force from each bending test was obtained by following the same logic (to find the shear force value where linearity ends). **Table 2** shows the measured shear forces associated with the assumed irreversible deformations of the grout.

In the bending tests, connections transferred bending moment and axial compression in addition to shear force. Due to bending moment and axial compressive force, compressive stresses act across the joint. Friction can transmit shear forces if such compressive stresses exist.[12] The coefficient of friction is a direct measure of the amount of friction between those surfaces. By comparing the load-displacement patterns, the positive effect of (static) friction was observed. In the bending tests B01-0, B01-50, and B01-100, the presence of friction force enhanced the stiffness of the connections by extending the linear part of the shear force–displacement pattern (delaying cracking). Friction properties of the joint with thin steel plates were weaker. In the test setups B02-0, B02-50, and B02-100, bending moment and axial compression did not have a similar effect, but performances were parallel with those of the shear tests (Fig. 5).

In the absence of friction forces due to external loading, the shear load is transferred from the column to the foundation only through a connection between the bolts and column shoes. In bolted precast concrete column connections, bolts pass through oversized bolt holes, which means that they may not be in direct contact with column shoes. A major slip could endanger the serviceability limit states of the precast concrete column.[7] Thus, a slip-resistant connection should prevent a major slip. A slip-resistant connection is a friction-based connection, which develops when the upper nuts of the anchor bolts are tightened against column shoes and should not be confused with friction between the column and grout, which is affected by their joint faces. While the slip-resistant connection based on tightening of nuts transfers shear from column to bolts, friction forces from external loading transfer shear directly from the column to grout. Table 2 shows the maximum shear forces together with shear forces associated with major slips, as measured by LVDTs 11 to 14. All connections experienced the loss of stiffness before the major slip occurred.

Evaluation of available analytical methods based on European standards

In the current design practice in Europe, one can calculate the design value of shear resistance of bolted steel column bases using Eq. (2) in accordance with section 6.2.2 Eq. (6.3) of EN 1993-1-8.

$$F_{v,Rd} = nF_{vb,Rd} + C_{f,d}N_{c,Ed} \tag{2}$$

where

n = number of bolts in the base plate

$F_{v,Rd}$ = design shear resistance of the anchor bolt

$C_{f,d}$ = coefficient of friction between the base plate and grout layer

$N_{c,Ed}$ = design value of the normal compressive force

The coefficient of friction $C_{f,d}$ for sand-cement mortar[6] is 0.2. For other types of grouts, the coefficient should be determined by testing in accordance with EN 1990, Annex D.[7]

The design shear resistance of the anchor bolt $F_{v,Rd}$ is the lesser of the design bearing resistance for the anchor bolt $F_{1,vb,Rd}$ and the design shear resistance of the anchor bolt $F_{2,vb,Rd}$: (Eq. [3])

$$F_{v,Rd} = \min(F_{1,vb,Rd}, F_{2,vb,Rd}) \tag{3}$$

The bearing resistance for the anchor bolt in an oversized bolt hole is calculated with Eq. (4).

$$F_{1,vb,Rd} = 0.8\frac{k_1\alpha_b f_{base,u} d_b t_{base}}{\gamma_{M2}} \tag{4}$$

where

k_1 = coefficient in accordance with Table 3.4 of EN 1993-1-8

a_b = coefficient in accordance with Table 3.4 of EN 1993-1-8

$f_{base,u}$ = ultimate strength of the base plate (the column shoe in the case of the precast concrete column)

d_b = bolt diameter

t_{base} = thickness of the base plate

γ_{M2} = partial safety factor = 1.0 in the serviceability limit states

When using common dimensions and geometries for bolts and base plates, the design shear resistance of the anchor bolt is more critical, and can be calculated using Eq. (5):

$$F_{2vb,Rd} = \frac{a_b f_{ub} A_s}{\gamma_{M2}} \tag{5}$$

where

a_b = 0.44 – 0.0003f_{yb} (f_{yb} in MPa)

f_{yb} = yield strength of the anchor bolt

f_{ub} = ultimate strength of the anchor bolt

A_s = effective cross-sectional area of the thread

The minimum and maximum limitations for the nominal yield strength are 235 MPa (34.1 ksi) and 640 MPa (92.8 ksi), respectively.

