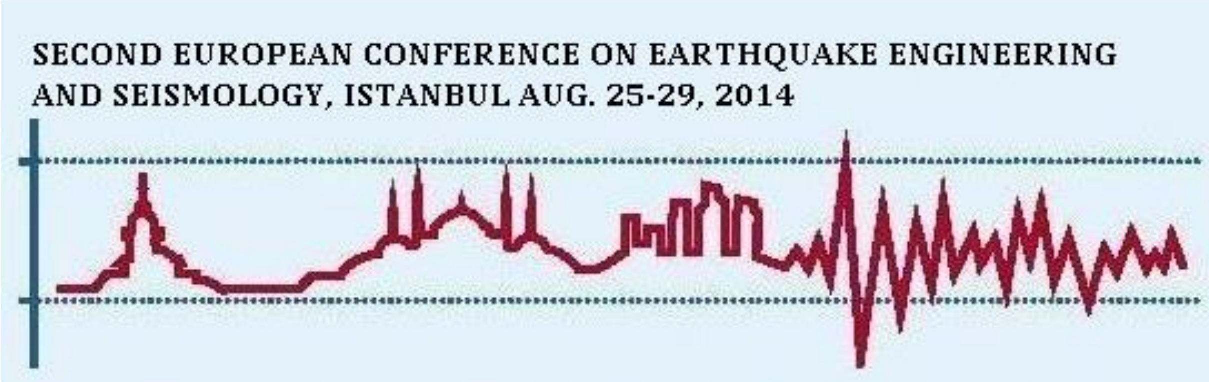


SEISMIC RESPONSE OF DOWEL CONNECTIONS IN PRECAST INDUSTRIAL BUILDINGS

Blaž ZOUBEK[1], Matej FISCHINGER[2], Tatjana ISAKOVIC[3]

ABSTRACT

The dowel connection is the most common type of connections used to join columns and beams in typical European industrial buildings. This type of connection should be designed according to the capacity design approach. In this way, the seismic energy dissipates in the columns without premature brittle failure of the connections. To achieve such behaviour of the structure, the strength of the dowel connections should be appropriately estimated.

To be able to estimate the strength of the dowel connection properly, it is essential to define the type of its failure first. In general there are two possible types of failure: (1) local failure due to the simultaneous crushing of concrete in front of dowel and yielding of the dowel (2) global failure due to the spalling of concrete, in front of the dowel and. The first type of failure is mostly observed in connections, where the dowel is well embedded into the connected elements), while the second type of failure typically occurs, when the dowel is relatively close to the edge of the connected elements (beams and columns).

In the paper, both failure modes are analysed and the appropriate procedures for the estimation of the real beam-column dowel connections in precast structures are proposed.

INTRODUCTION

Precast buildings have been frequently built in many European countries. Predominant type of these structures consists of an assemblage of columns tied together with beams (Fig. 1). Such buildings house a predominant share of industrial facilities in many European countries, while recently they are frequently used for multi-storey apartment buildings, shopping centres and other buildings where large numbers of people gather.

It is obvious that the behavior of precast systems depends heavily on the performance of the specific connections between precast elements. Different types of connections exist, however the pinned connection using steel dowel is the most common. The knowledge of the highly complex inelastic seismic behavior of such connections has been very limited. For this reason, the FP7 project SAFECAST - Performance of Innovative Mechanical Connections in Precast Building Structures under Seismic Conditions (Toniolo, 2012) was started in 2009. In the project, full-scale specimens of

[1] B. Sc., University of Ljubljana, Faculty of Civil and Geodetic Engineering, Ljubljana, Slovenia, blaz.zoubek@fgg.uni-lj.si
[2] Prof. dr., University of Ljubljana, Faculty of Civil and Geodetic Engineering, Ljubljana, Slovenia, matej.fischinger@fgg.uni-lj.si
[3] Prof. dr., University of Ljubljana, Faculty of Civil and Geodetic Engineering, Ljubljana, Slovenia, tatjana.isakovic@fgg.uni-lj.si

typical connections as well as prototype structures were experimentally and analytically investigated. The study related to the response of the dowel connections is presented in this paper.

Possible failure mechanisms of these connections are two: (a) local failure characterized by the simultaneous yielding of the dowel and crushing of the surrounding concrete (Vintzeleou and Tassios, 1986; Tanaka and Murakoshi, 2011; Zoubek et al. 2013) and (b) global failure, characterized by spalling of the concrete between the dowel and the edge of the column or the beam (Vintzeleou and Tassios, 1996; Psycharis and Mouzakis, 2012).

Local failure mechanism normally occurs if the dowel is placed relatively far from the concrete edge. This type of failure is in the majority of cases ductile. It was tested and analysed in many studies (e.g. those cited above); thus it is explained in more detail, than the global type of failure.

