Prediction of crack widths in NZS 3101 and its significance in the seismic design of connections

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ABSTRACT

Crack width is a driving parameter for fastener qualification; however, it is not considered in the seismic design of steel-to-concrete connections with post-installed fasteners. There are examples in the international standardization where seismic performance category selection of fasteners for design is based on predicted crack widths from code formulae. Such practice has not been demonstrated to be safe. In the present research a comparative statistical analysis was performed on the NZS 3101 crack width estimation formula together with another five proposals selected from codes and from the literature, using the same experimental crack width database. It was aimed to study whether the NZS 3101 crack width estimation formula shows similar or different model error characteristics to other models. It was demonstrated that none of the six crack width models is robust. All studied crack width models overestimate the small actual crack widths and underestimate the large actual crack widths. The paper gives recommendations to address these observations.

1 INTRODUCTION

Cracking of reinforced concrete structures due to bending or tension usually has great significance on the structural behaviour. Cracking is inherent to concrete and being the main contributor to the composite action of steel reinforcement embedded in concrete. Tensile capacity of concrete is limited compared to its compressive strength, and the related material failure in tension is brittle (van Mier, 1997). For the control of cracks, reinforcement is placed where concrete is subjected to tension, to ensure the formation of distributed, narrow cracks. Cracking, however, may compromise the watertightness of concrete, the protection of reinforcement against corrosion, and the appearance of the structure from an aesthetic point of view. It is noted that permeability of concrete is the most important factor for the corrosion of the reinforcement and is usually quantified by diffusion coefficients (Richardson, 2002). The permeability of concrete increases due to cracking as the diffusion coefficient of cracked concrete increases with larger crack widths (Picandet et al, 2009). The literature, however, is inconsistent about the existence of a direct correlation between the corrosion rate of embedded reinforcement and crack width. It is assumed that corrosion is more likely to cause cracking rather than to be a result of the already existing cracks, notwithstanding that cracks have the tendency to accelerate an already initiated corrosion process in concretes of inadequate permeability (Vidal et al, 2004; Otieno et al, 2010). The significance of cracking is especially high for post-installed steel-to-concrete connections since there is a high probability that fasteners installed in non-cracked concrete will be intercepted by a crack when cracks form. The load bearing capacity of fasteners is reduced in cracked concrete and depending on the type of the force transfer mechanism (undercut, expansion, bonding) this strength loss can be significant (20 to 80 percent) for large crack widths (Eligehausen et al, 2006). During earthquakes it is expected that cracks are formed in structural concrete members coinciding the location of fasteners. Consequently, all fasteners that transfer seismic loads should be suitable for use in cracked concrete where the cracks cyclically open and close for the duration of the earthquake. Maximum crack widths during earthquakes could be significantly higher than those expected under service loads. In the seismic assessment of fasteners, the crack widths are chosen for values that are believed to be representative for serviceability and ultimate (also referred to as suitability) conditions related to earthquake events. These crack widths are selected based on observations, theoretical analyses, engineering judgements and stipulations. Some limited experimental data are also available on the cyclic crack width opening and closing in concrete during simulated earthquakes. While crack width is a driving parameter for fastener qualification, it is not considered in the seismic design according to either EN 1992-4 (i.e. NZS 3101) or ACI 318. It means that the design requirements (i.e. demand) for fasteners qualified for seismic applications do not depend on the predicted crack width calculated during the structural analysis. This paper provides a comprehensive statistical analysis of the NZS 3101 crack width model together with another five proposals selected from codes and the literature. It is demonstrated that none of the six crack width models is robust. All studied crack width models overestimate the small actual crack widths and underestimate the large actual crack widths. Consequences of these observations on the seismic design of fasteners is provided in the paper.

2 MODELLING OF CRACK WIDTHS

2.1 History of research

The interest in, and the research of cracking in reinforced concrete is as old as the material, reinforced concrete itself. There are no detailed studies available from the first half century of the history of reinforced concrete (1850-1900), simply because the design and construction methods of reinforced concrete were considered as trade secrets in those early years. Following more than twenty years of secret experiments, Thaddeus *Hyatt* published in 1877 the first short monograph dedicated to this new construction material and technology (Hyatt, 1877). The first detailed handbook on reinforced concrete theory and experiments was published by Armand *Considère* in 1899 (translated to English in 1903) (Considère, 1903). Considère's handbook made the first reference to the bond stress distribution along the reinforcement in the vicinity of structural cracks; see Figure 1a.

In their comprehensive literature review Borosnyói and Balázs (2005) concluded that despite the more than 100 years of intensive research activity, there is no globally accepted formulation for crack width in reinforced concrete. Lapi et al (2018) drew a similar conclusion, adding that the prediction capacity of a model is not necessarily increased as the model is more refined. According to Borosnyói and Balázs (2005), the available formulas could be classified in four categories:

a) Calculation of crack width in an analytical way by solving the differential equation of bond-slip,

b) Calculation of crack width by semi-analytical equations, where the models include simplifications either on bond stress or on strains,

c) Calculation of crack width by empirical relationships based on fitting of a large number of experimental data, with or without explicit expression of crack spacing and average strain of reinforcement,

d) Numerical models for direct or indirect consideration of cracks (fracture mechanics models, FEM models, damage models, smeared crack models, etc.).

