Structural Engineering and Mechanics, *Vol. 66, No. 2 (2018) 203-215*
DOI: https://doi.org/10.12989/sem.2018.66.2.203

Development and evaluation of punching shear database for flat slab-column connections without shear reinforcement

Shahram Derogar*[1], Ceren Ince[2] and Parthasarathi Mandal[3]

[1]*European University of Lefke, Faculty of Engineering, Department of Civil Engineering, Lefke, Northern Cyprus, TR-10 Mersin, Turkey*
[2]*Civil Engineering Program, Middle East Technical University, Northern Cyprus Campus, Kalkanli, Guzelyurt, North Cyprus, via Mersin 10, Turkey*
[3]*School of Mechanical, Aerospace and Civil Engineering, The University of Manchester, Manchester, M13 9PL, United Kingdom*

(Received April 17, 2017, Revised January 22, 2018, Accepted February 9, 2018)

Abstract. A large body of experiments have been conducted to date to evaluate the punching shear strength of flat slab-column connections, but it is noted that only a few of them have been considered for the development of the ACI Code provisions. The limited test results used for the development of the code provisions fall short of predicting accurately the punching shear strength of such connections. In an effort to address this shortfall and to gain an insight into the factors that control the punching shear strength of flat slab-column connections, we report a qualified database of 650 punching shear test results in this article. All slabs examined in this database were tested under gravity loading and do not contain shear reinforcement. In order to justify including any test result for evaluation punching shear database, we have developed an approved set of criteria. Carefully established set of criteria represent the actual characteristics of structures that include minimum compressive strength, effective depths of slab, flexural and compression reinforcement ratio and column size. The key parameters that significantly affect the punching shear strength of flat slab-column connections are then examined using ACI 318-14 expression. The results reported here have paramount significance on the range of applicability of the ACI Code provision and seem to indicate that the ACI provisions do not sufficiently capture many trends identified through regression of the principal parameters, and fall on the unsafe side for the prediction of the punching shear strength of flat slab-column connections.

Keywords: punching shear; flat slab-column connections; test; regression

1. Introduction

A flat plate structural system consists of a slab with uniform thickness supported directly on columns without any beams. Flat plates provide architectural flexibility due to the absence of beams, more clear space, less total building height, easier formwork, and consequently shorter construction time. They also are economical structural systems for medium height buildings. However, flat plates can suffer from brittle punching failure that occurs due to transfer of shear force and unbalanced moment between slabs and columns, and can seriously impair the structural performance of these systems. Despite research progress and code improvements, predicting the punching shear strength of slab-column connections in flat plate structures is among concerns of both the research community and field designers.

There are numerous experimental studies conducted to predict the shear strength of flat slab-column connections without shear reinforcement. For the most part, the proposed relationships designed to fit the limited set of test results and design requirements are empirical. Moreover, the code provisions often do not take into account the key factors that affect the performance of these connections. For instance, the ACI 318-14 provisions to predict punching shear strength of flat slab-column connections developed from the work by Moe (1961) fall into this pattern. In addition, ACI provisions only consider the concrete compressive strength as the main parameter to predict the punching shear strength and ignore the other key factors such as flexural reinforcement ratio and slab effective depth that play a significant role in the prediction of shear strength of flat slab-column connections.

For the nominal concentric punching shear capacity of two-way reinforced concrete slabs, ACI 318 has been using the equation $V_n = 0.33\sqrt{f_c'}b_o d$ for almost 60 years now. (This expression is discussed in detail in Section 5.2). Although ACI Committee 326 (Shear and Diagonal Tension) provided recommendations for this expression which was initially addressed in the 1963 code, it must be emphasise that this expression is solely based on the subtle modifications of the design procedure developed and reported in Moe (1961). It is worth mentioning that Kinnunen and Nylander (1960) provided significant knowledge and understanding to the punching shear modelling with their research work published in 1960, which immediately after encouraged the other researchers to further develop the punching models. This work therefore is regarded as the basis of the development of the models to predict the behaviour of slab-column connections

*Corresponding author, Assistant Professor
E-mail: sderogar@eul.edu.tr

ISSN: 1225-4568 (Print), 1598-6217 (Online)

under punching shear action in the research media.

The first report associated with punching shear strength was published by Talbot (1913) almost a century ago. Since then, a large number of experimental investigations on punching shear problem for various purposes have been published. Loo and Chiang (1993) assessed the code provisions using series of experiments on punching shear conducted at the University of Wollongong in 1990s. Gardner (1996) established a data bank of 142 tests on punching shear of slabs. Later in 2001, CEB FIP Bulletin 12 (2001) published a data bank that contained 250 tests on slabs. Hamada *et al*. (2008) published the largest data bank that comprised 300 tests on slabs without shear reinforcement. His data bank consisted data from Bulletin 12 and more than 130 other experiments that published in the Japanese language. Hafidi *et al.* (2013) published a databank which comprised of 280 tests which mostly cover the databank from CEB FIP bulletin 12 as well as new experiments from literature. These studies; notwithstanding, there is still a need to construct a larger database with more carefully assigned criteria both for the evaluation of the existing ACI provisions and to gain an insight into the key parameters that significantly influence the prediction of punching shear strength of flat slab-column connections.

The conversion factors of concrete compressive strength for different control specimens are provided that are used to construct the database in this paper. *Collection of punching shear database* (CPSD) is established using the data from the literature as they were reported by the researchers and formed the development of the bases of the *Evaluation punching shear database* (EPSD). CPSD is then carefully evaluated using the assigned set of criteria in order to move a given test result for inclusion in EPSD. The methodology used for the development of EPSD used to assess the test data with the ACI code provision in this paper is mainly adopted from the work reported in Reineck *et al*. (2003, 2013). The major focus of this paper is therefore on the development and the evaluation of punching shear database (EPSD) to assess the test data with the ACI code provisions using the regressed key parameters that affect the prediction of the punching shear strength of flat slab-column connections. The parameters investigated in this paper are the concrete compressive strength, slab depth, slenderness ratio and steel reinforcement ratio. The narrative in the paper is concluded with an invitation for participating in this effort to establish code provisions that better predict the punching shear strength of slab-column connections.

2. Conversion values for concrete compressive strength

The database constructed for the evaluation of ACI provisions to predict punching shear strength of flat slab-column connections contains widely different control specimens for determination of concrete compressive strength from experiments around the world. It is therefore essential to standardize the compressive strength values determined on different control specimens in order to make comparisons between different tests as well as between tests and the code equation. Studies reported in Reineck *et al.* (2003) and Reineck *et al*. (2013) formed the basis of the construction of the database reported in this paper and therefore, the strength values reported here are the uniaxial compressive strength f_{1c}, of slender prisms. Conversion factors used in this database are taken from Fib Bulletin 12 (2001) and Aslani *et al.* (2017) which are mainly based on the Model Code (1990). These are summarized in Table 1. Different concrete compressive strength values are converted into the uniaxial prism strength f_{1c}, which is determined from a prism with dimensions of 100×100×500 mm. The uniaxial prism strength f_{1c}, can be calculated in Eqs. (1)-(2)

Table 1 Conversion factors of concrete compressive strength of different control specimens

Specimen type and size, mm	Relation	Specimen type and size, mm
Cylinder $D150\times H300$	$f'_{c,cyl,150,300} = 1.05 f'_{c,cyl,100,300}$	Cylinder $D100\times H300$
Cube 150×150	$f'_{c,cube,150} = 0.90 f'_{c,cyl,100}$	Cube 100×100
Cylinder $D150\times H300$	$f'_{c,cyl,150,300} = 0.80 f'_{c,cube,150}$	Cube 150×150

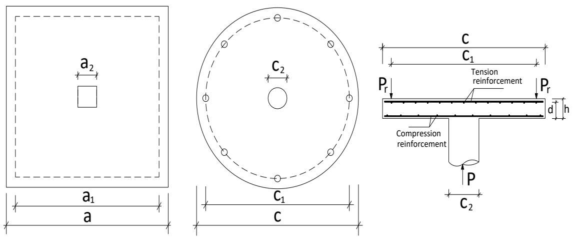

Fig. 1 Representative geometrical dimensions of slabs and test configuration

$$f_{1c} = 0.79 f_{c,cube} \quad (1)$$

$$f_{1c} = 0.95 f_{c,cyl} \quad (2)$$

Where $f_{c,cube}$ is the uniaxial compressive strength of cubes 150×150×150 mm and $f_{c,cyl}$ is the uniaxial compressive strength of cylinder with a diameter equal to 150 mm and h = 300 mm.