The global failure is more likely to occur, if the dowel is placed closer to the edge of the column or the beam (e.g. about six diameters of the dowel or less), due to the spalling of the concrete between the dowel and the edge. When there are no stirrups in the critical region, this failure will be brittle, since the capacity of the connection is governed by the tensile failure of the concrete between the dowel and the edge of the column or the beam. However, if there are stirrups placed around the dowel, the strength of the connection increases and the global failure mechanism changes from brittle to ductile.

The related experimental (Fuchs et al., 1995) and analytical studies (Vintzeleou and Tassios, 1996) on the global type of failure are quite rare. They are mostly performed on quite simple specimens or models. For example, in (Vintzeleou and Tassios, 1996) the important contribution of the stirrups to the capacity of the connection as well as to the type of failure was neglected, while in (Fuchs et al., 1995) it was considered only implicitly. For this reason the findings of these studies cannot be directly applied to beam-to-column dowel connection in precast buildings, since the stirrups around the dowel usually considerably influence the strength of the connection and change the global failure from brittle to ductile. In majority of practical cases, the procedures proposed in previously mentioned studies are very conservative, leading to impracticable design solutions (note that normally considerable seismic demand on the connection is obtained following the capacity design rule).

This paper examines both types of failure of the dowel connections but more attention is devoted to the less investigated global failure. The new procedure for estimation of the strength, which explicitly takes into account the contribution of the stirrups in the critical region around the dowel, is proposed. The procedure is based on the appropriate usage of the strut and tie model. Further on some modifications of the procedures, which are typically used to estimate the local resistance, are suggested.

The proposed procedures are applied on the specimens tested in the frame of the SAFECAST project at the University of Ljubljana (UL) and National Technical University of Athens.

Figure 1. The most common structural system of precast industrial buildings in Europe consists of an assemblage of slender cantilever columns, tied together by beams.

LITERATURE REVIEW

As already mentioned in the introduction, possible failure mechanisms of the beam-column dowel connection are two (Fig. 2): (a) local or (b) global failure mechanism. Local failure mechanism is characterized by the simultaneous yielding of the dowel and crushing of the surrounding concrete. In general, this type of failure is ductile. Global failure is characterized by spalling of the concrete between the dowel and the edge of the column or the beam. If there are no stirrups in the critical region in between the dowel and the edge of the column or the beam, this type of failure is brittle. Stirrups usually change the type of the failure to ductile as well as considerably influence the strength of the connection.

In the literature, experimental and analytical investigations of both failure modes can be found. Existing semi-empirical and analytical formulas for the estimation of capacity of the dowel connections are briefly overviewed in the following paragraphs for local and global type of failure, respectively.

Figure 2. Local ductile failure and global failure of a dowel connection

When the distance of the dowel from the edge of the column or the beam is relatively large (e.g. when this distance exceeds six diameters of the dowel) the local ductile failure of the connection is likely to occur. The concrete in front of the dowel is crushed. This allows the dowel to deform. Consequently a plastic hinge is formed at certain depth.

This failure mechanism is quite well investigated and presented in several studies (Engström, 1990; Vintezeleou and Tassios, 1986; Tanaka and Murakoshi, 2011; Zoubek et al., 2013). To predict the capacity of the connection, when the local failure is critical, several formulas are proposed in the following form (Engström, 1990; Vintezeleou and Tassios, 1986; Psycharis and Mouzakis 2012):

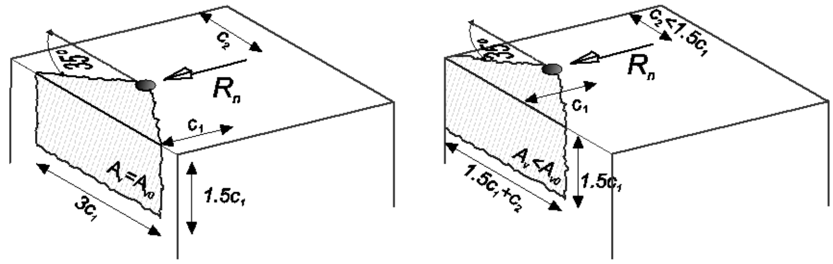
$$R_d = \alpha \, d_d^{\ 2} \sqrt{f_c \, f_{sy}} \, , \qquad (1)$$

where d_d is the diameter of the dowel, f_c is the uniaxial compressive strength of concrete, f_{sy} is the yield strength of the steel used for the dowel and α is a coefficient taking into account the increase of the strength of concrete due to spatial stress state and the eccentricity of loading. Eq. (1) was analytically as well as experimentally evaluated elsewhere (Vintezeleou and Tassios, 1986; Engström, 1990, Psycharis and Mouzakis, 2012).