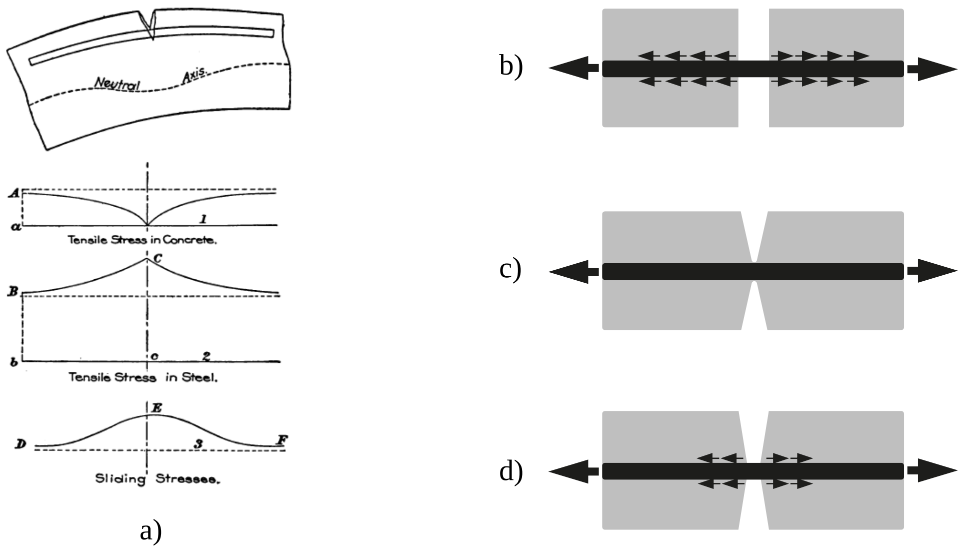
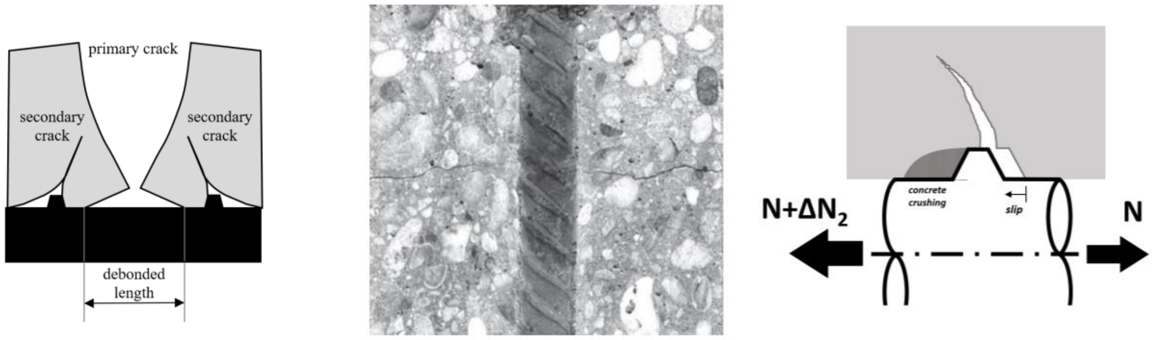


Figure 1: Examples of historical milestones in crack width research; a) Bond stress distribution along the reinforcement in the vicinity of structural cracks (Considère, 1903); b) The bond-slip approach (Saliger, 1936); c) The no-slip approach (Broms, 1965); d) The combined approach (Ferry Borges, 1966) (Note: images b) to d) are based on Carino and Clifton, 1995).

2.2 Limitations in crack width modelling

Two basic, fundamental assumptions can be found in the literature for the theoretical modelling of cracks in reinforced concrete; 1) the bond-slip approach, proposed by Rudolf *Saliger* in 1936, (see Fig. 1b) and 2) the no-slip approach proposed by Bengt *Broms* in 1965, (see Fig. 1c). Both assumptions are still in use today in different code proposals and have also been modified by numerous researchers. In 1966, Julio *Ferry Borges* combined the two basic assumptions into one model (see Fig. 1d), which was later adopted in e.g. the Eurocode 2 and the *fib* Model Code 2010. The governing action in the bond-slip approach is the bond capacity that allows the transfer of stresses from the reinforcement to the concrete while the no-slip approach relies primarily on the deformations in the concrete cover. In the bond-slip approach the reinforcement carries the total force at a crack and the strain in the cracked concrete is zero. Along both sides of the crack, bond stress over the reinforcement-concrete interface can transfer forces from the reinforcement to the concrete. The difference between the elongation of the reinforcement and the concrete is defined as the slip. The largest slip is observed in the crack, and the crack width is considered as the sum of slips at the two faces of the crack. In the bond-slip approach the crack width is assumed to be constant over the concrete cover (Fig. 1b). In the no-slip approach perfect bond is assumed (i.e. no slip occurs between the reinforcement and the concrete), and strain compatibility exists between the steel and concrete at the level of the reinforcement, far from a crack. At the crack, the force in the reinforcement is assumed to act as a concentrated load on the concrete, and in accordance with *de Saint-Venant*'s principle, this peak stress in the concrete is assumed to become uniformly distributed at a distance from the crack that is proportional to the concrete cover. In the no-slip approach the crack width is assumed to be zero at the level of the reinforcement and is considered to increase linearly over the concrete cover (Fig. 1c). A further assumption to mention in connection with cracks in reinforced concrete is the deterioration of bond, or debonding of reinforcement at the crack, first proposed by Yukimasa *Goto* in 1971 and later discussed by many other researchers. According to this concept, as schematically shown in Figure 2a, near to a crack (that is referred as primary crack), on each side of the crack the bond stresses are reduced or non-existing. The extent of this deterioration of bond can hardly be measured experimentally, however, its existence can be conceptualized by experimental strain measurements that showed a somewhat constant strain level in the reinforcement over a distance on both sides of a primary crack. The concept may be explained by the conical secondary cracks forming at the ribs of the reinforcement (Goto, 1971). Cracking in structural concrete is a complex phenomenon and none of the existing theoretical or numerical models can capture its entire complexity yet. Two examples of structural cracks are illustrated in Figures 2b and 3 from the experiments of Borosnyói and Snóbli (2010). It can be observed in Figure 2b that the structural crack is initiated by a rib on the reinforcing bar; experimental observations made by Borosnyói and Snóbli (2010) demonstrated that structural cracks in reinforced concrete elements always coincide with a rib on the reinforcing bar. Stress peaks are formed in the concrete at the tips of the ribs that result in the formation of cracks at very low load levels, and with this, movement (i.e. slip) of the reinforcing bar in the concrete can occur. The ribs of the reinforcing bar restrain this movement by bearing against the concrete lugs between the ribs. Slip is a result of two phenomena (Lutz and Gergely, 1967): 1) wedging action, when the ribs push the concrete away from the rebar, and 2) concrete crushing, when the high bearing pressure in front of the ribs crushes the cement mortar (Fig. 2c). This phenomenon is well-explored in the literature, see e.g. Gambarova and Rosati (1996). It can be observed in Figure 3 that the trajectory of a structural crack in reinforced concrete follows the weakest path and can run either along the interface of aggregate particles (due to the high local porosity and low local strength of the Interfacial Transition Zone, ITZ), or can cut through weaker aggregate particles. Structural cracks in normal concrete always exhibit tortuosity and are never formed as a straight plane. It can also be observed in Figure 3 that the width of the crack is not constant along its length within the concrete cover and shows larger width at the surface of the concrete member (right side of the image in Fig. 3) and smaller width at the level of the reinforcing bar (left side of the image in Fig. 3). Only a limited number of studies are available in the technical literature that investigated the crack width variation within the concrete cover. Well established relationships between the crack widths at the level of the reinforcement and that of on the concrete surface are not known yet. None of the existing crack width models includes the direct consideration of the change of the crack width along the concrete cover, apart from some theoretical stipulations related to the curvature of flexural members in certain proposals.



