

ETA TESTS AND DESIGN OF HPKM COLUMN SHOE CONNECTIONS

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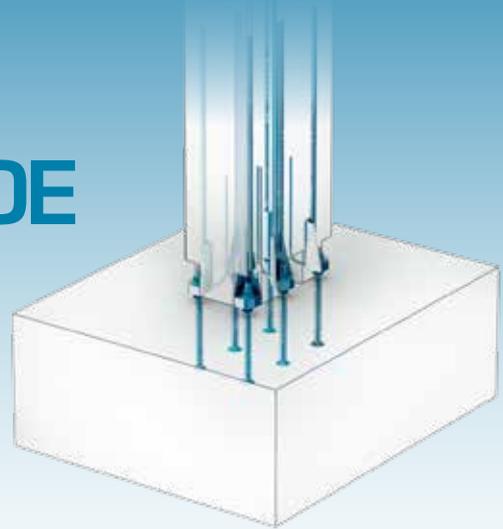
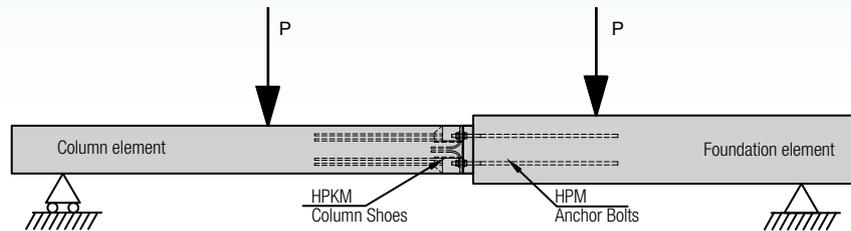


Figure 1. Illustration of full-scale bending resistance (BR) test arrangements.



INTRODUCTION

HPKM Column Shoes are fastening components used to create cost-effective stiff connections between precast concrete columns and foundations or precast columns and other columns. Making concrete columns using precast technology enables many competitive advantages to be realized, including speed of construction, accurate tolerances, high fire resistance, and high quality. Connections between precast columns are quick and easy to install, while also being economical. Standardized products will enhance fast deliveries and speed up lead times. Peikko's focus is to ensure the free movement of HPKM Column Shoes without technical or trade barriers within the European Union. CE marking can be perceived as a product passport for this purpose. CE marking is also a declaration by the manufacturer that the product meets certain public safety requirements, such as mechanical and fire resistances. Based on ETA-13/0603 [7], HPKM Column Shoes received the right to use the CE marking. Peikko Group aims to make the design process quicker and easier. ETA will simplify designers' work, because the same design rules and methods, essentially a common design language, can be used everywhere.

PRE-TESTS FOR CUAP AND INITIAL TESTS FOR ETA

In recent decades, some studies have been carried out on precast column connections and bolted connections. However, these studies did not answer every question, failing to address stiffness, shear behavior, and fire impacts. There were no widely agreed practices for verifying the performance of bolted connections. That is why some pre-tests for the Common Understanding of Assessment Procedure (CUAP) design principles were required to calibrate the ETA test arrangements. The aim of the pre-tests was to verify that the proposed design methods fit safely with practiced design principles. These pre-tests were carried out in 2001, 2003, 2004, and 2009. The initial tests specified in CUAP [11] were required for all of Peikko

Group's certified manufacturing units. These tests were carried out in 2012 and 2014.

The purpose of the tests is to measure the bending or shear resistance of the connection and compare the bending stiffness of the column inside and outside the column shoe zone (see Fig. 2). A common target of all tests was to obtain reliable and real behavioral information about HPKM column connections and their main components. The tests were designed to confirm that the HPKM Column Shoe connections will behave rigidly and the failure mode of the connection is ductile in all conditions. Only longer HPM/P Anchor Bolts were used in the concreted test specimens to keep the focus purely on the column connection's behavior. This selection enabled some typical failure modes for shorter headed HPM/L Anchor Bolts, such as concrete cone, blow-out, and pull-out failures, to be eliminated.

In all test specimens the concrete grade was C30/37 according to EN 1992-1-1 [1]. The column shoe connection was grouted with self-compacting, rapidly hardening, low-shrink grout. The maximum aggregate size of the grout was 4mm and the concrete grade C50/60. The steel material strength of HPKM Column Shoes and HPM/P Anchor Bolts were verified by tension tests before concreting the test specimens.

Table 1. Column shoe types, anchor bolts, and sizes of test specimens.