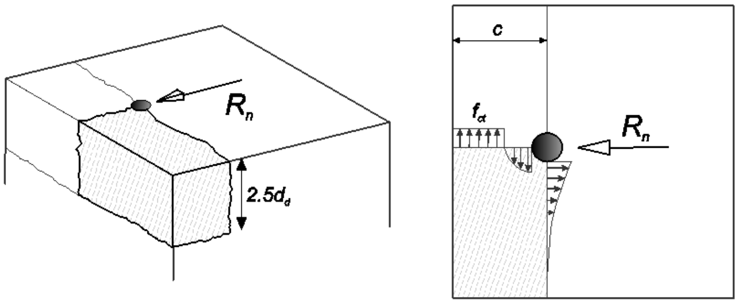
The global type of failure (characterized by spalling of the concrete between the dowel and the edge of the column or the beam) is expected to occur when the dowel is relatively close to the edge of the column or the beam. When there are no stirrups in the critical region around the dowel, the failure is brittle. It occurs when the principal tensile stresses exceed the tensile strength of the concrete. Usually, stirrups change the type of failure to ductile. They also considerably influence the strength.

Data about this type of failure are only few. The related capacity can be estimated by semi-empirical (Fuchs et al., 1995; CEN, 2009; ACI Committee, 2008) and analytical formulas (Vintzeleou and Tassios, 1986, 1987). Two of them (Fuchs et al., 1995; Vintzeleou and Tassios, 1986, 1987) are presented in the following paragraphs.

A comprehensive investigation of the steel fastenings in concrete elements was performed by Fuchs et al. (1995). Based on the extended experimental study, a concrete capacity design (CCD) method for the estimation of the concrete capacity of an individual anchor in a concrete structural member under shear loading towards the free edge was proposed (Fig. 3a). The method was later on (with some minor modifications) adopted by the CEN technical standard (CEN, 2009). More details about the procedure can be found in (Fuchs et al., 1995; CEN, 2009).



(a)



(b)

Figure 3. Crack pattern and failure mechanism (a) as proposed by Fuchs et al. (1995) and (b) as proposed by Vintzeleou and Tassios (1986)

Contrary to the Fuchs' proposal, Vintzeleou and Tassios (1986, 1987) solved the problem analytically. They assumed that the crack in a concrete element propagates from the dowel to the edge of the concrete section in the direction of loading, as it is demonstrated in Fig. 3b. Following the Vintzeleou's and Tassios' approach, the strength of the connection is estimated taking into account two cantilever beams, which are formed in front of the dowel. The shear strength of an eccentric dowel connection is limited by the flexural strength of these cantilevers (Fig. 3b). Vintzeleou and Tassios (1986, 1987) did not consider the stirrups situated in the critical region around the dowel. The importance of this reinforcement was observed in several experiments (DeVries et al., 1998; Zaghi and Saiidi, 2010; Psycharis and Mouzakis, 2012; Fischinger et al., 2012), and will be demonstrated in the next section.

The common conclusion for both presented procedures is that they are over conservative in the case of dowel connections in precast structures, leading to unfeasible and many times also unrealistic design solutions. When the presented procedures are used to estimate the strength of dowel connections in precast structures, the estimated value can be as small as the third of the actual strength.

PROPOSED PROCEDURES FOR THE ESTIMATION OF THE STRENGTH CAPACITY

Dowel connections, in which the dowel is placed close to the edge of the concrete elements, are susceptible to splitting of concrete between the dowel and the edge of the column or the beam. When there are no stirrups in the critical region around the dowel, the failure typically occurs due to exceeded tensile stresses (see Fig 1. (right) and Fig. 3(a)). The failure is brittle.

In the critical region around the dowel, precast elements are typically reinforced by a quite compact transverse reinforcement. For example, the axial distance between the stirrups is typically around 5 cm. Such reinforcement changes the stress field and typically changes the type of the failure of the connection from brittle to ductile. The influence of the stirrups to the strength of the connection depends on their amount. If the precast elements are reinforced by a relatively large amount of stirrups, the strength provided by stirrups will be typically larger than the tensile strength of the concrete itself. In such cases the strength of the connection increases after cracking of the concrete. However, if fewer stirrups are provided, the tensile strength of concrete can be larger from the strength of stirrups. In such cases the strength of the connection is typically reduced after cracking of the concrete. In the study, presented in this paper, it is considered that the global strength of dowel connection is provided by stirrups after cracking of the concrete (the contribution of the concrete to the strength is neglected). As it is discussed before, the strength defined in this way can be larger or smaller than the strength provided by the tensile strength of concrete.

Taking into account the crucial role of stirrups, a different approach from those, presented in the previous section, was applied to estimate the strength of dowel connections. The stirrups were considered explicitly, employing the strut and tie model as it is illustrated in Fig. 4.

To investigate the distribution of stresses in the eccentric dowel connections, a FE numerical model in ABAQUS (ABAQUS, 2011) was established (Zoubek et al., 2013), based on the experiments, performed in the frame of SAFECAST project and presented in (Psycharis and Mouzakis, 2012; Fischinger et al., 2012). Using this model, the equivalent trusses corresponding to various typical configurations of the dowel connections are defined (see Fig. 4).