a) b) c)

Figure 2: Concepts and observations in the anatomy of structural cracks in concrete at the level of the reinforcing bar; a) The debonded length concept after Goto (1971); b) Experimental observation of structural crack coinciding with a rib of the reinforcing bar (Borosnyói and Snóbli, 2010); c) Conceptual illustration of crack formation at the rib of the reinforcing bar after Gambarova and Rosati (1996).

Figure 3: Image of a structural crack in reinforced concrete from the experiments of Borosnyói and Snóbli (2010); Note: the surface of the concrete member is at the right side of the image and the level of the reinforcing bar is at the left side of the image (Borosnyoi-Crawley and Gyurkó-Nagy, 2023).

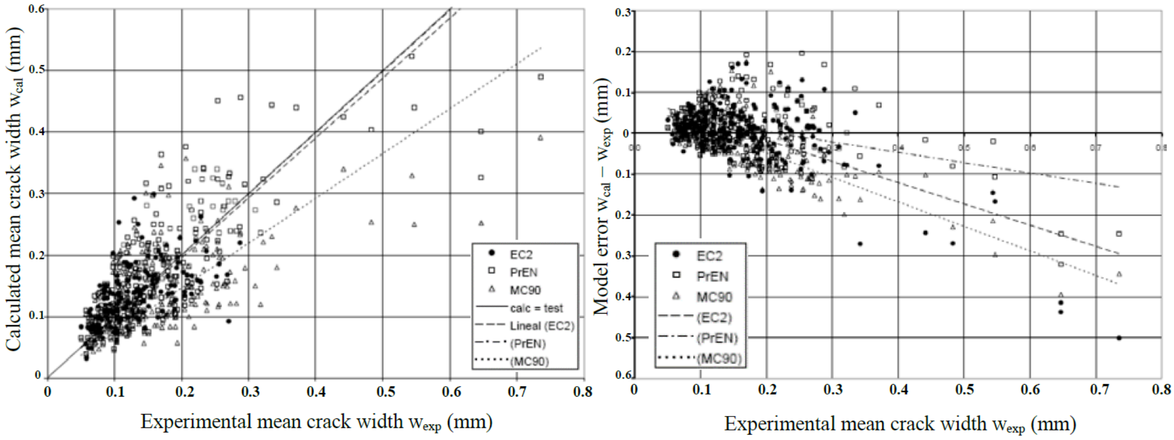


Figure 4: Model error of the Eurocode 2 crack width formula (EC2 Commentary, rev A 31-03-2017)

2.3 Practical crack width modelling

Crack width is typically measured on the outer surface of the concrete structure. Design codes also limit crack widths on the concrete surface with the intention of reducing reinforcement corrosion, water- and air permeability, or for aesthetical reasons. Since the crack width varies within the concrete cover and has different value at the level of the reinforcement than that of observed on the concrete surface, as well as the tortuosity of structural cracks strongly depends on the concrete composition and strength, limiting the surface crack width is a dubious practice for corrosion prevention or watertightness control and could support only the aesthetic crack width control. Reflecting on this, e.g. ACI CODE-318 removed all recommended explicit crack width limits after the ACI 318-95 edition. Other concrete structure codes however, including NZS 3101, define limits for the maximum allowable surface crack width of reinforced concrete flexural members in serviceability conditions. Table C2.1 of NZS 3101:2006 recommends these limiting crack widths in the range of 0.2 to 0.5 mm, depending on load categories, exposure classification and material (i.e. reinforced or prestressed concrete). It is highlighted that crack width estimating proposals found in the literature or in design codes were developed for serviceability limit state conditions where the tensile stress in the reinforcement is relatively low and is not expected to reach the yield stress level. A common technique in the development of crack width estimating proposals, especially in the case of empirical and semi-empirical models, is the calibration of the model parameters to arbitrarily selected experimental databases. Consequently, none of these models can be considered to be universal, their limit of application is determined by the database used for the calibration, however, it is possible to quantify the model error of these models related to their own calibration database. One example is illustrated in Figure 4 that indicates the model error of the Eurocode 2 and Model Code 1990 crack width estimating proposals based on the same calibration database. It can be observed that crack width estimation is generally a very uncertain approach. The error in the calculated crack widths is apparently in the range of the actual crack widths (i.e. ±100 % error) over both the serviceability limit state range and beyond. It can also be observed that the models tend to overestimate the small crack widths (resulting in uneconomic reinforcement ratios) and tend to underestimate the large crack widths (generating safety risk in the design).

2.4 The NZS 3101 crack width model

The current formulae for crack control (Section 2.4.4 in NZS 3101:2006) have been added to the standard in 2006 and have been modified by Amendment 3 in 2017. The crack width model in NZS 3101 is largely based on Frosch (1999). The general formulae as of the time of submitting this paper for publication are as follows:

$$w = 2.0\beta' \frac{f_{s,ch}}{E_s} g_s$$

where:

$$\beta' = \frac{y - kd}{d - kd}$$

y is the distance from the extreme compression fibre to the fibre being considered,

kd is the depth of the neutral axis,

d is the effective depth, distance from extreme compression fibre to centroid of tension reinforcement,

$f_{s,ch}$ is the change in stress in the reinforcement, equal to $f_s - 0.5\ f_{s,c}$ where $f_{s,c}$ is the stress in the reinforcement when the stress in the concrete alongside the reinforcement is zero prior to crack formation. To allow for the influence of shrinkage on crack width the stress change is taken as the stress sustained in the reinforcement after the crack has formed, f_s, plus half the compression stress induced by final shrinkage, $f_{s,c}$. Taking half of the induced compression by final shrinkage makes a nominal allowance for the shrinkage that would have occurred in the test beams used to develop the equations, and the reduced effect of shrinkage that develops after initial cracking has occurred on further increase in crack widths. The value of $f_{s,c}$ may be ignored when the long-term unrestrained shrinkage strain in the concrete is less than 400×10^{-6}.