Column Shoe Type	Anchor Bolt Type	Size of column element		Size of foundation element		Number of tests
		h x b (mm ²)	Length (mm)	h x b (mm ²)	Length (mm)	
HPKM 16	HPM 16/P	235 x 235	2200	335 x 335	1850	1
HPKM 16	HPM 16/P	230 x 230	1600	330 x 330	1500	2
HPKM 24	HPM 24/P	270 x 270	2850	370 x 370	2000	1
HPKM 24	HPM 24/P	250 x 250	2150	350 x 350	1900	1
HPKM 39	HPM 39/P	380 x 380	3800	480 x 480	3000	1
HPKM 39	HPM 39/P	360 x 360	2540	460 x 460	2000	2

Table 2. Bending test results from the VTT Research Report [8]. f_{gr} mean strength of grout, $f_{bolt,y}$ yield strength of bolt, b width of column section, A_{sp} area of threaded section of bolt, d_{up} and d_{low} effective depths of upper and lower bolts, M_t theoretical bending resistance of connection, and M_e experimental bending resistance of connection.

Test	f_{gr} MPa	$f_{bolt,y}$ MPa	b mm	A_{sp} mm ²	d_{up} mm	d_{low} mm	M_t kNm	M_e kNm	M_e/M_t
H16-BS.b	51.6	562	235	157	50	185	35.3	39.8	1.25
H16-BR	51.6	562	235	157	50	185	35.3	39.2	1.24
H24-BR.b	49.3	573	270	353	50	220	83.8	98.3	1.30
H39-BS.c	45.0	556	380	976	60	320	312.7	349.7	1.23
H39-BR	48.2	556	380	976	60	320	314.7	349.3	1.25

BENDING RESISTANCE (BR) TESTS

These tests aimed to confirm that the HPKM Column Shoe connection has at least an equal bending resistance as a cast-in-situ column. The tests were carried out in 2012 [8].

The experimental bending resistance M_e and the theoretical bending resistance M_t based on the measured material properties and nominal geometry as well as the ratio of the experimental to theoretical bending resistance are given in Table 2.

The characteristic value of moment ratio M_t/M_e , calculated in accordance with CUAP, equals 1.19 or 1.20 when the coefficient of variation is regarded as unknown or known, respectively. This means that the applied design method is safe for axial and bending resistance of HPKM Column Shoes.

BENDING STIFFNESS RESISTANCE (BRS) TESTS

The bending stiffness of bolted precast column connections was perhaps the most important and interesting subject of the ETA tests. The design according to ETA-13/0603 is based on Eurocode 2 (EC 2, which is standard EN 1992-1-1) which in turn assumes purely theoretical values for buckling lengths. The aim is to apply the design rules of EC 2, developed for slender cast-in-situ columns with continuous reinforcement, to precast columns with column shoe connections. In a column shoe connection the stiffness of the joint is lower than that of the column, but the real stiffness of the cast-in-situ column connection is not known. Figure 2 illustrates the different stiffness zones in precast and cast-in-situ columns.

The precast column A is compared with a reference column B which is cast-in-situ and continuously reinforced with the same main reinforcement as the precast column, see Fig. 2. It has to be shown that:

- The bending resistance of the column shoe connection is at least as high as that of the reference column
- The stiffness of the precast column with column shoes is at least as high as that of the reference column

The latter requirement is important for slender columns because their load-bearing resistance depends on the stiffness of the connections. The basic idea is to compare the stiffness of a precast column with that of the reference column, not to draw conclusions about the absolute stiffness values.

In **Zone 1** the column shoes have no effect on the stiffness. In **Zone 2** the flexural stiffness is very high due to the overlapping of the anchor bars of the column shoes and the main reinforcement of the column. In **Zone 3** the flexural stiffness is

low due to the reduced effective concrete section at the end of the column and eccentric tension in the column shoes.

Assume that the bending stiffness outside the column shoe zone is (EI). The stiffness of the lower part of the column shoe zone is typically lower than (EI), say $\alpha \cdot (EI)$ ($\alpha < 1.0$), and that of the upper part of the column shoe zone is $\beta \cdot (EI)$ ($\beta > 1.0$). This is different from a cast-in-situ column shown in Figure 2 in which the reinforcement is continuous and constant over the entire height of the column. Column B is chosen for reference because the design rules in EN 1992-1-1 were developed for this case and, even though a spliced connection may be more common in practice, the design codes do not require splicing. The tests also focused on finding the real stiffness $\varphi \cdot (EI)$ of a cast-in-situ column connection in zone 3 ($\varphi < 1.0$?).

There is a wide variety of load cases and boundary conditions for a given column cross-section, column shoe, and column height. To ensure that conclusions on the connection's performance are safe, the focus must be the cases in which the stiffness of the column shoe connection plays an important role and in which the risk of overestimating the resistance is highest. A mast (cantilevered column) was chosen for the structure to be examined because:

- Its behavior is sensitive to the geometric nonlinearity that makes stiffness an important issue
- Masts emphasize the negative mechanical consequences of a flexible connection

The bending stiffness of a section typically increases with increasing axial force. Therefore, to ensure safe conclusions are drawn, the stiffness is measured without axial force and the deflection of the mast column is compared in a load case without axial force. This enables a conservative (probably overconservative) evaluation method to be obtained.