In the first column of Fig. 4 typical configurations of the connections are presented. The related strut and tie model is shown in the second column. The third column presents the stresses, calculated by the FE analysis. In the last column of Fig. 4, closed expression of the strength capacity of dowel connections is given. This strength is defined as a force corresponding to the yielding of the first layer of stirrups. The complete utilization of the compression struts is connected with the local ductile failure mechanism which is described later on in the paper and is not considered in Fig 4.

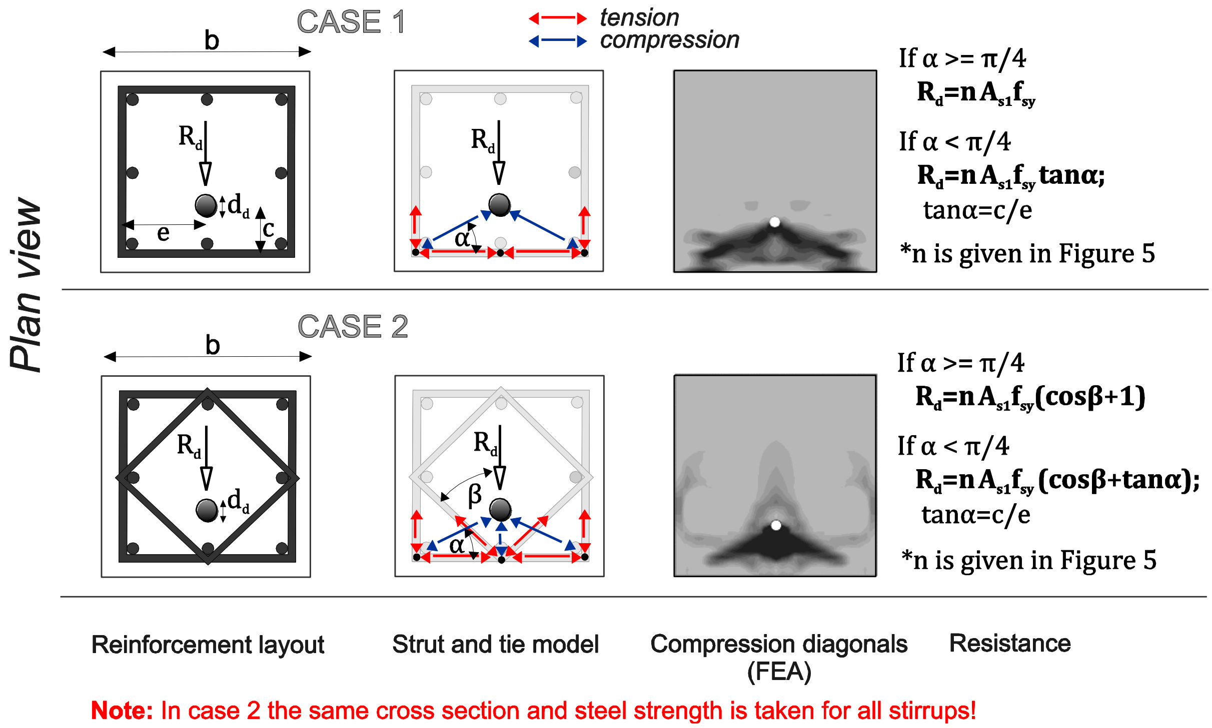


Figure 4. Proposal for the calculation of the resistance of the eccentric dowel connection for two possible reinforcement layouts often used in practice (for other cases please see Zoubek et al., 2014)

The critical region, where the rupture of the concrete is typically observed, is not limited to one cross-section of column or beam, but it is spread along certain length along the dowel. Consequently, more than one stirrups' layer is activated. All these stirrups influence the strength of the connection. Based on the FEM analysis and based on the experimental data, it has been observed that the height of the critical region can be defined as it is illustrated in Fig. 5.

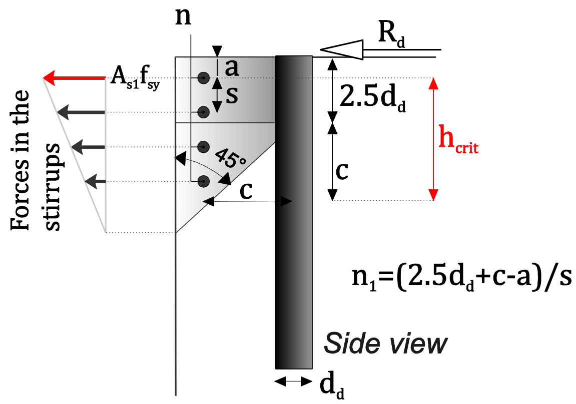


Figure 5. Schematic presentation of assumed distribution of stresses in the confining bars and the number of the activated stirrups n