E_s is the modulus of elasticity of the reinforcing steel,

g_s is the distance from the centre of the nearest reinforcing bar to the surface of the concrete at the point where the crack width is being calculated; for the case where a crack width is being calculated between two bars, the critical value of g_s is given by:

$$g_s = \sqrt{(s/2)^2 + c_m^2}$$

s is the centre-to-centre spacing of the bars,

c_m is the cover distance measured from the centre of the bar to the surface of the concrete.

3 RESEARCH MOTIVATION

As it was mentioned above, while crack width is a driving parameter for fastener qualification, it is not considered in the seismic design according to either EN 1992-4 (i.e. NZS 3101) or ACI 318. This practice was first commented in the German National Annex to EN 1992-4 that recommends the use of C1 seismic qualified anchors if the calculated characteristic crack width is 0.3 mm $\leq w_k \leq$ 0.5 mm, and recommends the use of C2 seismic qualified anchors if the calculated characteristic crack width is 0.5 mm $\leq w_k \leq$ 0.8 mm (where C1 and C2 are in the parlance of EN and EOTA documents). The characteristic crack width is to be calculated by Eurocode 2 (EN 1992-1-1). Such recommendation is arguable, especially for large crack widths like 0.5 mm $\leq w_k \leq$ 0.8 mm for C2 seismic performance category and may generate safety risk. As it was shown in Figure 4, the model error is unacceptably high in the 0.5 mm $\leq w_{exp} \leq$ 0.8 mm range, with a 0.15-0.5(!) mm error. Later, Australian Standard AS 5216:2021 adopted the idea of the German National Annex, recommending that the characteristic crack width is to be calculated by AS 3600. Since the crack width prediction in AS 3600 is practically identical with that of Eurocode 2, the same concerns can be raised. Selecting seismic performance category for post-installed fastener design based on predicted crack widths from code formulae may be unsafe, lacks a well-established scientific background and, consequently, needs very careful engineering judgement in actual seismic design of steel-to-concrete connections.

The main target of the present research was to perform a comparative statistical analysis of the NZS 3101 crack width estimation formula together with another five proposals selected from codes and from the literature, using the same crack width database. It was aimed to see if the NZS 3101 crack width estimation formula shows similar or different model error characteristics to those demonstrated for Eurocode 2 in the literature. This paper summarizes the results of a more detailed report (Borosnyoi-Crawley, 2023a).

4 MODEL CALIBRATIONS

4.1 Existing database

For the calibration of the crack width model proposed in NZS 3101, the database of McLeod (2019) was used. The database compiled by McLeod (2019) is a collection of experimental beam, slab and tie test results that were conducted between 1956 and 2016. For the purposes of the present research, the beam and slab specimen data were used from McLeod (2019), and the tie specimen results were omitted. It is assumed that the crack widths in the database correspond to the maximum crack widths of primary cracks, as it was reported by McLeod (2019) that the crack widths were the maximum measured crack widths.

4.2 Database filtering

For the calibration of the NZS 3101 crack width model the following filter was used to narrow the database for the limits that fit more closely the NZS 3101 provisions:

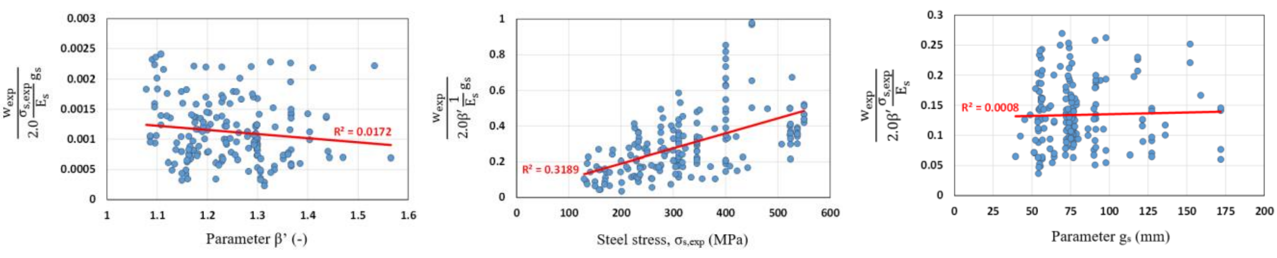
- Concrete strength – max. 85 MPa, in accordance with Clause 8.6.3.2 of NZS 3101 (max. 70 MPa) with an allowance of 15 MPa overstrength in accordance with Clause 2.6.5.5 of NZS 3101; Note: Clause 5.2.1 of NZS 3101 prescribes a minimum 20 MPa specified compressive strength of concrete.
- Rebar diameter – between 8 mm and 40 mm.
- Rebar stress – below 675 MPa, in accordance with Clause 5.3.3 of NZS 3101 (max. 500 MPa) with an allowance of $\phi_{o,fy}$ =1.35 overstrength in accordance with Clause 2.6.5.5 of NZS 3101.
- Rebar spacing – larger than 20 mm; Note: Clause 8.3.1 of NZS 3101 specifies minimum 25 mm rebar spacing.

The filtering resulted in a reduction of the original test data to 204 specimens. The 204 specimens in the filtered database covers a range of crack widths w = 0.05 to 0.60 mm.