Equation (6) is the design method currently used in Europe for the design of bolted connections of precast concrete column bases.[1]

$$V_{Rd} = n_c k_s F_{vb,Rd} + \mu F_{cd} \tag{6}$$

where

V_{Rd} = design shear resistance of the bolted connection

n_c = number of individual column shoes that are transversely and horizontally compressed against the end of the column

k_s = shear resistance factor

μ = coefficient of friction = 0.2

F_{cd} = design value of compressive force resultant (Load factor 1.0 is applied in the serviceability limit states.)

From a mechanical point of view, Eq. (2) and (6) are equivalent. They follow a similar logic, with the following deviations:

- k_s adjusts the design method in Eq. (2) for bolted column shoe connections
- F_{cd} also considers compressive stresses due to bending moment
- n_c considers column shoes that are transversely and horizontally compressed against the end of the column

The method in Eq. (6) was used for assessing the maximum shear force in the serviceability limit states. Nominal and measured mean material properties were used without the partial safety factors and applied transverse and horizontal loads without the load factor. Therefore, Eq. (6) takes the form of Eq. (7) for further use.

$$V_R = n_c k_s[\min(F_{1,vb,R}, F_{2,vb,R})] + \mu F_c \tag{7}$$

where

V_R = calculated maximum shear force in the serviceability limit states

$F_{1,vb,R}$ = bearing resistance for the anchor bolt

$F_{2,vb,R}$ = shear resistance of the anchor bolt

F_c = compressive force

The main difference between the two methods is in that while Eq. (2) allows the assumption of simultaneous action of all bolts in the cross section, Eq. (7) assumes that bolts placed only within a single row carry the shear load. The developers of the current design method defined the shear resistance factor k_s by comparing the calculated and measured failure loads, and a conservative value 1.0 was proposed for design.[5]

Currently, there are no requirements or recommendations for assessing the effect of transverse load on the behavior of the bolted connection in the serviceability limit states. As mentioned earlier, cracks started to appear in the grout at shear forces significantly lower than those associated with the ultimate load. Cracking was associated with the loss of stiffness of the connection. In practice, visible cracks should be avoided for durability and aesthetic reasons. Thus, it is reasonable and conservative to assume that the shear forces associated with the loss of the stiffness of the joint, which can be associated with grout deformations, define the serviceability limit states of the connection.

In the shear tests with no significant contribution of joint friction, stiffness reduction occurred at a low load level. If the current design method[1] is considered, the first part of Eq. (7) should be able to describe the performance without the effect of joint friction When the connections also transferred bending moment and axial compression, stiffness reduction occurred at much a higher load level, and the latter part of Eq. (7) should be able to describe the positive effect of joint friction

To evaluate the effect of friction, a compressive force resultant from bending moment had to be defined. Just as shear force was converted from the applied load in Eq. (1), a similar relationship between the applied load and bending moment M for single-span structures is presented as Eq. (8).

$$M = \frac{Pa\left(L^2 - a^2\right)}{2L^2} \tag{8}$$

Equation (8) results in M equal to $0.284P$. Tensile $F_{M,t}$ and compressive $F_{M,c}$ forces from bending must balance the bending moment in the connection. These forces $F_{M,t}$ and $F_{M,c}$ were generated from tensile stresses in bolts and compressive stress in concrete (**Fig. 7**). For practicality, it was assumed that the elastic range of the concrete stress-strain relationship continues to 45% from the mean compressive strength f_{cm}.[13] Thus, the concrete stress remained within elastic range and the authors considered $0.45f_{cm}$ as the maximum stress. The system was also simplified by assuming that the tensile force $F_{M,t}$ is transferred only by one row of bolts. The balance Eq. (9) was used to solve y_c, which was then applied to calculate internal force resultants using Eq. (10).