Taking into account the typical shape of the ruptured concrete (see Fig. 3a) it can be assumed that the width of this region is approximately constant (see Fig. 5). Its depth, however, is changing as it is illustrated in Fig. 5. At the top of the column all over the height of 2.5 d_d (d_d is the diameter of the dowel) this depth is equal to the distance between the dowel and the axes of the stirrups. Then it is gradually annulled. Taking into account the results of the FEM analyses and experimental data, it has been observed that the depth is reduced almost linearly at an angle of 45° (see Fig. 5 for more details). Thus the total height of the critical region h_{crit} can be expressed as:

$$h_{crit} = 2.5\, d_d + c - a \tag{2}$$

where d_d is the diameter of the dowel, c is its distance of the dowel to the axes of the stirrups and a is the vertical distance of the first stirrup's layer from the top of the column.

The number of the engaged stirrups n can be defined, taking into account the vertical distance between stirrups (see Fig. 5) as:

$$n = h_{crit} / s + 1 \tag{3}$$

If the global resistance of the dowel connection is sufficient, the local failure mechanism, should be also checked. This failure mechanism is characterized by the local compression failure of the concrete in front of the dowel and simultaneous yielding of the steel dowel (Fig. 6). The local compression failure of concrete in front of the dowel means that the compression diagonals in Fig 4 are overstressed, since the compressive stresses in concrete are the highest at the contact with the dowel. Due to spatial stress state in concrete, according to the results of the numerical analysis (Zoubek et al. 2013), compressive strength of concrete can be increased up to 2-3 times compared to the uniaxial compressive strength f_c .

Assuming the failure mechanism, presented in Fig. 6, the depth of the plastic hinge a_u, which forms in the dowel, can be determined taking into account that the shear force in the dowel at depth au is zero. Considering the simultaneous failure of the dowel and the concrete in front of it, and considering the equilibrium of the moments at the top of the dowel, its flexural resistance can be expressed as:

$$M_{pl} = W_{pl}\, f_{sy} = d_d^{\,3} / 6\; f_{sy} = 3 f_c\, d_d\, a_u^{\,2} / 2 \tag{4}$$

where $M_{pl}= d_d^3/6\, f_{sy}$ is the plastic flexural resistance of the dowel, d_d is diameter of the dowel and f_{sy} is the medium yield strength of steel of the dowel and f_c is the uniaxial compressive strength of concrete. From the Eq. (4), one can easily obtain the depth of the plastic hinge a_u at failure:

$$a_u = 1/3\, d_d \sqrt{f_{sy} / f_c}\ , \tag{5}$$

Then the resistance of the dowel can be calculated by simple integration of stresses in front of the dowel to the depth of the plastic hinge:

$$R_{d\,u} = 3\ f_c\ d_d\ a_u = d_d^{\ 2}\ \sqrt{f_c\, f_{sy}} \tag{6}$$

If two dowels are used instead of one, the resistance is simply doubled.

Eq. (6) is of the similar form as Eq. (1), which has been already evaluated and confirmed in several investigations, including (Psycharis and Mouzakis 2012):

$$R_{d\,u} = 1.1\ d_d^{\ 2}\ \sqrt{f_c\, f_{yk}} \tag{7}$$

for small relative rotation between the adjoining beam and the column. Note that in Eq. (7), characteristic value of yield strength of steel is used. If $f_{yk}=f_{sy}/1.15$, than the formula transforms to:

$$R_{d\,u} = 1.03\ d_d^{\ 2}\ \sqrt{f_c\, f_{sy}}\ , \tag{8}$$

which is almost the same as the Eq. (6).

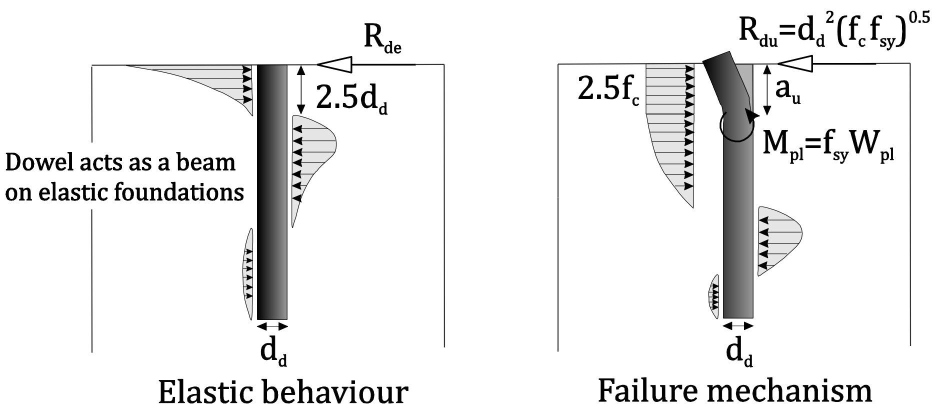


Figure 6. Elastic behaviour of the dowel and the failure mechanism – simultaneous failure of the dowel and the concrete in front of the dowel