Determination of f_{1c} is carried out using the following criteria:

• The tested values of concrete compressive strength on prisms should be taken, unless these could be regarded as not reliable, particularly for the case of small number of tested control specimens.

• The value f_{1c} converted from $f_{c,cyl}$ should be taken, unless the values converted from cubes could be regarded as more reliable, for the case of a high number of tested control specimens.

• The value f_{1c} converted from cubes is taken if no cylinder tests are available, due to the high scatter of this kind of test.

3. Development of the shear database

3.1 Establishing a test database

The punching shear database constructed in this paper

Table 2 Individual set criteria for the establishment of EPSD

Criterion	Description of the individual criterion
Concrete compressive strength	f_{1c}>10 MPa
Slab depth	h>50 mm
Slenderness ratio	a/d>2.0
Steel reinforcement	Ribbed bars

for the slab-column connections without shear reinforcement consists of data for material strength, cross section geometry, loading and support boundary conditions, flexural reinforcement, and the measured punching failure load. These data parameters and notation are given in the Notation. Representative geometrical dimensions of square and circular slabs along with the cross-sectional test configuration are shown in Fig. 1.

It must be noted that the CPSD consists of rectangular and circular slabs without shear reinforcement. The data collected for the punching shear database was initially reported either in SI or Imperial units. In order to evaluate the data reported in CPSD effectively, all the data has been converted into SI units and is summarized in Appendix A.

The test data found in the literature was critically investigated for quality, test procedure and data accuracy. For instance, data with missing information such as strength of concrete and reinforcement, flexural reinforcement ratio, slab depths and ultimate load strength were omitted in the development of the CPSD.

3.2 Criteria for setting up evaluation punching shear database (EPSD)

Determining the assigned criteria was vitally important, particularly for the establishment of a set of certain test results that form the Evaluation Punching Shear Database (EPSD). Therefore, a set of criteria were determined and test results that satisfy these criteria were then included in EPSD for the assessment of the code provision. In this paper, set criteria (SEC) consists of concrete compressive strength, effective depth of slab, column size and flexural reinforcement ratio. Table 2 shows the set criteria (SEC) that a test result satisfied to participate in EPSD.

As it is not practically accepted to meet concrete compressive strength below 10 MPa in construction practice, the number of test results that failed to satisfy this criterion for concrete compressive strength (f_{1c}) are excluded from the EPSD. In addition to the concrete compressive strength, data with slab depth less than 50 mm was also excluded from the EPSD because this causes the reduction in the compression zone height and significantly decreases the punching shear strength of the connection.

Apart from the criteria determined for the SEC, checks against membrane action, bond failures and slenderness ratio of the slabs needed to be considered before a set result could be included in SEC. These factors played a significant role in the development of EPSD for the assessment of the code provisions. Stein *et al*. (2007) however reported that, many studies conducted previously often wrongly reported

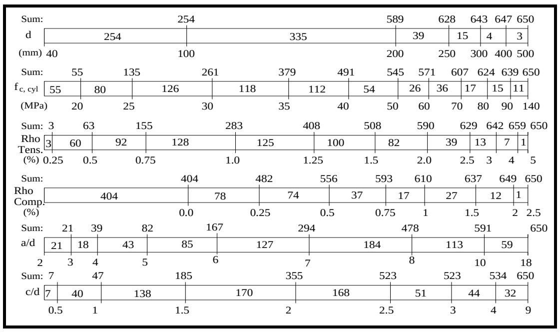

Fig. 2 Distribution of parameters in 650 members of CPSD

the failure mechanism and misled the prediction of the actual punching shear strength of slab-column connections.

A total of 741 slab-column connections without shear reinforcement were initially collected. Of these, 91 experiments which fulfilled neither SEC nor checks against membrane action, bond failures and slenderness ratio of the slabs were disregarded for further evaluation in CPSD. The checks against membrane action, bond failures and slenderness ratio of the slab are summarized herein:

• Membrane action enhances the punching shear strength and ductility of slabs and therefore the slabs that are subjected to membrane action are excluded from CPSD.

• Due to insufficient data provided on the detailing of flexural reinforcement, provisions of location of strain gauges and boundary condition of tests specimens, bond failures that take place at end supports could only be recognized based on the data provided in the subsequent published articles. Slabs failed due to the anchorage are then excluded from the CPSD in this paper.

• The slenderness ratio, span to depth ratio, (a/d) of the slabs reported in the literature are between 1.40 and 24 where a is the radial distance from the column face to the bearing points and d is the effective depth. Slenderness ratio is known to have a significant influence on the punching shear strength and this fact is not taken into account in the code provisions. It is reported in Lovorovich and McLean (1990) that there is systematic increase in the punching shear strength of slabs with the systematic decrease in the slenderness ratio below 2.0 and therefore tests with slenderness ratio less than 2.0 were excluded from the CPSD.

For instance, Li (1997) conducted an experimental study on the punching shear behaviour of slabs with hoop reinforcement however this has resulted in an increase in the punching shear capacity due to the development of the membrane action and therefore these datasets were eliminated from the Evaluation punching shear database as the main focus of the paper is addressing solely the punching shear strength of flat slab-column connections. Some of the datasets reported in Kinnuen and Nylander (1960) as well as Bazant and Cao (1987) were also excluded from the Evaluation punching shear database due to the bond failure mechanisms taken place prior to the expected punching failure. Several experiments from Yitzhaki (1966) used plain bars, instead of ribbed bars for the flexural reinforcement and hence these datasets were also excluded from the Evaluation punching shear database.

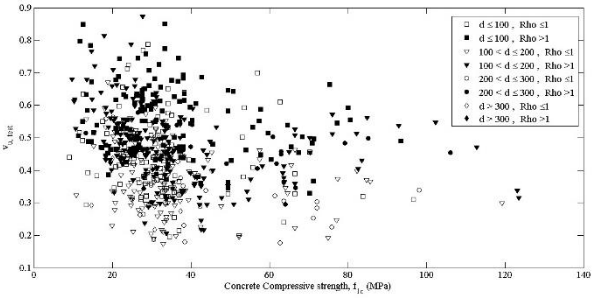

Fig. 3 Influence of concrete compressive strength on non-dimensional shear force for 650 tests of EPSD

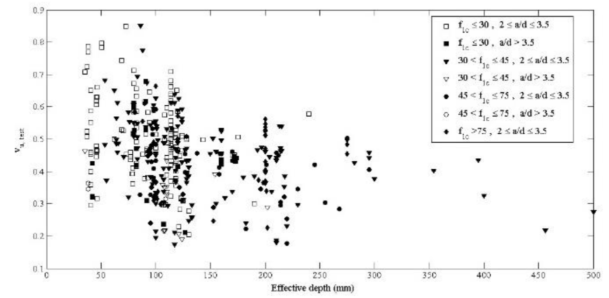

Fig. 4 Influence of effective depth on dimension free shear force for 650 tests of EPSD