4.3 Database correlations regarding the NZS 3101 model

To understand which parameter of the model has correlation with the experimental data, correlation charts have been created for the following rearrangements of the NZS 3101 model, see Figure 5:

$$\beta' \sim \frac{w_{exp}}{2.0\frac{\sigma_{s,exp}}{E_s}g_s} \qquad \sigma_{s,exp} \sim \frac{w_{exp}}{2.0\beta'\frac{1}{E_s}g_s} \qquad g_s \sim \frac{w_{exp}}{2.0\beta'\frac{\sigma_{s,exp}}{E_s}}$$



a) b) c)

Figure 5: Correlation charts for the NZS 3101 crack width model parameters (Borosnyoi-Crawley, 2023a)

The following observations can be made for Figure 5. There is moderate/low correlation with the steel stress (R^2 = 0.3189). There is no or very low correlation with parameter β' (R^2 = 0.0172) and parameter g_s (R^2 =

0.0008). It is apparent that parameters β’ and g_s are not robust in the NZS 3101 crack width model, and the only parameter that somewhat correlates with the experimental data in the filtered database of McLeod (2019) is the steel stress. This observation is based on 204 test specimens from twelve literature sources; therefore, it cannot be generalized, and further studies are needed with the use of different experimental databases to confirm or disconfirm the findings of this preliminary research.

4.4 Comparative analysis of different models

The calibration of the NZS 3101 – Frosch (1999) crack width model has been extended with the calibration of another five, arbitrarily chosen models from the literature; JSCE (2007), Eurocode 2 (EN 1992-1-1:2004), Oh and Kang (1987), Martin et al (1980), Gergely and Lutz (1968). Figure 6 summarizes the general observations, indicating the w_{exp}/w_{cal} ratio over the range of the experimental crack widths in the filtered database (0.05 to 0.60 mm). Table 1 gives the SSE and RMSE model error parameters for w_{cal}. Table 2 summarizes the model uncertainty statistical parameters for the ratio w_{exp}/w_{cal}. Further details can be found in the cited literature (Borosnyoi-Crawley, 2023a).

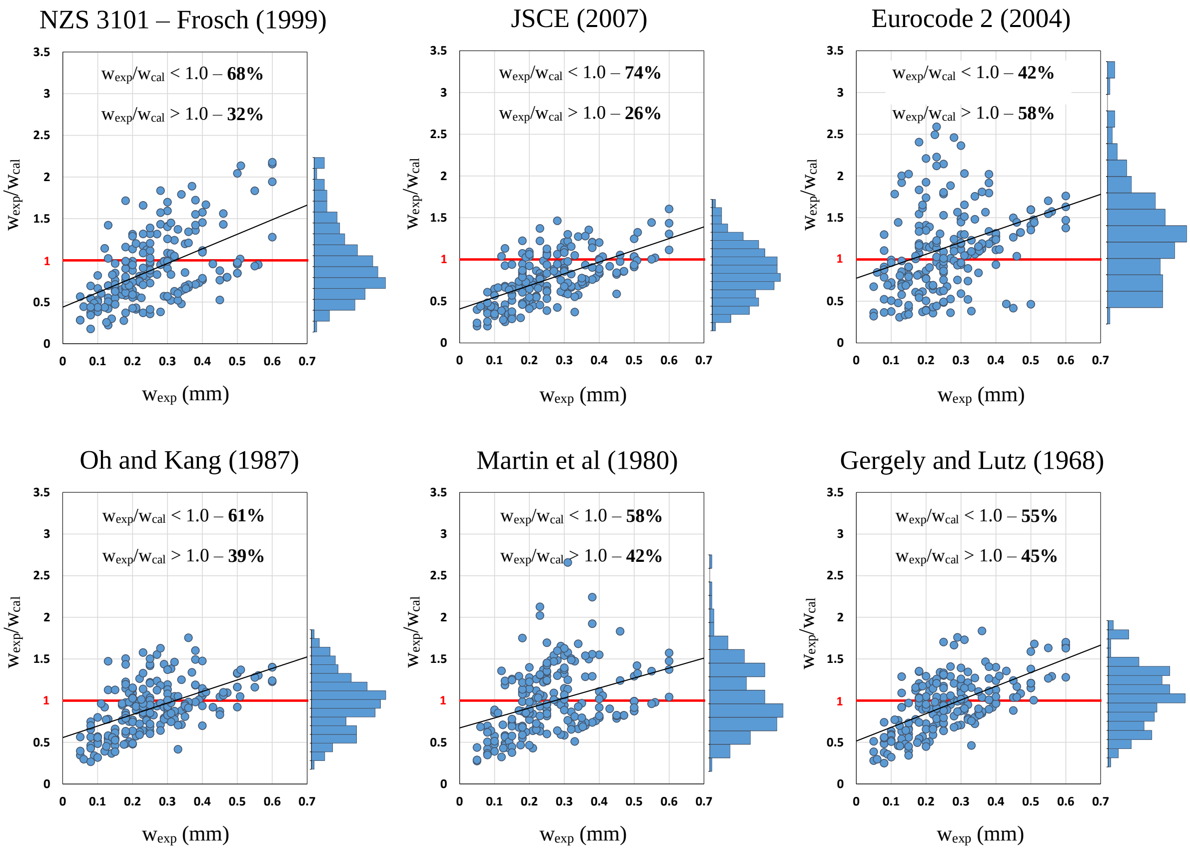


Figure 6: Ratio of the experimental and calculated crack widths (w_{exp}/w_{cal}) (Borosnyoi-Crawley, 2023a)

The following observations can be made for Figure 6 and Tables 1 and 2:

- The root mean square error (RMSE) is the highest for the NZS 3101 – Frosch (1999) crack width model and the lowest for the Oh and Kang (1987) crack width model.
- The coefficient of variation (CoV) is the highest for the Eurocode 2 (2004) crack width model and the lowest for the Oh and Kang (1987) crack width model. The coefficient of variation of the NZS 3101 – Frosch (1999) crack width model is the second highest.
- The range of the w_{exp}/w_{cal} ratio is the highest for the Eurocode 2 (2004) crack width model and the lowest for the JSCE (2007) crack width model.