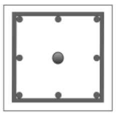
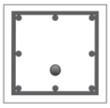
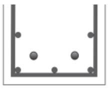
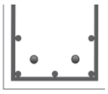
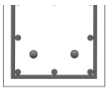
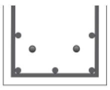
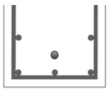
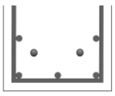
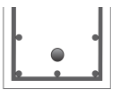
APPLICATION OF THE PROPOSED PROCEDURE

Analytical procedures, proposed in previous section, were evaluated by means of the experiments, performed in the frame of the SAFECAST project (Table 1). For detailed description of the experiments please see (Psycharis and Mouzakis, 2012; Fischinger et al., 2012).

Both failure mechanisms were analysed and the corresponding resistance was defined. Taking into account the smaller value of strength, the critical failure mechanism was identified. The analytical estimation of the resistance was then compared with the experimental results (Table 2 and Fig. 7).

In Table 2, analytically estimated strengths and failure mechanisms are compared with the values observed during the experiments. Failure mechanisms were estimated quite well. In the majority of cases the analytically predicted strength is slightly smaller than the measured values. However the difference is in the order of the accuracy of the input parameters. The largest difference between the analysis and experiment is obtained for specimen 1D25d10. The analysis underestimated the resistance for 23%. The predicted strength was 54kN while the measured value was 70kN.

Table 1: An overview of the specimens tested at UL and NTUA

Specimen	Plan view (scheme)	Dowel(s)	Reinforcement layout according to Fig. 2--	Material strengths* [MPa]	Stirrups	e** [cm]	c** [cm]	a** [cm]
S1-2		1ϕ28	case 1	f_{cm} = 50MPa f_{ym} = 580MPa f_{sym} = 560MPa	ϕ10/4cm	21.5	21.5	2.5
S6-2		1ϕ28	case 1	f_{cm} = 50MPa f_{ym} = 580MPa f_{sym} = 560MPa	ϕ10/4cm	21.5	9.0	2.5
S7-2		2ϕ25	case 3	f_{cm} = 50MPa f_{ym} = 540MPa f_{sym} = 560MPa	ϕ8/5cm	6.5	7.0	7.0
2D25d10		2ϕ25	case 3	f_{cm} = 35MPa f_{ym} = 580MPa f_{sym} = 560MPa	ϕ12/5cm	6.5	6.5	7.0
2D25d15		2ϕ25	case 3	f_{cm} = 30MPa f_{ym} = 580MPa f_{sym} = 560MPa	ϕ12/5cm	6.5	11.5	7.0
2D25d20		2ϕ25	case 3	f_{cm} = 30MPa f_{ym} = 580MPa f_{sym} = 560MPa	ϕ12/5cm	6.5	16.5	7.0
1D25d10		1ϕ25	case 1	f_{cm} = 35MPa f_{ym} = 580MPa f_{sym} = 560MPa	ϕ12/5cm	16.5	6.5	7.0
2D16d10		2ϕ16	case 3	f_{cm} = 35MPa f_{ym} = 560MPa f_{sym} = 560MPa	ϕ12/5cm	6.5	6.5	7.0
1D32d20		1ϕ32	case 1	f_{cm} = 30MPa f_{ym} = 560MPa f_{sym} = 560MPa	ϕ12/5cm	16.5	16.5	7.0

* f_{cm} – measured mean uniaxial compressive strength of concrete; f_{ym} measured uniaxial yields strength of steel used for dowels; f_{sym} measured uniaxial yields strength of steel used for confinement

**e - distance between the centre of the perimeter hoops and the centre of the dowel in the direction perpendicular to loading; c - distance between the centre of the perimeter hoops and the centre of the dowel in the direction of loading; a - distance between the centre of the first perimeter hoop and the top (or bottom) surface of the concrete element under consideration

Table 2: Summary of experimental and analytical behaviour of specimens

Specimen	Direction of loading*	Failure mechanism (experiment)	Failure mechanism (analytical)	Resistance (experimental) [kN]	Resistance (analytical) [kN]
S1-2	Pull=Push	Local failure	Local failure	150	134
S6-2	Pull	Global failure	Global failure	95	81
S6-2	Push	Local failure	Local failure	120	134
S7-2	Pull	Global failure	Global failure	160	149
2D25d10	Pull	Global failure	Global failure	130	136
2D25d10	Push	Local failure	Local failure	200	178
2D25d15	Pull	Global failure	Local failure	175	165
2D25d15	Push	Local failure	Local failure	200	165
2D25d20	Pull	Global failure	Local failure	180	165
2D25d20	Push	Local failure	Local failure	200	165
1D25d10	Pull	Global failure	Global failure	70	54
1D25d10	Push	Local failure	Local failure	90	89
2D16d10	Pull=Push	Local failure	Local failure	70	72
1D32d20	Pull=Push	Local failure	Local failure	150	133