Two stiffness tests on column type A, H16-BS and H39-BS, were carried out (see Table 3). In 2009, stiffness test PV380 on column type B was also carried out for reference. The axial strain on the top and at the bottom of the bended test specimen was determined by measuring the differential displacement by horizontal transducers as shown in Fig. 3. Pursuant to the design principles of ETA-13/0603, 90% of the nominal yield resistance of the connection can be exploited.

Figure 2. Different stiffness zones of cantilever column.

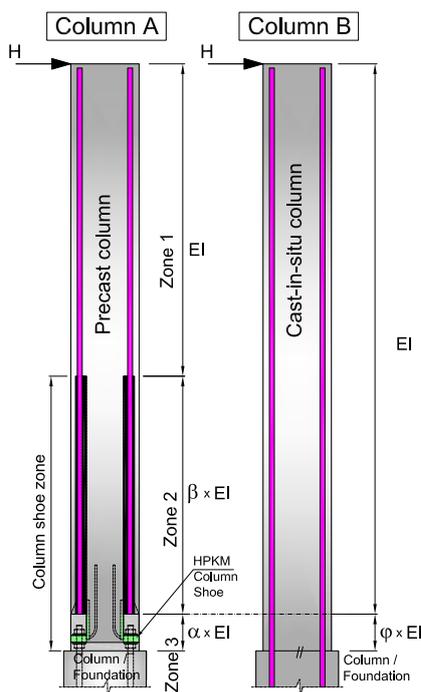


Figure 3. Illustration of full-scale bending stiffness resistance (BRS) test arrangements.

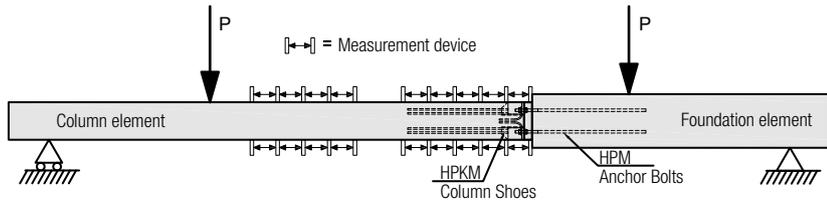
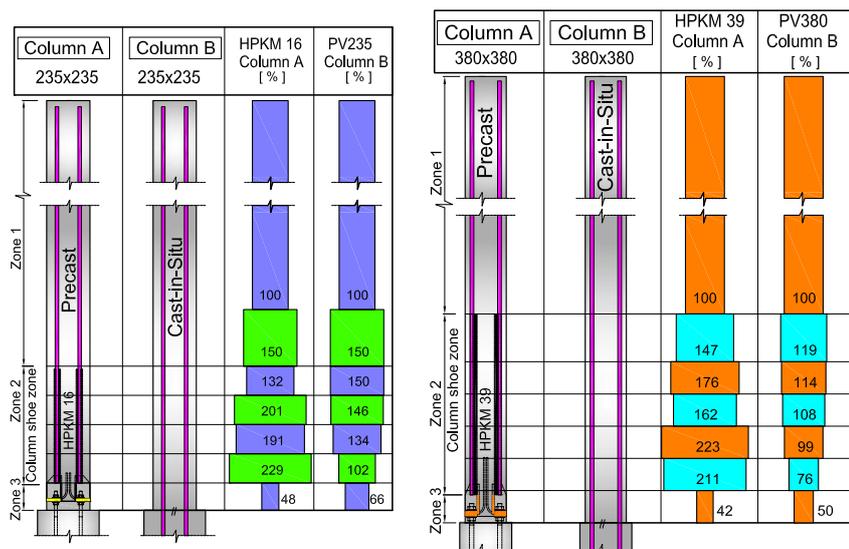


Table 3. Column shoe types, anchor bolts, and sizes of test specimens in stiffness tests.

Column Shoe Type	Anchor Bolt Type	Size of column element		Size of foundation element		Number of tests
		h x b (mm ²)	Length (mm)	h x b (mm ²)	Length (mm)	
HPKM 16	HPM 16/P	235 x 235	3200	335 x 335	1850	1
HPKM 39	HPM 39/P	380 x 380	5400	480 x 480	2500	1
Cast-in-Place column PV380		380 x 380	5260	450 x 450	3000	1

The relative stiffness of the subzones of the column are given in Fig. 4 according to the VTT Research Report [8].