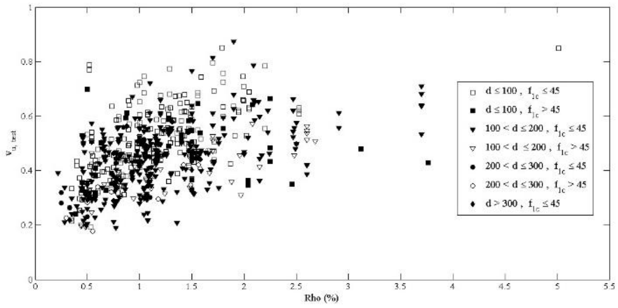

Fig. 5 Influence of flexural reinforcement ratio (percent) on dimension free shear force for 650 tests of EPSD

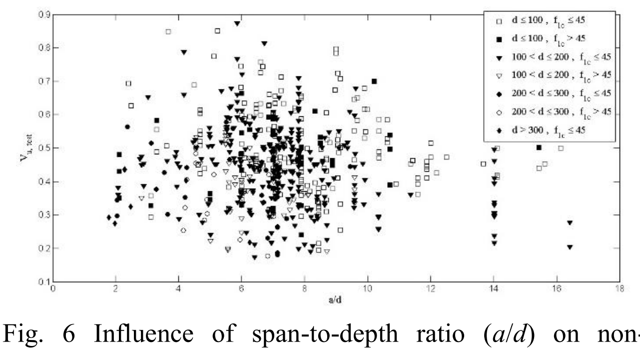

Fig. 6 Influence of span-to-depth ratio (a/d) on non-dimensional shear force for 650 tests of EPSD

Studies conducted by El-Ghandour *et al*. (2003) and Hassan *et al*. (2013a, 2013b) used fibre-reinforced polymer bars as flexural reinforcement in these connections and therefore they were also not included in the Evaluation punching shear database.

4. Presentation of the evaluation shear database

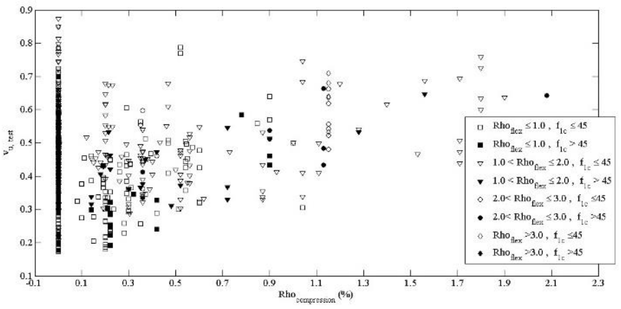

Fig. 7 Influence of compression reinforcement ratio on non-dimensional shear force for 650 tests of EPSD

The authors and individual parameters for each test in Evaluation Punching Shear Database (EPSD) created using the set criteria (SEC), are summarized in Appendix A. EPSD comprises 650 tests that all satisfy the set criteria and checks against membrane action, bond failures and slenderness ratio of the slabs. The distribution of parameters of 650 tests are shown in Fig. 2. Effective depth, concrete compressive strength, flexural reinforcement ratio, span to depth ratio and column size to depth ratio are used for describing the distribution of the test data. These are discussed in more detail when dealing with Figs. 3 to 7.

In order to address the effects of dominant parameters, the punching shear strength is expressed in terms of the non-dimensionalized shear force, $v_u=V_u/f_{1c}b_od$. Punching shear strength is then plotted versus the primary parameters in Figs. 3 to 7. Range of the parameters shown in Fig. 2 are also visualized in Figs. 3 to 7. It can be seen in Figs. 3 to 7 that the scatter for dimension-free shear force ranges between 0.20 and 0.89.

Fig. 3 shows the plot of the non-dimensional shear force values versus the uniaxial compressive strength f_{1c}=0.95 f'_c. Most of the test data had low compressive strength values, for instance, 40 percent of tests (261 out of 650 in Fig. 2) had concrete compressive strength less than 30 MPa. On the other hand, only 7 percent of tests (43 out of 650 in Fig. 2) had concrete compressive strength above 70 MPa. It can be seen in Fig. 3 that the non-dimensional force decreases with increasing concrete compressive strength. The results shown in Fig. 3 also indicate that, unsurprisingly, the increase in punching shear strength is not proportional to the increase in concrete compressive strength.

The non-dimensional shear force values are plotted versus the effective depth (d) of slabs are shown in Fig. 4. It is seen in Fig. 4 that considerable amount of tests were conducted for small depths. In fact, 39 percent (254 out of 650) had values $d \leq 100$ mm, 52 percent (335 out of 650) had values 100 $<d\leq200$ mm and 9 percent (61 out of 650) had d>200 mm. ACI 318-14 does not account for the size effect on the punching shear strength. The results shown in Fig. 4 clearly demonstrate that the size effect significantly affects the punching shear strength particularly for slabs thicker than 150 mm.

In Fig. 5, the test values for the non-dimensional force, υ_u, are plotted versus the flexural reinforcement ratio (percent), $\rho = A_s/bd$. 45 percent of the test data (283 out of 650) are below ρ=1 percent. 9 percent of the test data (60 out of 650 in Fig. 2) are above ρ=2 percent. 47 percent of the test data (307 out of 650) lie between ρ=1 percent and ρ=2 percent. It is known that in practice, such connections are generally designed to have low flexural capacity in order to have more ductile behaviour in seismic scenarios. The results shown in Fig. 5 appear to indicate that the increase in the flexural reinforcement ratio results in a significant increase in punching shear strength and therefore should be taken in consideration in the Code Provisions.

The non-dimensionalized shear force values are plotted versus the span-to-depth ratio (a/d) in Fig. 6. It is shown in Fig. 6 that 88 percent of the tests (574 out of 650) were carried out for a/d<10. There is a gradual decrease in the shear capacity with the increase in a/d above 10. In practice, the slabs are slenderer than beams and therefore, slenderness should be considered for the slabs particularly with high span-to-depth ratio.

In Fig. 7, the test values for the non-dimensional shear force, υ_u, are plotted versus the compression reinforcement ratio (percent), $\rho_{Bot} = A_{s,Bot}/bd$. 65 percent (404 out of 650) of the test data did not contain compression reinforcement where as 32 percent of the test data (152 out of 650) are below ρ=1.0 percent. Only 6 percent of the test data (40 out of 650) are above ρ=1 percent. It is seen in Fig. 7 that the increase in the compression reinforcement resulted in an increase in punching shear strength however, this trend is more markedly seen in slabs with low flexural reinforcement ratio. The results reported in Derogar (2014) indicate that compression reinforcement plays a significant role in increasing the punching shear strength of slabs particularly with low tensile reinforcement, and are in a good agreement in Fig. 7. It is worth nothing however that in practice, slabs are generally designed to have low flexural capacity to enhance the ductility of the overall structure especially in seismic load scenarios. The use of compression reinforcement therefore significantly enhances the punching shear strength of slabs and it is strongly suggested by the authors that this effect should be considered in codes.

5. Comparison with recommended design equations

5.1 Procedure for comparisons

The ratio of the measured ultimate punching shear strength, $V_{u,test}$, to the calculated value from ACI Code Equation, $V_{u,cal}$, is used for the comparison and is defined as the safety factor, γ, in this paper. The safety factor, γ, is given in Eq. (3).

$$\gamma = V_{u,test}/V_{u,cal} \tag{3}$$

The predicted value of a given test is considered to be conservative when γ>1. The material properties introduced in the formula were the average values obtained from the test reports. Test results show a scatter and it is necessary to use a characteristic value of $\gamma_{5\ percent}$ for a conservative expression. $\gamma_{5\ percent}$ indicates the 5 percent fractile of all tests considered to be conservative. The upper and lower average values of the safety factor are defined as given in Eqs. (4)-(5) respectively.