*”Pull” - direction towards the closer edge of the section; “Push” - opposite direction

Very good correlation between analytical and experimental values is illustrated graphically in Fig. 7, using the resistance ratio, defined as r = calculated resistance/actual resistance. Two groups of results are presented. The grey diamonds and circles represent resistance ratios, which correspond to analytical values, obtained taking into account mean values of concrete and steel strength. Circles indicate local failure, while diamonds indicate global failure. Mean material characteristics are defined based on the uniaxial compressive tests of concrete cylinders and uniaxial tensile test of steel bars. The prediction of the proposed procedure (Fig. 7) for mean material characteristics agrees very well with the experiment. The estimate of the mean of the resistance ratio is $\bar{r} = 0.92$ and standard deviation $S_r^* = 0.093$.

When the mean values of strengths were reduced to corresponding design values (defined according to Eurocode 2 (CEN 2005) and Eurocode 8 (CEN 2004b) standards; see Eqs. 9 and 10), the resistance ratios presented by red diamonds and circles were obtained. Design material characteristics were calculated as:

$$f_{cd} = f_{ck} / \gamma_c = (f_{cm} - 8MPa) / \gamma_c ; \gamma_c = 1.5 \tag{9}$$

$$f_{yd} = f_{yk} / \gamma_s = f_{yk} / 1.15 / \gamma_s ; \; \gamma_s = 1.15 \tag{10}$$

Taking into account the above expressions, the ratio between the design and mean strength calculated using formula 10 for the local failure is 1.54-1.65 $R_{du,m}$ for the concrete strengths given in Table 1:

$$R_{d\,u} = d_d^{\;2} \sqrt{f_{cd}\, f_{yd}} = d_d^{\;2} \sqrt{f_{cm} - 8MPa / 1.5 \cdot f_{ym} /(1.15)^2} = (1.54 \div 1.64) R_{du,m} \tag{11}$$

For the global failure the ratio is $1.15 \cdot \gamma_s$=1.32 and does not depend on the concrete strength, since the strength of the connection is limited by the yielding of stirrups.

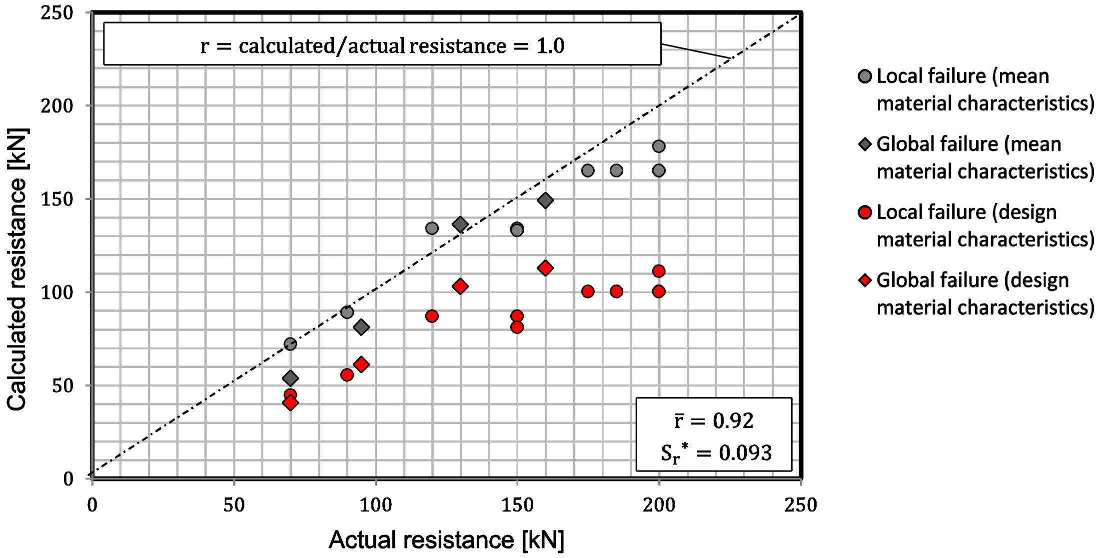


Figure 7: Comparison of the calculated and actual resistance of the dowel connections

CONCLUSIONS

The capacity and different types of failure of the beam-to-column dowel connection in precast industrial buildings was studied. Two types of failure were analysed: (a) local failure, characterized by the simultaneous yielding of the dowel and the crushing of the surrounding concrete and (b) global failure, characterized by spalling of the concrete between the dowel and the edge of the column or the beam.

When the distance of the dowel from the edge of the column or the beam is relatively large, the local ductile failure of the connection is likely to occur. Otherwise the global failure can be expected. The local failure mechanism is relatively well investigated and presented in several studies. Thus only some minor changes for the prediction of the related strength are proposed.