Figure 4. Application of test results to columns A and B. Relative bending stiffness of subzones (%).



When analyzing the test results, the relative stiffness of precast column A and cast-in-situ column B in zone 1 are naturally equal (100%). In zone 2, and especially in its lowest subzone, the relative stiffness of precast column A is significantly higher (229% and 211%) than that of the cast-in-situ column B (102% and 76%). The relative stiffness of the cast-in-place column B and its zone 3 is lower than zone 2, which means that factor $\varphi < 1.0$ (see Fig. 2 too). The relative stiffness of the column shoe connection in zone 3 is lower than in upper zone 2. Although the relative stiffness of zone 3 of the precast column A is lower than that of the cast-in-situ

column B, the difference is minor. When compared with column B, the stiffer zone 2 of column A will compensate for the weaker stiffness in zone 3. The calculated deflections at the top of the columns, based on the measured test data, were 380.7 mm and 450.2 mm for HPKM 39 and PV380, and 125.7 mm and 129.7 mm for HPKM 16 and PV235, respectively. In both cases column A with column shoes is stiffer than column B. Hence, in accordance with CUAP [11] and ETA-13/0603 [7], all HPKM Column Shoes are classified to stiffness class B50, which means that the column is designed assuming a rigid column shoe connection. The tests confirmed that the

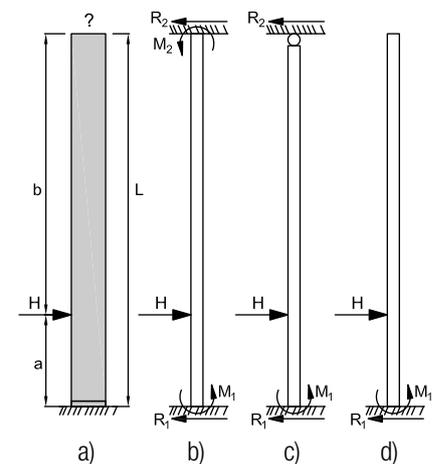
bolted connection with a grouted joint between the precast column and base structure behaves in the same way as a corresponding monolithic reinforced cast-in-place column.

SHEAR TESTS

It is assumed that the maximum shear is caused by a moving vehicle which collides with a single column after hardening of the grout in the joint. For such an impact it is sufficient to consider a one-story-high zone above the column shoe level and below the next upper floor or roof.

The beams and floors carried by the column redistribute the reaction forces and moments at the top of the considered column section effectively. Therefore, the upper end of the column can be regarded as laterally fixed. The flexural rigidity of the upper end is between completely rigid and hinged, see mechanical models B and C in Fig. 5. For ordinary storey heights and vehicle collisions, the forces at the bottom (R_1 and M_1) are only slightly influenced by the bending stiffness at the top. Therefore, mechanical model C was adopted when designing the layout for the shear tests (see Fig. 6).

Figure 5. a) Section of column, rigidity of upper end unknown. b) Mechanical model B. c) Mechanical model C. d) Mechanical model D.



It is assumed that the axial force in the considered case (shear due to accidental actions, vertical loads according to the serviceability limit state) has little or no negative effect on the shear resistance and can be ignored in the test arrangements. Due to its temporary nature, the erection stage (joint not grouted) was not covered by shear tests.

In each shear test V_e , the highest shear force observed in the test, shall meet

Figure 6. Illustration of full-scale shear test arrangements.

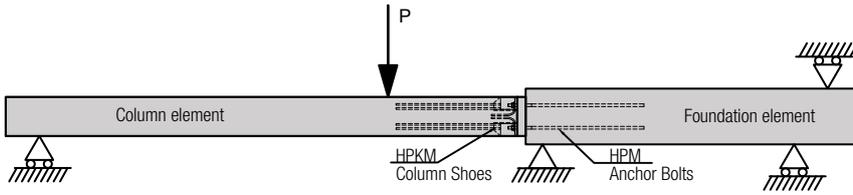


Table 4. Column shoe types, anchor bolts and size of tests specimens.

Column Shoe Type	Anchor Bolt Type	Size of column element		Size of foundation element		Number of tests
		h x b (mm ²)	Length (mm)	h x b (mm ²)	Length (mm)	
HPKM 16	HPM 16/P	235 x 235	3800	335 x 335	2000	1
HPKM 39	HPM 39/P	380 x 380	3800	480 x 480	2000	1

the requirement $V_e \geq 1.15 \cdot V_t$ acc. to CUAP [11], where V_t is the sum of the theoretical shear resistances of two active column shoes calculated according to EN 1993-1-8, Chapter 6.2.2 [4] using safety factor $\gamma_{M2} = 1.0$ and the measured yield strength of the anchor bolt. As shown in Table 5, the requirement was met in both shear tests [8].

