Durability Design Method Considering Reinforcement Corrosion due to Water Penetration

Hiroshi Ueda¹, Yuya Sakai²*, Koji Kinomura³, Kenzo Watanabe⁴, Tetsuya Ishida⁵ and Toshiharu Kishi⁶

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Abstract

The current durability design for carbonation estimates that dry concrete experiences higher corrosion; however, in actual concrete structures, higher corrosion is observed in wet concrete. To solve this contradiction, a durability design method considering water penetration is proposed, in which the reinforcement is assumed to be corroded by contacting with water. The reinforcement corrosion depth is calculated as the accumulation of the stepwise corrosion. To calculate the number of annual contacts of water with the reinforcement, an equation to calculate the annual frequency of precipitation and a coefficient to take into account the effect of the water supply conditions are established. The comparison of the proposed method with an existing carbonation verification method indicates that the proposed method reflects the situation of an actual concrete structure more appropriately. The proposed durability design method was incorporated in the Standard Specifications for Concrete Structures of the Japan Society of Civil Engineers in 2017.

1. Introduction

Carbonation is known as one of the main causes of reinforcement corrosion in concrete structures (Parameswaran et al. 2008). Generally, when designing a concrete structure, durability should be considered even if the structure is not located in an environment in which it is subjected to chloride attacks or freezing thawing cycles, and the amount of corrosion due to carbonation must be determined at the design stage. In the standard specifications and guides for concrete structures (China CCES01:2004 2004; EN206-1:2000 2000; fib 2006; ISO16204:2012 2012), carbonation is considered as an important factor of deterioration of concrete structure and this has contributed to assure the durability during the service life. In addition, carbonation depth is considered not only in the design of cover thickness, but also in the maintenance of concrete structures, as it can indicate deterioration and the need for repair work (Parrot 1994; Ann et al. 2010; Widyawati et al. 2015).

However, a recent survey on existing structures revealed that almost no correlation exists between carbonation and reinforcement corrosion (Lollini et al. 2016, Stefanoni et al. 2018). Carbonation progresses at a faster rate in a dry environment than under wet conditions (Bertolini et al. 2005; Leemann and Moro 2017); therefore, the verification for carbonation according to the Standard Specifications for Concrete Structures of the Japan Society of Civil Engineers (JSCE-SSCS) indicates that corrosion occurs faster in a dry environment. However, in reality, corrosion occurs faster in a wet environment. Recent reports have indicated that considerable reinforcement corrosion occurs in concrete structures involving a supply of water (Enevoldsen et al. 1994; Jones 1996; Andrade and Castillo 2010). If no water is supplied, nearly no reinforcement corrosion is observed even when the carbonation depth is larger than the concrete cover thickness (Lollini et al. 2016). A dry environment leads to nearly no corrosion on the rebar unless the structure is in a severely corrosive environment (Song et al. 2017). This is because the occurrence of reinforcement corrosion requires water and oxygen (Bentur et al. 1997; Jones 1996). Accordingly, a chapter concerning the effect of water was added in the maintenance section of the JSCE-SSCS that was published in 2013.

Based on the above studies, we propose a new durability design method in this paper considering the effect of water explicitly. This will provide a seamless relationship between the design and the maintenance of concrete structures. It should be noted that reinforcement corrosion in concrete structures also occurs as a result of chloride and chemical attacks (Marchand et al. 2001) and these attacks are strongly connected to the presence of water; however, these phenomena are beyond the scope...
of this work. The proposed design method focuses exclusively on concrete structures that require carbonation verification in the existing durability design approach. The proposed design method was incorporated in JSCE-SSCS in 2017.

2. Overview of the proposed durability design method

In the durability design for reinforcement corrosion due to carbonation, the corrosion depth has been a widely used indicator. However, as mentioned previously, if the concrete is dry, reinforcement corrosion does not exhibit considerable progress even when the carbonation depth is larger than the concrete cover thickness (Lollini et al. 2016). In addition, even in cases in which the carbonation depth is small, the occurrence of electrochemical reactions such as the formation of differential aeration cells (oxygen concentration cells) may cause reinforcement corrosion (Schaschl and Marsh 1960). Therefore, in the proposed durability design method, the reinforcement corrosion depth is employed as an essential indicator. In an environment that is not conducive for chloride and acid attacks, the reinforcement corrosion rate is low, and the amount of the supplied water and oxygen is the governing factor of corrosion progress. Therefore, in the proposed method, considering the penetration of water in the reinforcement, the expression presented as Eq. (1) is used to ensure that the reinforcement corrosion depth is smaller than the limit of the reinforcement corrosion depth, \textit{s}_{lim}, in the design life:

\[
\gamma_1 \times s_d / s_{\text{lim}} \leq 1.0
\]  

where \( \gamma_1 \) is the factor to take into account the structural properties of concrete, and in general, its value may be taken as 1.0 to 1.1, according to that specified for the verification for carbonation in the design section of JSCE-SSCS; \textit{s}_{\text{lim}} is the limit of reinforcement corrosion depth (mm); and \textit{s}_d is the design reinforcement corrosion depth (mm).