$$M - 0.45 f_{cm} \frac{y_c}{2} b_c \left[b_c - \left(e_b + \frac{y_c}{3} \right) \right] = 0 \tag{9}$$

where

y_c = compressed area between the grout and the precast concrete structure

b_c = width of the column section

e_b = center distance of the bolt from the column edge

$$F_{M,c} = -F_{M,t} = 0.45 f_{cm} \frac{y_c}{2} b_c \tag{10}$$

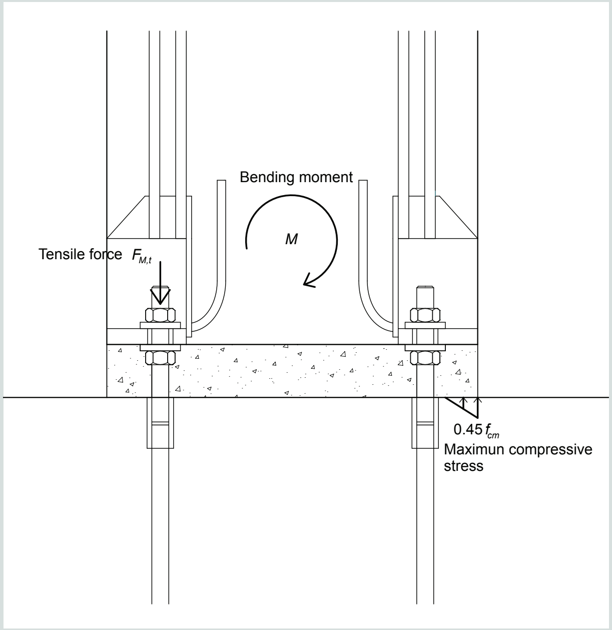

Figure 7. Internal forces balancing the bending moment. Note: f_{cm} = mean compressive strength of concrete.

The authors superimposed the resulting compressive force resultant $F_{M,c}$ with the axial load F to define the total compressive force resultant F_c. The effective cross-sectional area of the bolt thread A_s is 156 mm^2 (0.242 in.^{2}), and the number of effective bolts in shear n is 2. The mean compressive strength of the grout was calculated from the measured cubic strength (Table 2) according to EN 1992-1-1, Table 3.1.[14] By applying a cubic strength of 60.2 MPa (8.73 ksi) and assuming cylindrical strength to be 80% of cubic strength, the mean compressive strength was determined to be 56.2 MPa (8.15 ksi). Nominal yield and ultimate strengths were applied for the bolts (Table 2). In the bending tests, the maximum shear force was iteratively determined by increasing the load P and identifying when shear force from Eq. (1) exceeded the calculated maximum shear force according to Eq. (7).

The calculated maximum shear forces were compared with the measured values of ultimate shear force and shear force corresponding to the first reduction of stiffness in the joint. The statistical evaluation performed according to EN 1990 Annex D[7] demonstrates that while the current design methods lead to a conservative assessment of the resistance of the joint at the ultimate limit states, those methods are unconservative in assessing the behavior of the joint in the serviceability limit states (**Table 3**). The authors concluded the following:

- The first part of Eq. (5) cannot describe the cracking of grout and overestimates the maximum shear force when the friction forces are absent.
- The coefficient of friction 0.2 is not suitable for either joint type.

- Test setups 02-0, 02-50, and 02-100 show evidence that the coefficient of friction is zero or close to zero for joints treated with thin steel plates, whereas the test setups 01-0, 01-50, and 01-100 predict larger coefficients of friction for joints without treatment.