- The deviation of the mean value of the w_{exp}/w_{cal} ratio from 1.0 is the highest for the JSCE (2007) crack width model and the lowest for the Martin et al (1980) crack width model.
- The skewness of the distribution of the w_{exp}/w_{cal} ratio is the highest for the Eurocode 2 (2004) crack width model and the lowest for the Oh and Kang (1987) crack width model.
- The interquartile range (IQR = Q75 – Q25) is the narrowest and located the most central to 1.0 for the Oh and Kang (1987) and the Gergely and Lutz (1968) crack width models. The IQR is the widest for the Eurocode 2 (2004) crack width model. The IQR is located the least central to 1.0 for the JSCE (2007) crack width model.
- The highest rate of unsafe crack width estimation (underestimation) was resulted by the Eurocode 2 (2004) crack width model and the highest rate of conservative crack width estimation (overestimation) was resulted by the JSCE (2007) crack width model.
- The largest overestimation of an individual w_{exp} value was resulted by the NZS 3101 – Frosch (1999) crack width model where $w_{exp}/w_{cal} = 0.18$ was found for $w_{exp} = 0.08$ mm with the estimation of $w_{cal} = 0.45$ mm $= 5.6 \times w_{exp}$.
- The largest underestimation of an individual w_{exp} value was resulted by the Eurocode 2 (2004) crack width model where $w_{exp}/w_{cal} = 3.21$ was found for $w_{exp} = 0.40$ mm with the estimation of $w_{cal} = 0.12$ mm $= w_{exp}/3.21$.
- It can be observed that none of the six crack width models is robust. All the six crack width models overestimate the small actual crack widths and underestimate the large actual crack widths. It can be seen in Figure 6 that the trend lines of the w_{exp}/w_{cal} ratios intersect the horizontal line of 1.0 at around 0.3 mm crack width. Therefore, the crack width estimation w_{cal} for $w_{exp} < 0.3$ mm is conservative, while for $w_{exp} > 0.3$ mm the crack width estimation is unsafe.

It is apparent that the accuracy and the precision of the NZS 3101 – Frosch (1999) crack width model is not optimal. Regarding its accuracy, the crack width estimation is biased towards the conservative ($w_{cal} > w_{exp}$) estimations; the distribution of the w_{exp}/w_{cal} ratio has a strong positive skewness and the median value is $E(w_{exp}/w_{cal}) = 0.83 < 1.0$. Regarding its precision, the coefficient of variation is high (46.6%), and the model may overestimate (mostly the smaller) crack widths by more than 5-times and may underestimate (mostly the larger) crack widths by more than 2-times. Overestimation of the crack widths may result in uneconomic reinforcement ratios. Underestimation of the crack widths, especially since these are usually the larger crack widths, may compromise the serviceability limit state (SLS) performance or the durability performance of the structure. It can be concluded that the most accurate and most precise crack width model is the Oh and Kang (1987) crack width model, based on the calibration against 204 specimens available in the filtered database of McLeod (2019). The present research was based on limited data from twelve literature sources in the filtered database of McLeod (2019), therefore, further studies are suggested with the use of different experimental databases to confirm or disconfirm the findings presented herein.

Table 1: Model error parameters for w_{cal}.

	NZS 3101 Frosch (1999)	JSCE (2007)	EC2 (2004)	Oh and Kang (1987)	Martin et al (1980)	Gergely and Lutz (1968)
SSE*	4.39	3.55	4.41	1.66	2.10	1.79
RMSE*	0.154	0.139	0.147	0.090	0.102	0.094
$E(\lvert w_{exp} - w_{cal} \rvert)$	0.1144	0.1058	0.1066	0.0709	0.0827	0.0724
$m(\lvert w_{exp} - w_{cal} \rvert)$	0.0894	0.0792	0.0764	0.0565	0.0763	0.0608

In Table 1, RMSE is the root mean square error, which is a measure for the model's average deviation from the observed data. The RMSE is the square root of the squared average distance between the observed data and the modelled data, and reads:

$$\mathrm{RMSE} = \sqrt{\frac{\mathrm{SSE}}{\mathrm{n}}} = \sqrt{\frac{\sum_{i=1}^{n}\left(w_{exp} - w_{cal}\right)^2}{n}}$$

Table 2: Model uncertainty statistical parameters for ratio w_{exp}/w_{cal}.

	NZS 3101 Frosch (1999)	**JSCE (2007)**	**EC2 (2004)**	**Oh and Kang (1987)**	**Martin et al (1980)**	**Gergely and Lutz (1968)**
Minimum	0.18	0.20	0.31	0.27	0.27	0.25
5% percentile	0.37	0.33	0.39	0.45	0.43	0.45
25% percentile	0.60	0.57	0.75	0.71	0.69	0.72
Median	0.83	0.77	1.11	0.93	0.93	0.95
75% percentile	1.14	1.01	1.45	1.13	1.28	1.17
95% percentile	1.72	1.28	2.20	1.46	1.62	1.57
Maximum	2.18	1.61	3.21	1.75	2.66	1.84
Range	2.00	1.41	2.91	1.49	2.38	1.59
Mean	0.91	0.79	1.16	0.93	0.99	0.96
Standard deviation	0.425	0.293	0.565	0.308	0.399	0.329
Coeff. of var. (%)	46.6	37.3	48.6	33.2	40.1	34.4
Mode	0.96	1.03	1.60	1.33	0.99	1.21
Skewness	0.836	0.187	0.990	0.131	0.770	0.184
Kurtosis	0.355	-0.477	1.381	-0.449	1.018	-0.203

5 CONSEQUENCES FOR THE SEISMIC DESIGN OF CONNECTIONS

5.1 State-of-the-art and gaps in fastener assessment

ACI 355.2 "Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete" was published in July 2000 as a provisional ACI Standard (ACI 355.2-00) and then reapproved as an ACI Standard in January 2002 (ACI 355.2-01). In that document, simulated seismic tests were introduced both for tension and shear loads. The simulated seismic tests in ACI 355.2-00 received criticism multiple times since the first publication (e.g. Silva, 2001; Hoehler and Eligehausen, 2008; Mahrenholtz, 2012). It was demonstrated that the stepwise-decreasing cyclic tests do not provide very meaningful results in tension (Hoehler and Eligehausen, 2008). If cyclic tension tests for anchors are performed, those should be stepwise-increasing load cycles up to failure. As also pointed out by Silva (2001) the stepwise-increasing load cycling up to failure is preferable because it allows the calculation of stiffness throughout the entire anchor load cycling range selected for testing. A detailed study of the shortcomings of the ACI 355.2-00 simulated seismic tests can be found in the literature (Borosnyoi-Crawley, 2023b). After more than two decades of intense discussions, in April 2023 ACI Committee 355 decided to replace the existing simulated seismic testing protocols with the scientifically more established tests described in EOTA TR 049 for C2 seismic performance category (i.e. tests C2.1a, C2.1b, C2.2, C2.3, C2.4 and C2.5 in Table 2.4 of EOTA TR 049). The ACI CODE-355.2-23 draft was open for public discussion from December 20, 2023 to February 3, 2024. In accordance with the public discussion draft, fulfilment of the C2.1a, C2.1b, C2.2, C2.3, C2.4 and C2.5 requirements result in the anchor qualification for use in seismic design environments, and specifically in structures assigned to Seismic Design Categories (SDC) C, D, E and F in accordance with ASCE/SEI 7. Anchors that do not fulfil these requirements are limited to use in structures assigned to only SDC A and B. The old simulated seismic test protocols have been omitted from the ACI CODE-355.2-23 draft.