Global failure mechanism is less investigated. As a consequence, the existing procedures for the estimation of the related strength are in the majority of cases too conservative. The resistance can be underestimated as much as 3-4 times or more. The main reason is the contribution of the stirrups to the resistance of the connection, which is not sufficiently taken into the account. In the paper, a procedure for the estimation of the resistance against global failure is proposed. Taking into account an appropriate strut and tie model of the connections, the influence of the stirrups on the resistance as well as on the type of the failure is taken into account explicitly.

The comparison between the experiment and analytically calculated strength evidently demonstrates, that both proposed procedures - for the estimation of the resistance against local and global failure – agree very well with the experiment.

ACKNOWLEDGEMENTS

The presented research was supported by the SAFECAST project “Performance of Innovative Mechanical Connections in Precast Building Structures under Seismic Conditions” (Grant agreement no. 218417-2) in the framework of the Seventh Framework Programme (FP7) of the European Commission. Experiments of UL were realized in cooperation with the Slovenian National Building and Civil Engineering Institute (ZAG). The specimens were constructed at Primorje d.d. company. The research was partly supported by the Ministry of Education, Science and Sport of Republic of Slovenia.

REFERENCES

ABAQUS Theory Manual, version 6.11-3, Dassault Systèmes, 2011.

ACI 318-08. 2008. Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, Michigan, USA.

ACI Committee 318. 2008. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08). Farmington Hills, MI: ACI.

BS EN-1992-1-1: 2004. 2004. Eurocode 2, Design of Concrete Structures, General Rules and Rules for Buildings, British Standards Institution, London.

CEN. 2004a. Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings. CEN, Brussels.

CEN. 2004b. Eurocode 8: Design of structures for earthquake resistance – Part2: Bridges. CEN, Brussels.

CEN. 2005. Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings. CEN, Brussels.

CEN/TS 1992-1-1:2005, Design of fastenings for use in concrete - Part 4-2: Headed Fasteners, 89/106/EEC (No)

DeVries RA, Jirsa JO and Bashandy T (1998) "Effects of Transverse Reinforcement and Bonded Length on the Side-Blowout Capacity of Headed Reinforcement," Bond and Development Length of Reinforcement: A Tribute to Peter Gergely, American Concrete Institute, Farmington Hills, Mich.

Engström B. (1990) "Combined effects of dowel action and friction in bolted connections," *Nordic Concrete Research,* The Nordic Concrete Federation, 9:14-33

Fischinger M, Zoubek B, Kramar M, Isakovic T (2012) "Cyclic Response of Dowel Connections in Precast Structures," *15th World Conference on Earthquake Engineering*, Portugal, Lisbon, 24-28th September, 2012

Fuchs W, Eligehausen R, Breen JE (1995) "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACI Structural Journal*, 92(1): 73-94

Leonhardt F, (1975) Lectures in Concrete Structures – Second Part, Special Cases of Calculations. Springer-Verlag

Mander JB, Priestley MJN, Park R (1988) "Theoretical Stress-Strain Model for Confined Concrete," *Journal of Structural Engineering*, 114(8): 1804–1826

NZS 3101: 2006. 2006. Concrete Structures Standard. Part 1 - The Design of Concrete Structures and Part 2 - Commentary, Standards New Zealand, Wellington, New Zealand

Psycharis IN and Mouzakis HP (2012) "Shear resistance of pinned connections of precast members to monotonic and cyclic loading," *Engineering Structures*, 41: 413–427.

Tanaka Y, Murakoshi J (2011) "Reexamination of Dowel Behavior of Steel Bars Embedded in Concrete," *ACI Struct. J.*, 108(6):659-668

Toniolo G (2012). "SAFECAST Project: European research on seismic behavior of the connections of precast structures" *15th World Conference on Earthquake Engineering*, Portugal, Lisbon, 24-28th September, 2012

Vintzeleou EN, Tassios TP (1986) "Mathematical model for dowel action under monotonic and cyclic conditions," *Mag. Concr. Res.* 1986; 38:13-22.

Vintzeleou EN, Tassios TP (1987) "Behaviour of Dowels under Cyclic Deformations," *ACI Struct. J.*, 84(1): 18-30

Zaghi AE, Saiidi MS (2010) "Seismic Performance of Pipe-Pin Two-Way Hinges in Concrete Bridge Columns," *J. Earthquake Eng.*, 14(8):1253-1302

Zoubek B, Fahjan J, Isakovic T, Fischinger M (2013) "Cyclic Failure Analysis of the Beam-to-Column Dowel Connections in Precast Industrial Buildings Engineering Structures," Engineering Structures; 52:179-191

Zoubek B, Fischinger M, Isaković T (2014) "Estimation of the Cyclic Capacity of Beam-to-Column Dowel Connections in Precast Industrial Buildings," submitted for possible publication in Bulletin of Earthquake Engineering.