Evaluation of the analytical methods based on U.S. standards

This study primarily focused on the method based on European standards because that method is part of the European Technical Assessment for column shoes.[3] Additionally, methods based on U.S. standards were considered to widen the scope of the comparison, specifically, the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318-19R)*[15] and the American Institute of Steel Construction's *Specification for Structural Steel Buildings* (ANSI/AISC 360-16).[16]

According to section J3.6 in ANSI/AISC 360-16, the allowable shear strength R_n/Ω (where R_n is the nominal shear strength and Ω is the reduction factor for allowable stress design) of a snug-tightened or pretensioned high-strength bolt or threaded bar shall be calculated to the limit states of shear rupture using Eq. (11) for R_n and 2.00 for Ω.

$$R_n = F_n A_b \qquad (11)$$

where

F_n = F_{nv} = nominal shear stress = 372 MPa (54 ksi) according to Table J3.2 of ANSI/AISC 360-16

A_b = nominal unthreaded body area of bolt or threaded part = 201 mm^2 (0.312 in.^{2}) for bolt size M16

Allowable stress design is also referred to as the service load design.[17] Therefore, it was used for calculating the maximum shear force in the serviceability limit states. For one bolt, the allowable shear strength of threads with material similar to Grade 8.8 (116 ksi) steel is 37.4 kN (8.41 kip).

According to ACI 318-19 section 17.7.1, the shear strength of a cast-in headed bolt V_{sa} can be assessed by Eq. (12).

$$V_{sa} = 0.8\phi 0.6 A_{se,V} f_{uta} \qquad (12)$$

where

ϕ = strength reduction factor = 0.7 when concrete break-out is assumed to be the governing failure criterion (ACI 318-19 Table 17.5.3[b])

$A_{se,V}$ = effective cross-sectional area of an anchor in shear = 156 mm2 (0.242 in.2)

f_{uta} = ultimate strength of an anchor = 800 MPa (116 ksi)

A strength reduction factor of 0.8 is used with built-up grout pads. The authors chose to use a strength reduction factor of 0.7 to consider the failure of grout as critical behavior in the

Table 3 Statistical evaluation based on the European method

Test setup	V_R, kN	$V_{meas,SLS}$, kN	$V_{meas,ULS}$, kN	$V_{meas,SLS}/V_R$	$V_{meas,ULS}/V_R$
S03	49.8	40	410	0.80	8.23
S01	61.9	36	310	0.58	5.01
S01-oil	61.9	36	325	0.58	5.25
S02-plate	61.9	40	289	0.65	4.67
B01-0	80.0	65	298	0.81	3.73
B01-50	92.9	140	378	1.51	4.07
B01-100	105.7	150	388	1.42	3.67
B02-0	80.0	36	298	0.45	3.73
B02-50	92.9	40	313	0.43	3.37
B02-100	105.7	40	299	0.38	2.83
Mean				0.76	4.46
Deviation				0.40	1.52
COV				0.52	0.34

Note: *COV* = coefficient of variation; $V_{meas,SLS}$ = maximum measured shear force defined by serviceability limit states criterion; $V_{meas,ULS}$ = maximum measured shear force defined by ultimate limit states criterion; V_R = calculated maximum shear force in the serviceability limit states. 1 kN = 0.225 kip.

serviceability limit states. For one cast-in anchor bolt, the shear strength is 41.9 kN (9.42 kip) with Grade 8.8 (116 ksi) steel and 28.8 kN (6.47 kip) with Grade B500B (73 ksi) reinforcing steel.

Neither Eq. (11) nor Eq. (12) considers friction due to external compressive forces as a shear transfer mechanism, and ACI 318-19 does not allow connections to rely solely on friction. ANSI/AISC 360-16 recognizes friction due to the column vertical load as generally sufficient to transfer the shear from the column to the foundation, but it does not make design recommendations.

According to AISC's *Steel Design Guide 1. Base Plate and Anchor Rod Design,*[18] attention should be paid to the way the force is transferred from the base plate to the anchor bolts. When using oversized bolt holes, the guide recommends a cautious approach, such as considering only two of the anchor bolts to transfer the shear. This approach is similar to the one found in EOTA Technical report TR068.[2] **Table 4** compares the calculated maximum service state shear forces based on U.S. standards and measured values. The following conclusions were made:

- Calculation methods for bolt strength based on ACI 318-19 and ANSI/AISC 360-16 did not adequately predict the maximum shear forces in the serviceability limit states, as the calculations were both unconservative and unreliable.
- The biggest outliers were from tests 01-50 and 01-100. The ACI 318-19 and ANSI/AISC 360-16 calculation methods did not consider the effect of friction, which was considered a reason for improved performance in the tests.
- The ACI 318-19 and ANSI/AISC 360-16 methods demonstrated better consistency with the measured ultimate shear forces. Of the two methods, the one based on ACI 318-19, which is applicable only for high-strength bolts, provided better predictions.