The calculation procedures of \textit{s}_{\text{lim}} and \textit{s}_d, which were developed to be easy to apply in practical work, are introduced in Sections 3 and 4, respectively.

3. Calculation of limit of reinforcement corrosion depth

The value of \textit{s}_{\text{lim}} needs to be set appropriately to prevent the degradation caused by reinforcement corrosion, such as the cracking and spalling of concrete cover. However, if the reinforcement corrosion depth at which such degradation occurs is used as the value for \textit{s}_{\text{lim}} stochastically speaking, the actual corrosion depth may be more than this value. Though the reinforcement corrosion depth exceeding this limit does not pose an immediate threat pertaining to the safety and usability of the concrete structure, it may cause some problems, such as human injury and further degradation due to spalling of concrete cover. To prevent such a scenario, in the design, \textit{s}_{\text{lim}} needs to have sufficient allowance in terms of the reinforcement corrosion depth at which deterioration occurs, such as for cracks (corrosion crack). Considering this aspect, the value of \textit{s}_{\text{lim}} was assumed to be 30% of the rebar corrosion depth at which corrosion cracking occurs, that is, \textit{s}_{\text{crack}}:

\[
\textit{s}_{\text{lim}} = 0.3 \times \textit{s}_{\text{crack}}
\]  

A limit of the corrosion amount at which corrosion cracking occurs (\textit{d}_{\text{crack}}), with a value ranging from 10 to 100 mg/cm² was proposed (JSCE 2007b; Plyamahant et al. 2013; Takewaka and Matsumoto 1983), and it was reported that a value of 10 mg/cm² corresponded to a conservative estimation concerning the time of occurrence of corrosion cracking (JSCE 2007b). In addition, \textit{d}_{\text{crack}} was reported to increase with increase in the concrete cover thickness. Although the rate of increase differs depending on the condition (Oh et al. 2009), an increase of 10 mm in the cover was considered to lead to an increase of 10 mg/cm² in \textit{d}_{\text{crack}}. Moreover, it was reported that \textit{d}_{\text{crack}} varies depending on the reinforcement diameter and the type of corrosion product (Zhao et al. 2013). Based on these reports, in our proposed approach, the value of \textit{s}_{\text{crack}} with a 10 mm cover was set to 1.27 × 10^{-2} (mm) (= 10 mg/cm²) as a conservative setting and \textit{s}_{\text{crack}} was considered to increase with the cover. In addition, \textit{s}_{\text{crack}} was considered to increase only until the cover reached a value of 35 mm, and it took a constant value thereafter (\textit{s}_{\text{lim}} = 1.33 × 10^{-2} mm). Generally, \textit{s}_{\text{crack}} is affected by various factors such as the reinforcement diameter, interval of reinforcement, types of corrosion products, lateral cover thickness, and concrete strength. However, in the proposed equation, the cover is the only factor considered to affect \textit{s}_{\text{crack}} to ensure that the user does not need to perform reverification in case the other factors are changed. \textit{s}_{\text{crack}} can be expressed as in Eq. (3).

\[
\textit{s}_{\text{lim}} = 1.27 \times 10^{-2} \times 0.3 \times 0.1 \times c_d (\text{mm})
\]

\[
= 3.81 \times 10^{-2} \times c_d (\text{mm})
\]

where, \textit{c}_d is the design concrete cover used in durability design (mm).

Here, the value of 0.1 in Eq. (3) pertains to the coefficient used to multiply \textit{s}_{\text{crack}} by \textit{x} when the value of the concrete cover is 10 × \textit{x}. \textit{c}_d is calculated using Eq. (4) when the construction error is taken into account.

\[
c_d = c - \Delta c_e
\]

where \textit{c} is the concrete cover (mm), and \Delta c_e is the construction error in the concrete cover (mm).

In the JSCE-SSCS design section, the value of \Delta c_e is 15 mm for a pier and column, 10 mm for a beam, and 5 mm for a slab. \textit{c} should be larger than \Delta c_e when Eq. (4) is applied. Figure 1 shows the relationship between \textit{s}_{\text{lim}} and \textit{c}_d.
4. Design corrosion depth of reinforcement

4.1 Basic concept

Except in the case of concrete structures subjected to severely corrosive environment, water and dissolved oxygen are required for the progress of reinforcement corrosion. Therefore, the following assumptions are considered in the evaluation of the progress of reinforcement corrosion due to water. The proposed durability design method considers directly only the effect of liquid water; however, the effect of dissolved oxygen is considered implicitly. This point is explained in detail in later sections.