Table 3 Statistical evaluation of the test data

Average value γ	1.3912
Standard deviation s	0.3940
Coefficient of variation v	0.2843
5 percent fractile, $\gamma_{5\ percent}$	0.8410
95 percent fractile, $\gamma_{95\ percent}$	1.8498

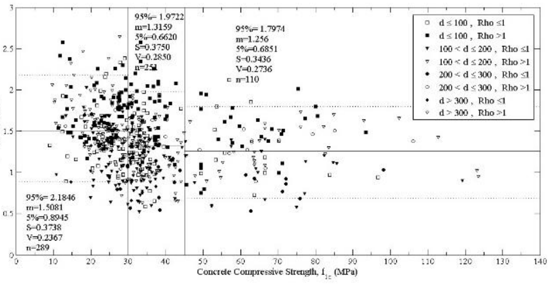

Fig. 8 Safety factor $\gamma = V_{u,test}/V_{u,cal}$ for ACI 318-14 plotted versus concrete compressive strength and statistical results for different groups

$$\gamma_{5\,percent} = \gamma_m - 1.645S \tag{4}$$

$$\gamma_{95\,percent} = \gamma_m + 1.645S \tag{5}$$

where $\gamma_{5\ percent}$ and $\gamma_{95\ percent}$ are the lower and upper average value of the safety factor respectively; and S is the standard deviation for the safety factor. The coefficient of variation, V, for the safety factor is given in Eq. (6).

$$V = S/m \tag{6}$$

Safety factor is therefore plotted versus the main parameters such as concrete compressive strength, effective depth and flexural reinforcement ratio to investigate the influence of these parameters on the punching shear strength of slab column connections.

5.2 Comparisons with the ACI 318-14 design equation

The most commonly used expression for the concrete contribution to punching shear strength in ACI Building Code Requirements is given in Eq. (7) which is an expression (a) from Table 22.6.5.2 in ACI 318-14.

$$V_c = 0.33\sqrt{f_c^{'}}b_o d \tag{7}$$

where V_c is in N, $f_c^{'}$ in MPa, b_o and d in mm.

Table 3 shows the statistical evaluation of the 650 tests in database for the safety factor.

It is shown in Table 3 that the coefficient of variation v = 0.2843 is quite high and the 5 percent fractile value is low with 0.8410. These results indicate that the ACI 318-14 is unsafe and therefore the precision of the safety factor is investigated in the following figures where concrete compressive strength, flexural reinforcement ratio and effective depth were plotted versus the safety factors.

The quality of the empirical equation for punching shear strength given in Eq. (7) is assessed as a measure of an

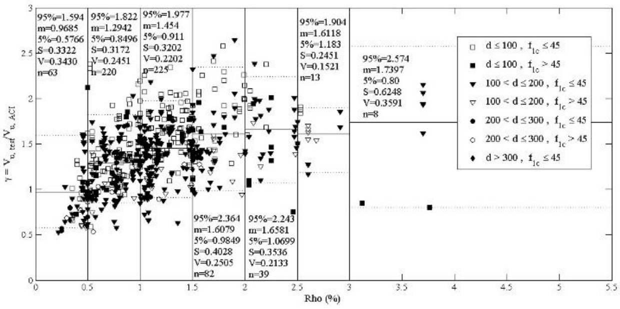

Fig. 9 Safety factor $\gamma = V_{u,test}/V_{u,cal}$ for ACI 318-14 plotted versus flexural reinforcement ratio (percent) and statistical results for different groups

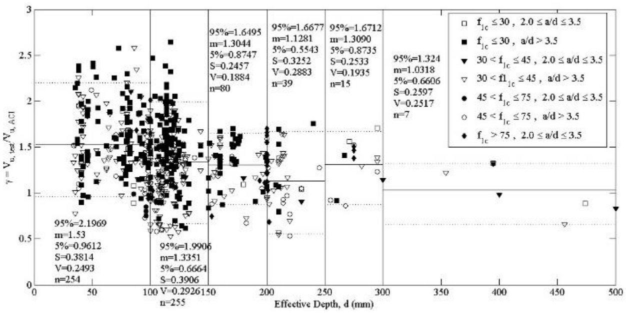

Fig. 10 Safety factor $\gamma = V_{u,test}/V_{u,cal}$ for ACI 318-14 plotted versus slab effective depth (mm) and statistical results for different groups

analytical model for slabs without shear reinforcement. The safety and precision of the ACI Code Equation for use in design practice is examined using the statistical distribution of γ as shown in Eq. (3).

Safety factor, γ, is plotted versus $f_{1c}=0.95\ f_{c,cyl}$ in Fig. 8. It must be underlined that the concrete compressive strength is the only parameter considered in ACI 318-14. The average, upper (95 percent fractile) and lower (5 percent fractile) values for the selected test data are illustrated using the horizontal lines shown in Fig. 8. When the distribution of the test data shown in Fig. 8 is studied, code equation appears to be unsafe for all ranges of the concrete compressive strength. The differences between the normal-strength and high-strength concrete are illustrated using the statistical values of groups of tests with respect to compressive strength. The average value of the safety factor is higher at normal-strength than that of high-strength concrete. While the standard deviation, s, does not vary much, the coefficient of variation seems to have slight fluctuations from the strength groups considered in Fig. 8. This means that the upper and lower values are quite different for normal- and high-strength concrete. The scatter seems to be the general trend for normal- and high-strength concrete however, the scatter is slightly larger specifically at f_{1c}>45 MPa that may be due to the quality of the production of high-strength concrete in test laboratories.

Safety factor, γ, is plotted versus the flexural reinforcement ratio $\rho=A_s/bd$ in Fig. 9. It must be noted that the flexural reinforcement ratio is not a parameter in ACI 318-14. Results shown in Fig. 9 indicate that safety factor is increasing with the increase in flexural reinforcement ratio. The higher values of flexural reinforcement ratio are normally provided for high-strength concrete to reduce the height of compression zone to a maximum of $0.5d$. It can be seen in Fig. 9 that the overall safety of ACI318-14 is only attained with high values of flexural reinforcement however, slabs with low reinforcement ratio appear to be unsafe as it is demonstrated using statistical evaluations for different groups of tests within small ranges of flexural reinforcement ratios.

Low flexural reinforcement ratio, particularly $\rho_{flex}<1.1$ percent, results in a great reduction in safety factors and indicates that the ACI 318-14 is unconservative in this range. For instance, the lower characterized value $\gamma_{5\ percent}$ for the safety factor is down to 0.576 and 0.8495 for $\rho_{flex}<0.5$ percent and $\rho_{flex}<1.0$ percent respectively. On the other hand, for higher reinforcement ratios, $\gamma_{5\ percent}$ of the safety factor is still smaller than the required value of 1.0. For instance, the lower characterized value $\gamma_{5\ percent}$ for the safety factor is down to 0.911 for $1.0<\rho_{flex}<1.5$ percent. The results shown in Fig. 9 clearly demonstrate that the average and the safety factors increase gradually with the increase in reinforcement ratios. Results discussed in this section validate the pronounced influence of the reinforcement ratio on the punching shear strength of slabs without shear reinforcement.