Upgrade of the current European methodology

The current European method complies with the standards for steel structures,[4] and the resistance depends on properties of steel parts. According to tests and the authors' interpretations, the serviceability limit states of the connection is governed by the cracking of grout. Consequently, one conclusion from the tests is that an upper limit should be added to the current method to achieve a better agreement with requirements in the serviceability limit states. The upper limit should only be used when considering the serviceability limit states. A series of new tests would be required to verify the general method for such a limit based on the physical response (cracking) of the joint . In those tests, investigators should vary the properties of the grouted joints. In the absence of such a series of tests, the authors defined the upper limit based on the existing design rules for concrete structures.

Table 4. Statistical evaluation based on the U.S. methods

Test setup	R_n/Ω, kN	V_{sa}, kN	$V_{meas,SLS}/R_n/\Omega$	$V_{meas,ULS}/R_n/\Omega$	$V_{meas,SLS}/V_{sa}$	$V_{meas,ULS}/V_{sa}$
S03	n/a	57.6	n/a	n/a	0.69	7.12
S01	74.8	83.8	0.48	4.14	0.43	3.70
S01-oil	74.8	83.8	0.48	4.34	0.43	3.88
S02-plate	74.8	83.8	0.53	3.86	0.48	3.45
B01-0	74.8	83.8	0.87	3.98	0.78	3.56
B01-50	74.8	83.8	1.87	5.05	1.67	4.51
B01-100	74.8	83.8	2.01	5.19	1.79	4.63
B02-0	74.8	83.8	0.48	3.98	0.43	3.56
B02-50	74.8	83.8	0.53	4.18	0.48	3.74
B02-100	74.8	83.8	0.53	4.00	0.48	3.57
Mean			0.86	4.30	0.77	4.17
Deviation			0.62	0.49	0.52	1.11
COV			0.72	0.11	0.68	0.27

Note: *COV* = coefficient of variation. R_n = nominal shear strength; $V_{meas,SLS}$ = maximum measured shear force defined by serviceability limit states criterion; $V_{meas,ULS}$ = maximum measured shear force defined by ultimate limit states criterion; V_{sa} = shear strength of cast-in headed bolt; Ω = reduction factor for allowable stress design. 1 kN = 0.225 kip.

In design rules for shear transfer through reinforced concrete structures and reinforced joints between concrete structures, it is common to assume that local crushing of concrete strut may govern the failure. For example, shear and punching shear verifications in EN 1992-1-1[14] have an upper limit representing the development of diagonal cracking in the concrete member specified as Eq. (13).

$$v_{c,max} = 0.5 v f_{ck} \tag{13}$$

where

$v_{c,max}$ = maximum stress in the strut

v = reduction factor for strength of diagonal strut = $0.6\left(1-\frac{f_{ck}}{250}\right)$ (f_{ck} in MPa)

f_{ck} = cylinder strength of grout

According to EN 1992-1-1, f_{ck} is considered to be 80% of the cubic strength. Consequently, the authors propose Eq. (14) as verification for the maximum shear force in the serviceability limit states $V_{R,mod}$.

$$V_{R,mod} = \{n_c k_s[\min(F_{1,vb,R}, F_{2,vb,R})] \leq \text{Vg,max}\} + \mu F_c \tag{14}$$

where

$V_{g,max}$ = maximum shear force transferred by grout

The authors propose that the maximum shear force transferred by grout is calculated as follows:

$$V_{g,max} = v_{c,max} A_c$$

where

A_c = total contact area = $n_{tot} d_b t_g$

n_{tot} = total number of bolts in the connection

d_b = bolt diameter

t_g = thickness of the grout layer

Table 5 compares the results from Eq. (14) to the measured shear forces corresponding to the first loss of stiffness. Introducing the upper limit $V_{g,max}$ makes the current method more conservative, but use of the coefficient of friction μ of 0.2 still leads to an unconservative result.