Table 5. Theoretical shear resistance V_t and highest shear force in test V_e .

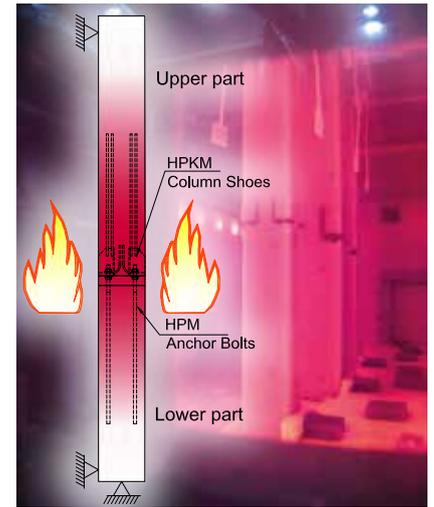
Test	V_t kN	V_e kN	V_e/V_t
H16-S	54.8	84.2	1,54
H39-S	376.5	444.2	1,18

H16-S is the shear test with HPKM 16 Column Shoes and column size 235 x 235 mm². H39-S is the shear test with HPKM 39 Column Shoes and column size 380 x 380 mm².

FIRE TEST

Test results were needed to evaluate fire resistance. The fire tests aimed to measure time-dependent temperatures within essential connection components. High temperatures ($t > 100$ °C) will weaken the mechanical properties of the steel and concrete and reduce the mechanical resistance of precast column connections. A fire resistance test was carried out in a horizontal furnace at the testing laboratory of VTT Expert Services Ltd. There were three test columns standing on the furnace floor. They were horizontally supported at the top of the columns (Fig. 7). Each test column was made of precast reinforced concrete and comprised two parts connected by HPKM Column Shoes and HPM Anchor Bolt

Figure 7. Fire tests arrangements.



Bolts. The column shoe types, bolt types, and test specimen dimensions are presented in Table 6.

Fire tests were made for the smallest (HPKM 16), medium-sized (HPKM 24), and largest (HPKM 39) column shoe types according to CUAP [14]. The temperature development of intermediate column shoe types HPKM 20 and HPKM 30 were evaluated from the numerical results using the finite element method (FEM) [13]. Only the minimum sizes of column sections for standard column shoes were

tested to ensure safe results (see Table 6). Temperature changes in bigger column sections will be very similar but the heat flow will be slower due to the higher heat absorption capacity. The test method was in accordance with standard EN 1365-4 [5]. The test was performed without mechanical loading and the outer edges of the column shoes were exposed to fire. The fire test was terminated after 130 minutes due to the very high temperature ($t > 1000$ °C) of the furnace and specimens.

Figure 8. Measuring points of column shoe B [9].

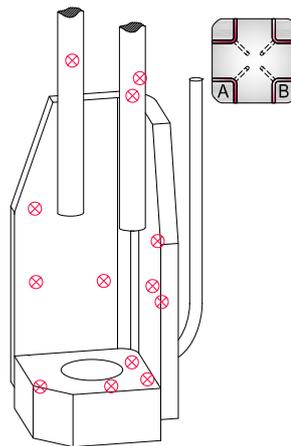


Figure 9. Measuring points of anchor bolts [9].

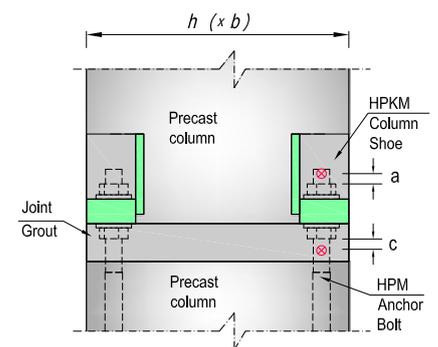


Table 6. Column shoe types, anchor bolts, and sizes of test specimens.

Column Shoe Type	Anchor Bolt Type	Size of lower part		Size of upper part		Number of tests
		h x b (mm ²)	Length (mm)	h x b (mm ²)	Length (mm)	
HPKM 16	HPM 16/P	235 x 235	1500	235 x 235	1000	1
HPKM 24	HPM 24/P	270 x 270	1500	270 x 270	1000	1
HPKM 39	HPM 39/P	380 x 380	1500	380 x 380	1000	1



Table 7. Average temperature t [°C] in critical section of anchor bolt [7].

Time [min]	HPKM 16	HPKM 20	HPKM 24	HPKM 30	HPKM 39
60	500	500	450	430	390
90	670	610	630	630	570
120	800	780	740	730	700

Both the experimental and FEM results very clearly suggested that the critical temperatures were in the anchor bolts on both sides of the base plate (see Fig. 9) and not in the anchor bars, base plate, and side plate of the column shoe [13]. For this reason the measured temperatures of the anchor bolts are used in fire design. The resulting temperatures to be used in fire design in accordance with the relevant Eurocodes are given in Table 7.