The "safety" factor is plotted versus the slab effective depth, d, in Fig. 10. The statistical evaluation for the different groups of tests shown in Fig. 10 illustrate the influence of the depth d on the punching shear strength. Once again, these parameters also are not considered directly in ACI 318-14. Fig. 10 shows that for all ranges of slab effective depth, ACI 318-14 predicts unsafe results. When the slab effective depth $d \leq 100$ mm, $\gamma_{5\ percent}=0.9612$ and indicates that only for this range of the slab effective depth, the ACI 318-14 provides relatively conservative predictions for punching shear strength of slabs. On the other hand, when the slab effective depth $d>100$ mm, 5 percent fractile, $\gamma_{5\ percent}$, is always below 1. For instance, when the slab effective depth is between $200<d<250$ mm, 5 percent fractile, $\gamma_{5\ percent}$, is 0.5543 and when the slab effective depth $d>300$ mm, 5 percent fractile, $\gamma_{5\ percent}$, is 0.6606. It must be reported that slab effective depth $d>200$ mm only consist of the 9 percent of the test data (61 out of 650) and we understand that there are limitations to the slab effective depth under laboratory conditions. The ACI Code provisions does not take into account the size effect and hence it overestimates the punching shear capacity of slab particularly with the thickness greater than 200 mm. The slab effective depth, d, plays a significant role in punching shear strength of slabs and strongly suggested to be considered in ACI 318-14.

6. Conclusions

This paper has examined the ACI code provisions for the prediction of the punching shear strength for flat slabs. It has focused on the key parameters that are not taken into consideration during the development of the code provisions. An approved set of criteria was assigned and selected test data under the agreed criteria that is then included in evaluating the punching shear database (EPSD). We have collated a qualified database of 650 punching shear test results in this paper. Apart from the critically set criteria, the checks against membrane action, bond failures and slenderness ratio of the slab are also carried out for the construction of evaluation punching shear database. All slabs examined in this database have been tested under gravity loading and do not contain shear reinforcement. The notation of the selected test data is listed in the Notation list and the database is presented in Appendix A with their principal data. In the interest of uniformity, the uniaxial compressive strength f_{1c} (strength of a slender prism) is derived using the conversion factors for concrete strength for the many different control specimens.

The punching shear strength code provision, expression (a) Table 22.6.5.2 in ACI 318-14, is investigated by the calculated safety factor of $\gamma=V_{u,test}/V_{u,cal}$ using the punching shear database. The statistical evaluations reported in this paper for the whole data indicated that ACI 318-14 is unsafe with a 5 percent fractile of only 0.84. The key parameters such as concrete compressive strength, flexural reinforcement ratio and slab effective depth, play a significant role in punching shear capacity of slabs, and are evaluated individually using refined plots. The expression in ACI 318-14 is found to be unsafe for most of the groups of test data examined. The results reported in this paper show that the ACI 318-14 equation is unsafe in all ranges of the concrete compressive strength. Although the flexural reinforcement ratio and the slab effective depth were not taken into consideration for the development of the code provisions, results reported in this paper indicate that the code predictions are often unconservative particularly for the range of low flexural reinforcement and for slabs effective depth d>100 mm. It is appropriate to conclude with the statement that there is still a need for the development of code provisions by performing more qualified regression experiments for the ranges that are not studied at the literature so that the predictive expression matches tests more closely.

Acknowledgments

The authors would like to acknowledge the comments by P. Gülkan.

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PL

Notation

a	side length of the slab
a_1	contraflexural distance of the slab
a_{2x}, a_{2y}	side length of a rectangular column in x and y direction
b	perimeter of the loaded area
b_o	shear perimeter
c_2	column diameter
d	effective depth of the slab
f_c	cylinder concrete strength
f_{1c}	concrete compressive strength of a prism
f'_c	concrete compressive strength
$f'_{c,cube,\ 100}$	compressive strength of cubes (100/100/100 mm)
$f'_{c,cube,\ 150}$	compressive strength of cubes (150/150/150 mm)
$f'_{c,cyl,100,300}$	compressive strength of cylinders ($\phi = 100\,\text{mm}$, h=300 mm)
$f'_{c,cyl,150,300}$	compressive strength of cylinder ($\phi = 150\,\text{mm}$, h=300 mm)
f_{cu}	characteristic concrete cube strength
f_y	yield strength of flexural reinforcement
h	slab depth
S	standard deviation
V	coefficient of variation
V_{ACI}	calculated punching shear strength according to the ACI 318-14
V_c	vertical component of the concrete resistance provided in the compression zone
V_{cal}	calculated punching shear capacity of the slab
V_{test}	measured punching shear strength of the slab
v_c	maximum design shear stress
v_u	shear stress
ρ_{flex}	ratio of tensile reinforcement, A_s, to $b.d$
ρ_{Bot}	ratio of compression reinforcement, A'_s to $b.d$
ρ_{Top}	ratio of tensile reinforcement, A_s, to $b.d$
γ	safety factor
$\gamma_{5\ \text{percent}}$	lower fractile, 5 percent
$\gamma_{95\ \text{percent}}$	upper fractile, 95 percent