According to EN 1992-1-1 section 6.2.5, the coefficient of friction 0.5 can be applied for joints between fresh grout and precast concrete. The surface of the precast concrete element can be classified as very smooth because it was cast against a wooden mold. When using thin plates as a release agent, the weakest joint (from friction aspect) is presumably formed between the plate and the precast concrete column. The authors have not been able to find information about friction properties in such joints. However, based on observations

Table 5. Statistical evaluation based on the upgraded European method

Test setup	$V_{R,mod}$, kN	$V_{R,mod1}$, kN	$V_{R,mod2}$, kN	$V_{meas,SLS}$, kN	$V_{meas,SLS}/V_{R,mod}$	$V_{meas,SLS}/V_{R,mod1}$	$V_{meas,SLS}/V_{R,mod2}$
S03	37.3	37.3	37.3	40	1.07	1.07	1.07
S01	37.3	37.3	37.3	36	0.97	0.97	0.97
S01-oil	37.3	37.3	37.3	36	0.97	0.97	0.97
S02-plate	37.3	37.3	37.3	40	1.07	1.07	1.07
B01-0	48.1	85.3	67.4	65	1.35	0.76	0.96
B01-50	60.7	146.4	104.9	140	2.31	0.96	1.33
B01-100	73.7	211.0	142.9	150	2.04	0.71	1.05
B02-0	48.1	37.3	37.3	36	0.75	0.97	0.97
B02-50	60.7	37.3	37.3	40	0.66	1.07	1.07
B02-100	73.7	37.3	37.3	40	0.54	1.07	1.07
Mean					1.17	0.96	1.05
Deviation					0.58	0.13	0.11
COV					0.49	0.14	0.10

Note: *COV* = coefficient of variation; $V_{R,mod}$ = calculated maximum shear force in the serviceability limit states; $V_{R,mod1}$ = calculated maximum shear force; $V_{R,mod2}$ = calculated maximum shear force; $V_{meas,SLS}$ = maximum measured shear force defined by serviceability limit states criterion. 1 kN = 0.225 kip.

made by the authors, the coefficient of friction is approximately zero when the connection is equipped with thin, loose steel plates. The maximum shear forces are redefined according to Eq. (14) with following values used for μ:

$V_{R,mod1}$

- $\mu = 0.5$ for tests 01-0, 01-50, and 01-100
- $\mu = 0$ for tests 02-0, 02-50, and 02-100

$V_{R,mod2}$

- $\mu = 0.4$ for tests 01-0, 01-50, and 01-100
- $\mu = 0$ for tests 02-0, 02-50, and 02-100

By adjusting the coefficients of friction, the conservativeness of the current method is improved. The third comparison suggests that the coefficient of friction μ equal to 0.4 is a better assumption for the joints between grout and precast concrete than the value recommended by EN 1992-1-1 (Table 5).

Conclusion

The tests presented in this paper and the authors' previous paper[8] demonstrate the capacity of bolted connections of precast concrete columns to transfer shear forces. The test results were analyzed and an interpretation of the shear force-displacement behavior of the connections was proposed. While the joint can carry relatively high ultimate loads, such loads are associated with deformations that are significantly larger than those associated with any initial cracking of grout in the joint. Thus, it is proposed to limit the maximum shear force that can be transferred by the joint to the level associated with the cracking of the grout.

The analytical approaches currently in use in Europe and the United Sates for the design of shear resistance of bolted connections were compared with the experimentally determined maximum shear forces. While the analytical methods lead to a conservative assessment of the ultimate resistance of the joint, those methods do not assess the effect of cracking in the grout on the behavior of the connection.