RESISTANCES AND DESIGN OF COLUMN CONNECTIONS ACCORDING TO ETA-13/0603

Peikko Group aims to offer easy and quick-use design tools for users. Manually designing cantilevered slender columns and columns with biaxial bending moments can be complex. The Peikko Designer® software and its Column Connection module is based on a column section design concept. The module covers N-M-interaction curves, shear design, fire design, and design for the erection stage.

RESISTANCE AGAINST AXIAL FORCE AND BENDING MOMENT

It has been verified in the initial type testing that precast columns with HPKM connections can safely be designed using the design rules developed for continuously reinforced cast-in-situ columns. The design rules are in accordance with EN 1992-1-1 [1] and take into account the behavior of the completed connection and its components (Fig. 10). National Annexes (NA) for Eurocodes must be taken into account. The resistance of the grouted section (joint) above the foundation and below the column shoes is calculated according to EN 1992-1-1 assuming that the section behaves as a concrete section reinforced with the anchor bolts.

Symbols:

- N_{Ed} = design value of axial force
- M_{Ed} = design value of bending moment
- h = length of column section
- b = width of column section
- d' = edge distance of bolts
- z = lever arm of bolts
- x = length of compression zone
- y = effective length of compression zone
- λ = parameter of compression zone acc. to EN 1992-1-1, Eq. 3.19
- $\epsilon_{s,i}$ = ultimate strain of bolt steel
- $\epsilon_{c,i}$ = ultimate strain of grout
- $N_{S,i,d}$ = force of anchor bolts
- $N_{C,d}$ = compression force of grout

The axial force-bending moment N-M-interaction curve of the HPKM Column Connection is calculated according to EN 1992-1-1 as follows:

Stress values in design:

$$\sigma_{sd,1} = \min \left\{ \epsilon_{s,1} \cdot E_s ; \eta_d \frac{f_{bolt,y}}{\gamma_{M2}} ; \frac{f_{bolt,u}}{\gamma_{bolt}} \right\}$$

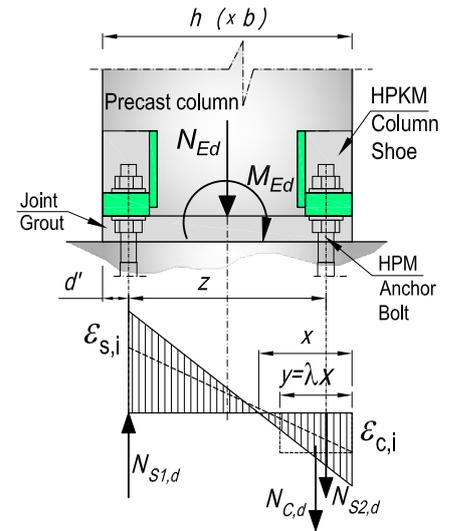
$$\sigma_{sd,2} = \min \left\{ \epsilon_{s,2} \cdot E_s ; \eta_d \frac{f_{bolt,y}}{\gamma_{M2}} ; \frac{f_{bolt,u}}{\gamma_{bolt}} \right\}$$

$$\sigma_{gr,cd,i} = \min \left\{ \epsilon_{c,i} \cdot E_c ; f_{cd,EC2} \right\} ; \epsilon_{c,i} \leq 3,5\text{‰}$$

Where $\sigma_{sd,i}$ is the stress in the anchor bolt, E_s the elastic modulus of the anchor bolt, η_d the reduction factor ≤ 0.90 , (the value is determined in initial type testing), $f_{bolt,y}$ the yield strength of bolt steel, $f_{bolt,u}$ the ultimate strength of bolt steel, γ_{M2} the material safety factor for the bolt according to EN 1993-1-1, Chapter 6.1 [3], γ_{bolt} the material safety factor for the bolt according to the relevant European Technical Approval ETA-02/0006 [6], $\Sigma A_{bolt,i}$ the total threaded area of bolts in tension or in compression, and $\sigma_{gr,cd,i}$ the design value of compressive stress of grout.

In addition to axial loads (N_{Ed}), concrete columns may be subjected to bending moment ($M_{ed,x}$; $M_{ed,y}$). Although it is possible to derive equations to evaluate the resistances of column connections subjected to combined loading, the equations may be complex to use, especially with bi-axial bending. For this reason, N-M-interaction diagrams for columns are generally computed by assuming a

Figure 10. Design principle acc to EC 2.