Appendix A

Slab No.	Researcher	Slab Thickness, h (mm)	Slab Depth, d (mm)	Column Size, a_2 (mm)	Rho Flex. ρ_{flex} (%)	Compression Flex. ρ_{Bot} (%)	Concrete strength f_{1c} (MPa)	Dimensions of Slab, a (mm)	contraflexural point, a_1 (mm)	V_{test} (kN)
1 ~ 6	Li (2000)	135 ~ 550	100 ~ 500	200×200 ~ 300×300	0.76 ~ 0.98	0.08 ~ 0.22	37.43	925×925 ~ 1975×1975	925×925 ~ 1975×1975	330 ~ 2681
7 ~ 21	Ramadan 1996 Cited in Swamy and Ali (1982)	125	98 ~ 102	150 × 150	0.58 ~ 1.30	0	31.5 ~ 84	Ø 1700	Ø 1372	169 ~ 405
22 ~ 38	Marzouk and Hussein (1991)	90 ~ 150	70 ~ 120	150 ×150 ~ 300×300	0.39 ~ 2.08	0 ~ 0.42	28.5 ~ 71.3	1700 × 1700	1500 × 1500	178 ~ 645
39 ~ 46	Tolf 1998 (cited in FIB bulletin 12)	120 and 240	98 and 200	Ø 125 and Ø 250	0.4 ~ 0.8	0	21.70 ~ 27.50	Ø 1270 and Ø 2540	Ø 1189 and Ø 2378	145 ~ 603
47 ~ 49	Stein *et al*. (2007)	150	118	250×250	0.45 ~ 0.98	0.54	24.40 ~ 28.22	1900×1900	1800×1800	329 ~ 438
50 ~ 60	Muttoni *et al.* (2009)	125 ~ 500	117 ~ 456	130×130 ~ 520×520	0.22 ~ 1.50	0.0 ~ 0.22	26.22 ~ 38.5	1500×1500 ~ 6000×6000	1500×1500 ~ 5700×5700	115 ~ 2153
61 ~ 74	Papanikolaou *et al.* (2005)	100 ~ 250	80 ~ 230	150×150	0.54 and 1.08	0	27.5 ~ 32.78	750×750	750×750	164 ~ 635.7
75 ~ 84	Ozden *et al.* 2006	120	100	200×200	0.73 ~ 2.25	0.37 ~ 1.13	17 ~ 77.24	Ø 1500	Ø 1200	188 ~ 691
85~ 87	Birkle and Dilger (2008)	160 ~ 300	124 ~ 260	250×250~ 350×350	1.10 ~ 1.54	0.19 ~ 0.40	29.83 ~ 34.39	1000×1000 ~ 1900×1900	1000×1000 ~ 1900×1900	483 ~ 1046
88 ~ 94	Regan (2004)	160	128	50×50 ~ 170×170	0.93 ~ 1.73	0.19	28.73 ~ 44.31	2000×2000	1830×1830	190 ~ 380
95 ~ 118	Regan (1986)	80 ~ 250	64 ~ 200	80×80 ~ 250×250	0.78 ~ 1.52	0	11.32 ~ 40.66	1200×1200 ~ 3000×3000	900×900 ~ 2745×2745	117 ~ 825
119~ 122	Lovrovich and McLean (1990)	100	83	Ø 100	1.71	1.71	38	Ø 700 ~ Ø1500	Ø 400 ~ Ø1200	129 ~ 204
123~ 124	Marti *et al.* 1977 (cited in FIB bulletin 12)	180 and 191	143 and 171	Ø 300	1.15 and 1.49	0.29 and 0.46	25 and 32.85	Ø2750	Ø2600	626 and 628
125~ 126	Schaefers 1977 (cited in FIB bulletin 12)	143 and 200	113 and 170	Ø 210	0.82 and 0.54	0	20.22 and 21.13	Ø1960	Ø1680	280 and 460
127~ 130	Lander *et al.* 1977 (cited in FIB bulletin 12)	110	80	Ø 100 ~ Ø 320	1.80	1.80	28.88 ~ 31.84	Ø1260	Ø1056	183~ 324
131~ 132	Lander *et al.* 1973 (cited in FIB bulletin 12)	147 and 280	127 and 240	Ø 226 and Ø 500	1.18 and 1.31	0	26.52 and 30.17	Ø 1400 & Ø 2900	Ø 1200 and Ø 2650	362 and 1662
133~ 134	Corley and Hawkins (1968) (cited in FIB bulletin 12)	146	111	203×203 ~ 250×250	1.01 and 1.51	0	42.18	2135×2135	1820×1820	266 and 334
135~ 154	Base (1966), Bernaert and Puech (1966) (cited in FIB bulletin 12)	140	102 ~ 124	203×203	1.00 ~ 1.90	0	15.2 ~ 39.33	Ø 1370	Ø 1370	247 ~ 541
155~ 166	Manterola (1966) (cited in FIB bulletin 12)	125	107	100×100 ~ 450×450	0.46 ~ 1.04	0.0 ~ 1.04	23 ~ 37.72	3250 × 3250	3000 × 3000	165 ~ 397
167~ 180	Moe (1961)	152	114	254 × 254	1.05 ~ 2.60	0.0	19.5 ~ 26.22	1830 × 1830	1780 × 1780	312 ~ 433
181~ 205	Istner and Hognested (1956) ((cited in FIB bulletin 12)	152	114	254×254 and 356×356	0.49 ~ 3.70	0.0 ~ 1.15	11 ~ 38.40	1830 × 1830	1780 × 1780	178 ~ 578
206~ 223	Kinuen and Nylander (1960)	149 ~ 158	117 ~ 128	Ø 150 and Ø 300	0.48 ~ 2.10	0.0	23.41 ~ 29.87	Ø 1840	Ø 1710	255 ~ 540
224~ 225	Marzouk *et al.* 1996	150	119	250×250	1.06	0.35	30.55 and 35.35	1900 × 1900	1870 × 1870	474
226	Marzouk *et al.* 1998	150	119	250×250	1.06	0.35	63.8	1900 × 1900	1900 × 1900	511
227	Marzouk and Jiang (1997)	150	119	250×250	1.093	0.30	63.65	1950 × 1950	1950 × 1950	511
228~ 229	Broms (2000)	180	150	250×250	0.44	0.21	19.95 and 25.56	2600 × 2600	2000 × 2000	360 and 408
230~ 235	Alexander and Simmonds (1992)	155	133	200×200	0.50 ~ 0.82	0.28 ~ 0.35	24.7 ~ 33.54	2750 × 2750	2750 × 2750	257 ~ 319
236~ 237	Binici and Bayrak (2003)	152	114	304×304	1.76	0.0 and 0.12	26.89	2133× 2133	2133× 2133	494 and 510
238	Ospina *et al.* (2003)	155	120	250×250	0.86	0.0	34.96	2150× 2150	1795× 1795	365.1
239~ 242	Chen and Li (2000)	100	75.2	150×150	0.56 and 1.23	0.0	16.06 and 32.68	1000 × 1000	840 × 840	103.9 ~ 225.7
243~ 244	McHarg *et al.* (2000)	150	110	225 ×225	1.11 and 2.15	0.36	28.5	2300 × 2300	2108× 2108	306 and 349
245~ 247	Swamy and Ali (1982)	125	100	150×150	0.35 ~ 0.72	0.2 and 0.31	36.95	1800 × 1800	1690 × 1690	130.7 ~ 221.7
248~ 251	Theodorakopoulus and Swamy (2003)	125	100	100×100~ 200×200	0.35 and 0.53	0.20	32.76 ~ 34.81	1800 × 1800	1690 × 1690	137 ~ 191
252	Osman *et al.* (2000)	150	120	250×250	0.50	0.0	35.91	1900 × 1900	1830 × 1830	310.2
253~ 260	Harajli *et al.* (2006)	55 and 75	40 and 55	100×100	1.0 and 1.50	0.0	24.13 ~ 33.73	670 × 670	670 × 670	45.9 ~ 113.8
261	Mokhtar *et al.* (1985)	150	108	250×250	1.56	0.38	34.2	1900 × 1900	1800 × 1800	408
262~ 263	Yamada *et al.* (1992)	200	156 and 160	300×300	1.23 and 1.53	0.62 and 1.53	20.5 and 24.7	2000 × 2000	1581 × 1581	441 and 658
264~ 270	Manterola (1966) (cited in FIB bulletin 12)	240	194 ~ 202	Ø 250	0.30 ~ 1.20	0.0	75.91 ~ 98.23	Ø 2540	Ø 2400	565 ~ 1041
271~ 283	Tomaszevicz (1993) (cited in FIB bulletin 12)	120 ~ 320	88 ~ 275	100×100 ~ 200×200	1.50 ~ 2.60	0.0 and 0.9	60.90 ~ 106.1	1500×1500 ~ 3000×3000	1100×1100 ~ 2500×2500	330 ~ 2400
284~ 285	Mowrer and Vanderbilt (1967)	76.2	51	152×152	1.70 and 2.20	0.0	12 and 14.50	914×914	914×914	113 and 122.8
286~ 289	Olivera *et al.* 2000	130	93 ~ 109	120×120	1.20 ~ 1.50	0.0	27.27 ~ 59.71	1800×1800	1650×1650	255 ~ 335
290~ 295	Ghannoum (1998)	150	110	225×225	1.11 and 2.15	0.36	25.75 ~ 63.75	2300×2300	2108×2108	301 ~ 485
296~ 297	Corley and Hawkins (1968)	146	111.3	254×254 and 203×203	1.03 and 1.54	0.0	17.75 and 19.33	2100×2100	1820×1820	265and 334.1
298~ 324	Rankins and Long (1987)	51 ~ 65	40 ~ 54	100×100	0.423 ~ 1.993	0.0	26.45 ~ 35.8	700×700	640×640	36.42 ~ 125.9
325~ 328	Sissakis and Sheikh (2007)	150	120	200×200	1.49 and 2.23	0.0	32.78 ~ 40.47	1500×1500	1350×1350	439 ~ 575
329	Beutel and hegger (2002)	230	190	400×400	0.806	0.235	20.81	2750×2750	2400×2400	615