Finally, a new analytical approach for the assessment of the effect of grout on the behavior of the structure in the serviceability limit states was proposed and verified against experimental results. The proposed method shows promising results. However, additional experimental research (primarily focusing on varying the dimensions and strength of grout) is needed to further calibrate and validate it.

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Notation

a = distance of the transverse load from the further support (roller)

a_b = coefficient in accordance with Table 3.4 of EN 1993-1-8

A_b = nominal unthreaded body area of bolt or threaded part

A_c = total contact area

A_s = effective cross-sectional area of the thread

$A_{se,V}$ = effective cross-sectional area of an anchor in shear

b_c = width of the column section

$C_{f,d}$ = coefficient of friction

COV = coefficient of variation

d = mean displacement of the column end

d_b = bolt diameter

e_b = center distance of the bolt from column edge

$f_{base,u}$ = ultimate strength of the base plate (the column shoe in the case of the precast concrete column)

f_{ck} = cylindrical strength of concrete material

f_{cm} = mean compressive strength of concrete material

f_{ub} = ultimate strength of the anchor bolt

f_{uta} = ultimate strength of an anchor

f_{yb} = yield strength of the anchor bolt

F = axial load applied to test specimens

F_c = compressive force

F_{cd} = compressive design force

$F_{M,c}$ = compressive force from bending

$F_{M,t}$ = tensile force from bending

F_n = nominal stress considered either as shear or tensile stress

F_{nv} = nominal shear stress

$F_{vb,Rd}$ = design shear resistance of the anchor bolt

$F_{1,vb,R}$ = bearing resistance for the anchor bolt

$F_{1,vb,Rd}$ = design bearing resistance for the anchor bolt

$F_{2,vb,R}$ = shear resistance of the anchor bolt

$F_{2,vb,Rd}$ = design shear resistance of the anchor bolt

k_1 = coefficient in accordance with Table 3.4 of EN 1993-1-8

k_s = shear resistance factor

L = span length

M = bending moment from the transverse load

n = number of bolts in the baseplate

n_c = number of individual column shoes that are transversely and horizontally compressed against the end of the column

n_{tot} = total number of bolts in the connection

$N_{c,Ed}$ = design value of the normal compressive force

P = transverse load applied on test specimens

R_n = nominal shear strength

t_{base} = thickness of the base plate

t_g = thickness of the grout layer

v = reduction factor for the strength of the diagonal strut

$v_{c,max}$ = maximum stress in the strut

V = shear force through the bolted connection in the bending tests

$V_{g,max}$ = maximum shear force transferred by grout

V_{meas} = maximum measured shear force

$V_{meas,SLS}$ = maximum measured shear force defined by serviceability limit states criterion

$V_{meas,ULS}$ = maximum measured shear force defined by ultimate limit states criterion

V_R = calculated maximum shear force in the serviceability limit states

V_{Rd} = design shear resistance of the bolted connection

$V_{R,mod}$ = calculated maximum shear force in the serviceability limit states

$V_{R,mod1}$ = calculated maximum shear force with adjusted coefficients of friction

$V_{R,mod2}$ = calculated maximum shear force with further adjusted coefficients of friction

V_{sa} = shear strength of cast-in headed bolt

y_c = compressed area between the grout and the precast concrete structure

γ_{M2} = partial safety factor

μ = coefficient of friction

ϕ = strength reduction factor

Ω = reduction factor for allowable stress design

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Abstract

Shear transfer through bolted precast concrete column connections is not a widely studied topic. The current design method is based on the standards for steel structures, and the developers adjusted it to conform with previous shear test results. The method adjusted for design in the ultimate limit states may not be suitable for assessing performance in the serviceability limit states, where the maximum shear force is limited either based on unallowed deformations or displacement. This paper assesses the suitability of the current design method for evaluating the maximum shear force in the serviceability limit states. When the authors applied the current method with load and safety factors usable in the serviceability limit states and compared the analytical results against the experimental data, they concluded that adjustments to existing design methods are needed. Improvements for design in the serviceability limit states are proposed.

Keywords

Bolted connection, column design, serviceability limit state, shear.

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