Design values of forces:

$$\rightarrow N_{S,1d} = \sigma_{sd,1} \cdot \Sigma A_{bolt,i} \quad (1)$$

$$\rightarrow N_{S,2d} = \sigma_{sd,2} \cdot \Sigma A_{bolt,i} \quad (2)$$

$$\rightarrow N_{C,d} = \sigma_{gr,cd,i} \cdot A_c \quad (3)$$

series of linear strain distributions, each corresponding to a particular point on the interaction diagram, and computing the corresponding values of $N_{Rd,i}$ and $M_{Rd,i}$. Once a sufficient number of points have been computed, the results are summarized in an interaction diagram. Since these points can only be solved iteratively, it is recommended that the Peikko Designer® software be used.

SHEAR RESISTANCE

The experimentally verified design method is in accordance with EN 1993-1-8 [4]. The design value of the shear force for a single column shoe on the active side (see Figure 11) is calculated as follows:

$$V_{Ed}^1 = \frac{V_{Ed} - \mu \cdot N_{Ed}}{n} \quad (4)$$

where V_{Ed} is the total shear force on the column connection, N_{Ed} the axial force on the column connection (if the column is

subject to a tensile axial force, $\mu \cdot N_{Ed} = 0$), n is the number of the individual active column shoes resisting the shear force and μ is the friction coefficient between base plate and grout ($= 0.20$ according to EN 1993-1-8, Chapter 6.2.2 [4]). The shear resistance V_{Rd} of a column shoe subjected to shear and compression must meet the requirement:

$$V_{Ed}^1 \leq V_{Rd} \quad (5)$$

The shear resistance of a single column shoe (see Table 8) is calculated according to EN 1993-1-8, Chapter 6.2.2, as follows:

$$V_{Rd} = \frac{\alpha_b \cdot f_{bolt,u} \cdot A_{bolt}}{\gamma_{M2}} \quad \text{where}$$

$$\alpha_b = 0.44 - (0.0003 \text{MPa}^{-1}) f_{bolt,y} \quad (6)$$

where $f_{bolt,u}$ is the ultimate strength of anchor bolt steel, A_{bolt} the tensile stress area of the anchor bolt, γ_{M2} the material safety factor for the bolt according to EN 1993-1-1, Chapter 6.1 and $f_{bolt,y}$ the yield strength of bolt steel.

Table 8. Design values of shear resistance of individual HPKM Column Shoe.

	HPKM 16	HPKM 20	HPKM 24	HPKM 30	HPKM 39
V_{Rd} [kN]	20.0	31.3	45.0	71.6	124.5

The minimum torque value of nuts according to ETA-13/0603 [7] is required when applying the shear resistances given above.

FIRE RESISTANCE AND DESIGN

Since European Standard EN 1992-1-2 [2] presents only complex and insufficient fire design methods for manually designing slender cantilever columns, the fire design of column connections is implemented into Peikko Designer® to enable quick and easy design. The fire design of the Peikko Designer® Column Connection is based on measured temperatures of fire tests carried out by VTT and FE analyses. The tests and analyses were necessary because EN 1992-1-2 does not give any temperature contours for precast column connections. In addition, in EN 1992-1-2, temperature contours are only given to column size 300 x 300 mm², or to circular sections with diameter 300 mm.

In principal, the fire design is similar to the design of reinforced concrete column in normal temperature, see Fig. 10.

Figure 11. Shear resistance of HPKM Column Connection in the final stage, when the joint is grouted.

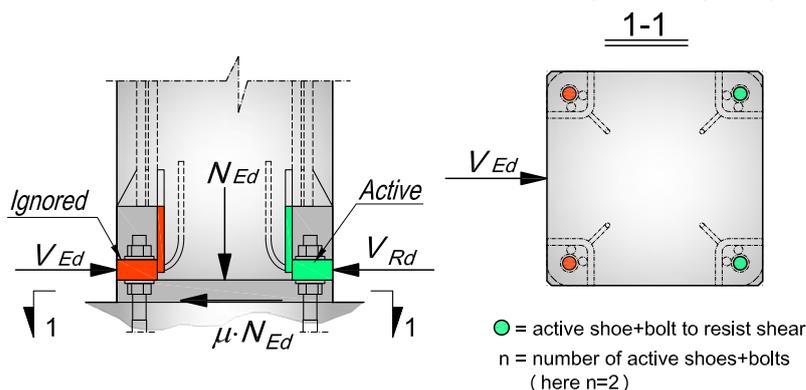
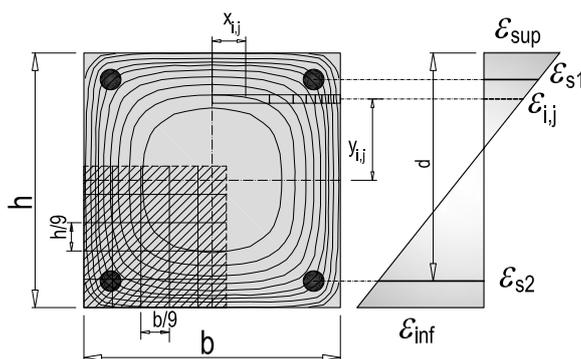


Figure 12. Design principle in fire exposure.