Appendix Continued

339~ 341	Bazant and Cao (1987)	101.6	96.2	Ø 101.6	2.46 and 3.76	0.0	55.54 ~ 50.24	Ø 508	Ø 407	11.51 ~ 15.56
342~ 343	Graf (1938) (cited in FIB bulletin 12)	302 and 504	271 and 474	300×300	0.576 and 1.04	0.0	13.21 and 14.73	1700×1700	1700×1700	1157 and 1647
344~ 345	Keefe (1954) (cited in FIB bulletin 12)	127	113	Ø 153	2.09	0.0	24.8 and 25.65	1090×1090	940×940	315 and 370
346	Franz (1963) (cited in FIB bulletin 12)	140	129	Ø 210	1.074	0.0	20.33	1960×1960	1680×1680	343
347~ 348	Narasiham (1971) (cited in FIB bulletin 12)	178	143	305×305	1.11	1.11	28.88 and 31.35	2280×2280	2000×2000	588 and 687
349	Mart *et al.* 1977 (cited in FIB bulletin 12)	180	152	Ø 300	1.40	0.432	33.63	Ø 2750	Ø 2600	627
350	Muller *et al.* 1977 (cited in FIB bulletin 12)	180	152	Ø 300	1.40	0.432	33.63	Ø 2750	Ø 2600	627
351	Broms (1990)	180	150	250×250	0.93	0.0	22.27	2600 × 2600	2000 × 2000	435
352~ 353	Chana and Desai (1992)	240 & 250	200 & 210	300×300 & 400×400	0.785 & 0.848	0.0 & 0.848	18.33 & 30.63	3000×3000& 4500×4500	2400×2400	1225 & 805
354~ 355	Morley (1977) (cited in FIB bulletin 12)	200 & 250	200 & 250	290×290 & 400×400	1.49 & 1.96	0.0	66.65 & 70.45	2000×2000& 2750×2750	1500×1500& 2250×2250	1100 & 1640
356~ 357	Esfahani *et al.* (2009)	100	73	150×150	0.84 & 1.59	0.0	21.85	1000×1000	920×920	138 & 210
358	Michel *et al.* (2007)	100	71	100×100	0.902	0.546	25.84	1280×1280	1200×1200	121.5
359~ 360	Regan and Samadian (2001)	200	160	200×200	1.26	0.21	27.57 & 31.24	3000×3000	2743×2743	560 & 587
361	Kim *et al.* (2009)	150	114	250×250	1.44	0.52	31.35	2260×2260	2160×2160	376
362~ 363	Scordelis *et al.* 1958	152.4	108	330×330	2.50	0.0	18.43 & 26.6	1828×1828	1320×1320	467 & 485
364 ~ 379	Yitzhaki (1966)	102	78 ~ 82	201×201 200×400 Ø 120 ~ Ø 333	0.526 ~ 1.33	14 ~ 27.5	13.5 & 27.3	1164×1164 ~ 1706×1706	1164×1164 ~ 1706×1706	98 ~ 306.5
380~ 396		51 ~ 152.4	40 ~ 121.7	Ø 101.6 ~ Ø 203	0.45 ~ 5.01	0.0	13.5 & 49.50	Ø 485 ~ Ø 1220	Ø 385 ~ Ø 1120	70.8 ~ 356.5
397	Kruger *et al.* (2000)	150	121	300×300	1.0	0.0	33.25	3000×3000	2750×2750	423
398~ 407	Alander (2000)	197 ~ 205	172 ~ 177	Ø 200 ~ Ø 900	0.465 ~ 1.18	0.0	18.32 ~ 24.85	1770×1770 ~ 2470×2470	1570×1570 ~ 2270×2270	478 ~ 1111
408~ 411	Israel (1959)	100	80	Ø 200 & 200×400	0.98 ~ 1.34	0.0	14.68 ~ 23.85	Ø 1160 & Ø 1700	Ø 1160 & 1700	Ø 151.7 ~ 244.7
412~ 415	Ozawa *et al.* (2000)	85 & 100	65 & 80	100×100	1.05 ~ 1.95	0.0	27.65 & 37.05	1000×1000	1000×1000	120 ~ 180
416~ 417	Matthys and Taerwe (2000)	120	88	Ø 150	1.29 & 1.79	0.0	33.35	1000×1000	900×900	294 ~ 313
418~ 422	Sakinis and Vainiunas (2009)	140	104 ~ 112	200×200	0.50 ~ 1.90	0.468	25.84 ~ 35.42	2135×2135	2000×2000	331.8 ~ 436
423~ 425	Yang *et al.* (2010)	150	112	225×225	0.64 ~ 1.36	0.36	33.54	2300×2300	2150×2150	282 ~ 382
426	Ruiz and Muttoni (2010)	250	210	260×260	1.50	0.20	32.3	3000×3000	2846×2846	974
427	Subedi and Baglin (2003)	138	102	320×320	1.97	0.48	54.72	1300×1300	1150×1150	395
428~ 429	Oliveira *et al.* (2000)	130	96	120×120	1.40 & 1.50	0.22	55.50	1800×1800	1650×1650	270 & 335
430~ 431	Vaz *et al.* (2000)	130	83 & 92	Ø 150	1.56	0.23	37.05	1800×1800	1650×1650	203 & 286
432~ 441	Lips and Muttoni (2010)	250 ~ 400	193 ~ 354	130×130 ~ 520×520	0.5 ~ 1.63	0.0	29 ~ 64.13	3000×3000	3000×3000	591 ~ 2491
442	Kunz *et al.* (2010)	250	210	260×260	1.50	0.0	32.07	3000×3000	3000×3000	974
443~ 450	Pisanty (2005)	140 ~ 200	112 ~ 171	200×200 ~ 300×300	0.95 ~ 1.31	0.0	18.05 ~ 25.08	1700×1700	1700×1700	355 ~ 835
451~ 455	Ebead and Marzouk (2005)	150	109	250×250	0.35 ~ 1.0	0.0	28.5 ~ 34.2	1900×1900	1830×1830	248 ~ 413
456~ 460	Teng *et al.* (2004)	150	110	200×200 ~ 200×600	1.65 ~ 1.81	0.87	34.0 ~ 40.95	2200×2200	1900×1900	423 ~ 649
461~ 470	Rizk and Marzouk (2011)	150 ~ 350	105 ~ 267.5	250×250 & 400×400	0.35 ~ 0.73	0.22	38.0 ~ 77.14	1900×1900	1830×1830	219 ~ 1722
471~ 478	Elshafey *et al.* (2011)	150 & 200	105 ~ 158	250×250	0.40 ~ 2.68	0.0	31.35 ~ 66.5	1900×1900	1830×1830	228 ~ 811
479~ 481	Sundquist and Kinnuen (2004) (cited in Elshafey *et al.* 2011)	120 & 145	100 & 125	Ø 150 & Ø 250	0.64 & 0.8	0.0	22.8 ~ 25.84	Ø 2400	Ø 2400	250 ~ 270
482~ 483	Samadi and Yasin (2008)	150	116	250×250	1.06	0.52	27.06 & 55.2	1500×1500	1470×1470	416 & 468
484~ 485	Faria *et al.* (2011)	100 & 120	70 & 90	200×200	1.2 & 1.91	0.16 & 0.2	18.77 & 23.56	2300×2300	2000×2000	191 & 199
486	Cho *et al.* (2010)	150	125	220×220	0.78	0.30	27	2500×2500	2470×2470	297
487	Abgossou *et al.* (2008)	100	70	100×100	0.98	0.57	32.3	1280×1280	1200×1200	121.2
488~ 491	Hughes and Xiao (1995)	50 ~ 80	38 ~ 60	132×132	0.80 ~ 1.50	0.29 & 0.35	35 ~ 39.5	860×860	800×800	66 ~ 122
492~ 494	Ramos et *al.* (2011)	100 & 125	80 & 101	100×100 & 200×200	1.28 & 1.67	0.22 ~ 0.35	33.52 & 37.25	2300×2300& 1500×1500	2000×2000 & 1300×1300	250 ~ 268
495~ 509	Forssel and Holmberg (cited in Oliveira *et al.* 2004)	130	106 ~ 109	120×120 ~ 120×600	1.07 & 1.10	0.0	41.04 ~ 50.92	1680×2280	1500×2100	240 ~ 446.4
510~ 513	Rizk *et al.* (2011)	350 & 400	245 ~ 295	400×400	0.50 & 1.58	0.18 ~ 0.22	38 ~ 72.2	2650×2650	2505×2505	1722 ~ 2513
514~ 516	Guidotti *et al.* (2011)	250	201 ~ 208	260×260	0.77 ~ 1.563	0.0	30 ~ 49.12	3000×3000	2760×2760	763 ~ 1094
517~ 520	Kamaraldinm (1990)	80	64	150×150	0.55 & 1.0	0.55 & 1.0	25.65 & 32.30	2000×2000	2000×2000	127 ~ 141