This decision of ACI Committee 355 is straightforward and can make an end to a more than two decades long debate around anchor qualification. By implementing exclusively the tests for C2 seismic performance category (as it is currently called in EN and EOTA documents), the updated ACI CODE-355.2 and ACI CODE-355.4 could become the international state-of-the-art documents for seismic qualification of fasteners. It is noted that currently ACI 318-19 (Reapproved in 2022) cites ACI 355.2-19 and ACI 355.4-11 for the qualification of post-installed mechanical and adhesive anchors, respectively. ACI 318 is updated and published in a 3-year cycle since 1992, to accommodate the International Building Code (IBC) update cycle. The next update for ACI 318 is going to be published in early 2025 and it is expected that the updated ACI CODE-355.2 and ACI CODE-355.4 will be cited in ACI 318-25. No information is currently available from EOTA regarding the C1/C2 debate or the international impact of the upcoming changes in ACI CODE-355 and ACI CODE-318. It is noted that EN 1992-4 is currently in the process for a revision in CEN/TC 250, but no details are publicly available of these activities.

5.2 Direct New Zealand impact

The current requirements in New Zealand for post-installed mechanical anchors and post-installed adhesive anchors (Section 17.5.5 in NZS 3101:2006) have been added to the standard in 2006 and have been modified by Amendment 3 in 2017. The requirements at the time of submitting this paper for publication are as follows: “Post-installed mechanical anchors and post-installed adhesive anchors shall pass the prequalification testing stipulated in ETAG 001, Annex E and be designed in accordance with EOTA TR 045”. The rest of Chapter 17 in NZS 3101:2006 provides design rules generally based upon ACI 318, but only covering cast-in-place ductile steel headed studs, headed bolts, hooked bolts and hooked steel plates with diameters less than 50 mm and embedment lengths shorter than 635 mm. ETAG 001 Metal Anchors for Use in Concrete Annex E: Assessment of Metal Anchors under Seismic Action was published in 2013 and has been superseded by EOTA TR 049 in 2016. This information can be found on the EOTA website https://www.eota.eu/etags-archive. EOTA TR 045 Design of Metal Anchors For Use In Concrete Under Seismic Actions was published in 2013 and has been superseded by EN 1992-4:2018 in 2018. This information can be found on the EOTA website https://www.eota.eu/technical-reports. Consequences of the aforementioned are: 1) Seismic design of fastenings (i.e. connections of structural elements and non-structural elements to the supporting concrete structure), which are used to transmit actions to the concrete substrate in accordance with NZS 3101:2006 and utilize either post-installed mechanical anchors or post-installed adhesive anchors shall follow EN 1992-4:2018; and 2) Seismic prequalification testing of post-installed mechanical anchors and post-installed adhesive anchors in accordance with NZS 3101:2006 shall follow EOTA TR 049 “Post-installed fasteners in concrete under seismic action”.

New Zealand is facing today a third paradigm shift in the seismic assessment and design of fasteners. The first happened in 2006 when Section 17.5.5 in NZS 3101:2006 recognized the then state-of-the-art ACI 355.2 recommendations (since 2001). The second happened in 2017 when Amendment 3 has updated the content of Section 17.5.5 in NZS 3101:2006, recognizing the existence of the then state-of-the-art EOTA (European Organisation for Technical Assessment) recommendations (since 2013). The 2024 reality is, again, different. The old simulated seismic test protocols of ACI 355 are expected to be retired now, and this can open new prospects for New Zealand too.

New Zealand should consider doing the same and retire the C1 seismic performance category qualification (i.e. the retired ACI 355.2-00 simulated seismic testing protocols) from NZS 3101. As it was demonstrated in the literature, and now formally admitted by the ACI Committee 355, the EOTA C2 test protocols provide a more realistic approach. It’s time for the next paradigm shift.

Since the Learned Society of Concrete New Zealand established the NZS 3101 Development Group in September 2023 to support the development of the New Zealand Concrete structures standard, everything is available now for the New Zealand engineering community to promote a third, smooth paradigm shift.

5.3 Current chapter in the ‘crack width story’

The research results of Borosnyoi-Crawley (2023a) summarized in this paper highlight some immediate no-go’s and at the same time direct towards the need of further future research. It has been demonstrated that the crack width prediction model of NZS 3101 exhibits the same challenges in the overestimation of small actual crack widths and the underestimation of large actual crack widths, similarly to other well-known crack width proposals. This observation confirms that the direction of the German National Annex to EN 1992-4:2018 and that of the Australian Standard AS 5216:2021 in the selection of seismic performance category for post-installed anchor design based on predicted crack widths from code formulae is a no-go for New Zealand, in the context of the current crack width prediction formula of NZS 3101. It is also presumed that this direction is a no-go with the currently internationally available other crack width prediction models too. The NZS 3101 Development Group must strongly oppose such ideas since currently no scientific evidence is available to support this direction. Notwithstanding that the fastener category known today as ‘C1 seismic performance category’ is going to disappear from the New Zealand practice soon, the selection of seismic performance category based on predicted crack widths will completely lose its relevance anyway. This does not mean, however, that the advancements can stop. There are still multiple gaps in the current state-of-the-art of seismic assessment of fasteners (i.e. the ACI CODE-355 Drafts, version December 2023 for public discussion), directly related to the arbitrarily selected 0.30 mm, 0.50 mm and 0.80 mm crack widths associated with the serviceability and suitability assessments of fasteners. These discrepancies and the potential New Zealand related future work are studied elsewhere (Borosnyoi-Crawley, 2024).