Peikko Designer® calculates the axial force–bending moment (N-M) resistance interaction curves for the given column connection cross section. When the strain values ϵ_{sup} and ϵ_{inf} at the edges are given (see Fig. 12) the strain in each element is fixed. When the temperature in each element is also known (see Fig. 13) the corresponding strain and temperature dependent stress values in each element can be calculated. From the known stress distribution the resulting axial force and bending moment are obtained. By varying ϵ_{sup} and ϵ_{inf} linearly, the resistance of the connection is obtained as an N-M-interaction curve.

The material strengths at each elevated temperature are defined according to EN 1992-1-2. In the connection's fire design a reduction factor, $\eta_d \leq 0.90$ [7], is also taken into account.

Figure 13. Example of temperature profile (°C) for a joint, $h \times b = 380 \times 380 \text{ mm}^2$.

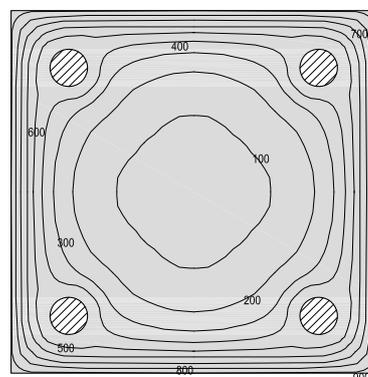


Figure 14. Erection stage.

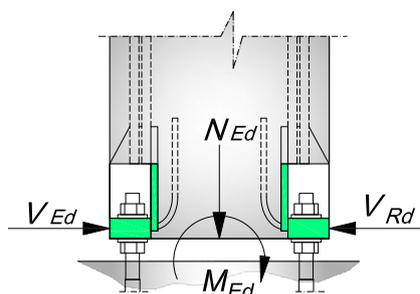
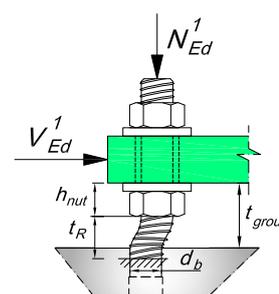


Figure 15. Notation for individual bolt.



ERECTION STAGE DESIGN

ETA-13/0603 also includes design rules for the erection stage when the joint is not grouted (see Figs. 14 and 15). Anchor bolts are the critical and decisive connection components in the erection stage.

Design is based on general stress calculation according to the following equation:

$$\frac{16|V_{Ed}^1|t_R}{\pi d_b^3} + \frac{4|N_{Ed}^1|t_R}{\pi d_b^2} \leq \min\left\{\eta_d \frac{f_{bolt,y}}{\gamma_{M2}}; \frac{f_{bolt,u}}{\gamma_{bolt}}\right\} \quad (7)$$

where V_{Ed}^1 is the shear load on a bolt, N_{Ed}^1 the axial load on a bolt (action effect) calculated from the total axial force N_{Ed} and bending moment M_{Ed} , d_b the diameter of nominal stress area in the thread of the anchor bolt, t_R the lever arm of the bolt = $(t_{grout} - h_{nut} + d_b/2)$, η_d with a reduction factor ≤ 0.90 , the value of which is determined in initial type testing, $f_{bolt,y}$ the yield strength of bolt steel, $f_{bolt,u}$ the ultimate strength of bolt steel, γ_{M2} the material safety factor for the bolt according to EN 1993-1-1, Chapter 6.1 and γ_{bolt} the material safety factor for the bolt according to the relevant European Technical Approval ETA-02/0006 [6]. The minimum torque value of nuts according to ETA-13/0603, Table 4 is applied.

CONCLUSION

ETA-13/0603 includes pioneering design rules for column shoe connections. The rules are in accordance with EN 1992-1-1 and EN 1992-1-2 (Eurocode 2) and take into account the behavior of the completed connection and its components. The rules for mechanical behavior and fire resistance have been verified by full-scale tests according to CUAP 03.02/06 [11]. Experimental concrete unit tests have confirmed that the stiffness of Peikko's column connection is at least as rigid as a continuously reinforced cast-in-situ column connection. HPKM Column Shoes and column connections fulfill the ETA-13/0603 requirements for mechanical, fire, and corrosion resistance, stiffness, shear resistance, and torque of nuts. Peikko has launched design software – Peikko Designer® – to facilitate column connection design. It can be downloaded free of charge from Peikko's website. ■

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