Appendix Continued

521~ 532	Sistonen and huovinen (2011)	200	160	Ø 200 ~ Ø 899	0.45 ~ 1.17	0.0	22.9 & 31.07	Ø 2300	Ø 2300	478 ~ 1111
533~ 535	Nguyen-Minh *et al.* (2011)	125	105	200×200	0.66	0.0	20.6	900×900 ~ 1500×1500	825×825 ~ 1425×1425	264 ~ 301
536~ 537	Cheng and Parra-Montesinos (2010)	152	127	152×152	0.56 & 0.83	0.0	45.32	1500×1500	1425×1425	379 & 433
538~ 540	Alam *et al.* (2009)	60 & 80	45 & 63	120×120	0.5 & 1.0	0.0	34.2	1305×1305	1200×1200	84.73 ~ 172
541	Erdogan *et al.* (2007)	150	114	250×250	1.23	0.6	30.4	2300×2300	2000×2000	500
542	Heinzmann *et al.* (2012)	350	295	Ø 400	1.23	0.27	33.75	4100×4100	3920×3920	1710
543	Farghaly *et al.* (2011)	120	97	100×100	1.28	0.39	42.47	1600×1600	1400×1400	179.5
544~ 545	Cited in FIB 12 (2001)	100 & 120	82 & 88	200×200	1.16 & 1.87	0.14 & 0.17	18.81 & 23.56	2300×2300	2000×2000	191 & 199
546	Soudki *et al.* (2012)	100	70	150×150	1.41	0.0	24.51	1220×1220	1220×1220	160.3
547	Oukaili and Tahir (2014)	70	54	150×150	0.64	0.0	33.91	1000×1000	900×900	101.7
548~ 549	Sagesta *et al.* (2011)	250	196 & 212	260×260	0.82 & 1.48	0.32 & 0.37	63 & 64.15	3000×3000	3000×3000	989 & 1431
550	Bompa (2011)	170	155	300×300	0.51	0.51	34.2	1500×1500	1440×1440	495
551	Ferreira *et al.* (2014)	180	143	300×300	1.48	0.0	37.25	2500×2500	2100×2100	779
552~ 568	Nightingale (1970)	51	42	Ø 115 ~ Ø 255	0.52 ~ 1.90	0.52 ~ 1.90	27.84 ~ 50.83	Ø 2745	Ø 2590	56.4 ~ 84
569~ 572	Inacio *et al.* (2015)	125	101.7 ~ 105	200×200	0.94 ~ 1.48	0.14	34.11 ~ 123.6	1650×1650	1530×1530	412.9 ~ 460.9
573	Sagaseta *et al.* (2014)	250	202	250×750	0.75	0.42	42.37	3000×3000	3000×3000	1069
574	Worle (2014)	200	155	Ø 300	2.245	0.42	35.83	Ø 2700	Ø 2400	612
575~ 576	Grimaldi *et al.* (2013)	200	170	200×200	0.90	0.90	57.48	2400×2400	2200×2200	828 & 878
577~ 578	Caldentey *et al.* (2013)	250	200	450×450	1.07	0.30	35.75	2800×2800	2500×2500	955 & 974
579~ 580	Widianto *et al.* (2009)	152.4	130	406×406	0.5 & 1.0	0.15	26.7 &29.83	4268×4268	4268×4268	311 & 401.2
581	Abdullah *et al.* (2013)	150	130	250×250	0.45	0.0	33.73	1800×1800	1600×1600	284
582~ 583	Radik *et al.* (2011)	152.4	109.5	305×305	0.44	0.0	52.25	1500×1500	1219×1219	256 & 263
584~ 585	Hassan *et al.* (2013a)	200 & 350	131.5 & 281.5	300×300	0.77 & 1.66	0.0	36.67 & 43.13	2500×2500	2000×2000	688 & 1692
586	Meisami *et al.* (2013)	105	76	150×150	2.20	0.35	40.28	1200×1200	1000×1000	241.7
587	Lewler and Polak (2011)	120	89	200×200	1.12	0.55	41.8	1800×1800	1500×1500	253
588	Duarte *et al.* (2008)	120	87	200×200	1.20	0.15	37.39	1800×1800	1500×1500	269
589~ 591	Hegger *et al.* (2009)	350 ~ 450	295 ~ 395	200×200	0.88	0.31 ~ 0.35	20.05 ~ 34.58	1400×1400 ~ 1800×1800	1400×1400 ~ 1800×1800	1478 ~ 2405
592	Lan *et al.* (2002)	200	155	350×350	1.68	0.40	33.90	2650×2650	2650×2650	738
593	Aziz *et al.* (2013)	70	50	150×150	0.50	0.0	57	1000×1000	1000×1000	206
594~ 596	Habibi (2012)	200	160	225×225 & 180×270	1.06	0.30	24.7 & 28.5	2300×2300	2000×2000	527 & 547
597~ 598	Oliveira *et al.* (2014)	130 & 150	91 & 121	200×500 & 150×150	1.36	0.0	33.73	2400×2400& 1800×1800	2000×2000 & 1650×1650	274 & 320
599	Borges *et al.* (2013)	200	154	200×600	1.31	0.20	39.9	3000×3000	2700×2700	843
600~ 608	Costa *et al.* (2012)	130	90 ~ 94	100×400 ~ 230×375	1.39 ~ 1.43	0.20	20.9 & 27.55	1800×1800	1800×1800	275 ~ 395
609	Youm *et al.* (2014)	200	170	300×300	0.28	0.17	38.57	2500×3000	2000×2500	670.4
610	Gouveia *et al.* (2013)	125	105	200×200	1.0	0.0	27.28	1650×1650	1600×1600	289.2
611	Hoang (2011)	150	115	150×150	1.28	1.28	46.55	1200×1200	1050×1050	443
612~ 613	Hunchate *et al.* (2014)	50	35	100×100	0.63	0.0	30.41 & 53.06	1100×1100	1050×1050	44.15 & 66.65
614~ 628	Urban (1994) (cited in Derogar 2014)	150	91 ~ 104	160×160 ~ 320×320	0.76 ~ 1.57	0.0	17.4 ~ 35	1800×1800	1600×1600	176 ~ 360
629~ 640	Roll *et al.* (1971)	61	42	102×102	1.14 ~ 2.53	0.0	25.53 ~ 31	737×737	591×591	61 ~ 88
641~ 650	Einpaul *et al.* (2016)	250	197 ~ 218	Ø83 ~ Ø660 & 260×260	0.74 ~ 1.59	0.29 ~ 0.36	32.50 ~ 41.9	Ø 3000 1700×1700 3900×3900	Ø 3000 1530×1530 & 3852×3852	530 ~ 1476