Regarding the relationship between crack widths and reinforcement corrosion, Angst (2019) summarized: “It is currently not possible to stipulate critical crack widths below which corrosion would not occur. Even very thin cracks are zones where transport of chlorides or the carbonation front is accelerated compared to the bulk concrete. Once corrosion starts, the crack width seems to be a minor influencing factor as the kinetics of the corrosion process depend also on a number of properties related to the zones between the cracks (cover depth, concrete transport properties, exposure conditions, etc.).”

Future research related to the current crack width prediction formula of NZS 3101 could be suggested in the following directions:

- Frosch (1999) revealed that for the model calibration generally the same data were used as those of the Gergely and Lutz (1968) study, except that the Rüsch and Rehm (1963) data were not included since the reinforcing steel did not conform to U.S. reinforcement standards, and the goal of the Frosch (1999) study was to compare with bars used in U.S. practice. Consequently, the range of validity of the Frosch (1999) model is limited to its experimental database used for calibration. It is suggested to perform further statistical analyses on larger experimental databases than the 204 test specimens from twelve literature sources in the filtered database of McLeod (2019) used in the present research (Borosnyoi-Crawley, 2023a).
- As it was demonstrated, the accuracy, precision and robustness of the currently available crack width prediction models are not optimal and crack width prediction is apparently still a quite dubious practice. Further research is needed worldwide to identify and capture the main driving parameters of structural cracking and to develop formulae that could provide more precise, but most of all, safer crack width estimation. To support this, publicly available databases of experimental results in an organized fashion would be very useful, similarly to the Tension Lap Splice Database and the Compression Lap Splice Database provided by ACI Committee 408.
- The recommended maximum surface width of cracks at the serviceability limit state for buildings given in Table C2.1 of NZS 3101 might need to be reconsidered, and the future purpose of the recommended crack widths needs further clarification. As it has been demonstrated in the literature, there is no direct correlation between surface crack widths and the rate of corrosion of the reinforcement. Therefore, surface crack width limits are not expected to prevent the corrosion of embedded reinforcing bars. More sensible durability performance parameters are related to permeability and electrical resistance of the concrete, with different diffusion coefficients (oxygen, chloride etc.), water sorptivity or the electrical resistance to limit. Watertightness depends more on the tortuosity of the cracks rather than the surface crack width. A more reasonable watertightness performance parameter could be the flow rate of cracked concrete to limit. The currently listed crack widths in Table C2.1 of NZS 3101 may serve a purpose as aesthetic limits, noting that certain current values are too conservative for such considerations.
- The expected maximum widths of cracks during earthquake events either within or outside the plastic hinge regions are yet to be discovered experimentally. While numerical models are able to reproduce the deformations of buildings relatively precisely, and the number of rotation cycles of e.g. a RC frame column-beam connection during a simulated earthquake event could be determined, numerical approaches are still not useable for the prediction of crack widths. Curvature of sections cannot be translated into crack widths even outside the plastic hinge regions (i.e. linear elastic steel response) without accepting the validity of a given crack width prediction model.
- The currently internationally available crack width prediction models have been experimentally validated on beam, slab and tie specimens under short-term or long-term service conditions, when the stress in the reinforcing bars is expected to be much lower than the yield stress. It is yet to be confirmed if the available models are valid for high tensile stresses in the reinforcing bars, close to their yield stress level – that is outside the original range of validation for these models. Large crack widths resulted from high steel strains during earthquake events have direct impact on the seismic assessment and design of fasteners, and the topic today is still quite a ‘terra incognita’.
- Crack width prediction models are not widely available for reinforced concrete walls and diaphragms. For the seismic design of fasteners in e.g. walls, the currently available assessment methods (ACI 355, EOTA TR 049) cannot provide useful fastener capacity information since those assessment methods were based on numerical simulations performed on RC frames and not on walls

(Wood et al, 2010). Further experimental research is needed to explore the cracking and deformation behaviour of walls and diaphragms especially under simulated earthquake loading.

6 CONCLUSIONS

The interest in, and the research of cracking in reinforced concrete is as old as the material, reinforced concrete itself. Cracking is inherent to concrete and being the main contributor to the composite behaviour of steel reinforcement embedded in concrete. Cracking of reinforced concrete structures due to bending or tension usually has great significance on the structural behaviour. Despite a more than 100 years of intensive research activity, there is no globally accepted formulation for crack width in reinforced concrete. Crack width is typically measured on the outer surface of the concrete structure. Design codes also limit crack widths on the concrete surface with the intention to reduce reinforcement corrosion, water- and air permeability, or for aesthetical reasons. Since crack width varies within the concrete cover and has different value at the level of the reinforcement than that of observed on the concrete surface, as well as the tortuosity of structural cracks strongly depend on the concrete composition and strength, limiting the surface crack width is a dubious practice for corrosion prevention or watertightness control and could support only the aesthetic crack width control. Regarding the steel-to-concrete connections with post-installed fasteners, crack width is a driving parameter for fastener qualification; however, it is not considered in the seismic design of connections. There are examples in the international standardization when seismic performance category selection of fasteners for design is based on predicted crack widths from code formulae. It cannot be demonstrated that such practices are safe in their original contexts. In this research a comparative statistical analysis was performed on the NZS 3101 crack width estimation formula together with another five proposals selected from codes and from the literature, using the same crack width database. It was aimed to demonstrate if the NZS 3101 crack width estimation formula shows similar or different model error characteristics to other models. It was demonstrated that none of the six crack width models is robust. All studied crack width models overestimate the small actual crack widths and underestimate the large actual crack widths. The observations confirmed that the selection of seismic performance category for post-installed anchor design based on predicted crack widths is not a recommended practice and should not be considered for adoption in NZS 3101, mostly for safety reasons. The accuracy, precision and robustness of the currently available crack width prediction models worldwide are not optimal and crack width prediction is apparently still a very uncertain approach. Further international research is needed to identify and capture the main driving parameters of structural cracking and to develop formulae that could provide more precise, but most of all, safer crack width estimation.

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