The assumptions can be listed as follows:

(a) The water with dissolved oxygen permeates from the concrete surface, and reinforcement corrosion begins when the water reaches the reinforcement.
(b) When the water evaporates from the reinforcement by drying or the dissolved oxygen is consumed, the process of reinforcement corrosion stops.
(c) The process of reinforcement corrosion starts again when the water with dissolved oxygen permeates into concrete and reaches the reinforcement.
(d) The reinforcement corrosion progresses by the repetition of this procedure.

As described later, the water penetration rate was calculated on the basis of the rate of increase of the liquid water depth, as determined from the position where the moisture content increases significantly during/after the water absorption test. In this work, we assumed that the water reached the reinforcement when the calculated water depth was equal to or larger than the cover thickness. Because the relationship between the water content and reinforcement corrosion is complicated, for simplicity, corrosion is assumed to occur when the liquid water, which contains dissolved oxygen, reaches the reinforcement. In the proposed method, the corrosion due to humidity was ignored because its corrosion rate is much smaller compared to corrosion due to rain water (Andrade and Castillo 2010).

Figure 2 shows the occurrence of the reinforcement corrosion in accordance with the abovementioned mechanisms. In these mechanisms, it is assumed that the contact of the structure with water with dissolved oxygen causes reinforcement corrosion; therefore, the reinforcement does not corrode unless the water reaches the reinforcement. The permeated water cannot reach the reinforcement when using a water supply in which the water is supplied to concrete surface for a short time or when the concrete has a dense pore structure and the water penetration rate is low. It is known that reinforcement corrosion hardly progresses when the environment around the reinforcement is kept wet or dry (Jones 1996; Andrade and Castillo 2010; Ahlström et al. 2016; Stefanoni et al. 2018), as seen in actual concrete structures (ACI 2014; Broomfield 2006). In contrast, if the concrete undergoes wetting and drying cycle, the rate of reinforcement corrosion increases (FHWA 2000). The proposed durability design method can reflect these aspects, and in this manner, the movement of oxygen is considered indirectly, even though this design method focuses explicitly on the movement of water.

4.2 Calculation of design value

The previous section described the employed assumption that the reinforcement corrosion progresses every time water reaches the reinforcement. In this case, the corrosion depth of the reinforcement can be expressed as the accumulation of the reinforcement corrosion depth owing repeated contact with water. Therefore, the design value of the reinforcement corrosion depth at the end of service life, $s_d$, can be expressed as a product of the service life and the design value of reinforcement corrosion depth in one year, $s_{dy}$, as shown in Eq. (5).
\[ s_d = \gamma_w \times s_{dy} \times t \]  

(5)

where \( \gamma_w \) is the safety factor used to account for the variation in \( s_d \) (in general, its value may be taken as 1.15, and in the case of high fluidity concrete, the value of the factor may be taken as 1.1); \( s_{dy} \) refers to the \( s_d \) per year \((\text{mm}/\text{y})\); \( t \) is the service life considering reinforcement corrosion due to water permeation \((y)\) (in general, its value can be up to a hundred years).

The values of \( \gamma_w \) and \( t \) were set to be the same as those in the durability design for carbonation in the JSCE-SSCS-design section. \( \gamma_w \) was set to be the same as \( \gamma_{cb} \) (safety factor to account for the variation in the design carbonation depth \( y_{c(d)} \)). \( s_{dy} \) was set to a constant value because its change over time is small before the corrosion crack is initiated in concrete. However, if it is known at the design stage that \( s_{dy} \) changes considerably during the design service life, the appropriate value should be set. \( s_{dy} \) can be expressed as in Eq. (6) assuming that the reinforcement corrosion depth increases gradually owing to the repeated contact of water with the reinforcement:

\[ s_{dy} = s_{dy1} \times N_w \]  

(6)

where \( s_{dy} \) is the reinforcement corrosion depth induced by one contact of water with the reinforcement, \( N_w \) is the number of times the water contacts the reinforcement for one year. The calculation of \( s_{dy1} \) is summarized in Section 4.3, and those of \( N_w \) and \( s_{dy} \) are summarized in Sections 4.4 and 4.5, respectively.

4.3 Corrosion depth of reinforcement induced by one contact with water

Nishikata et al. (1994) studied the corrosion rate of reinforcement subjected to wetting and drying cycles; they reported that the corrosion rate was large immediately after the reinforcement was immersed into water, and it decreased during the drying cycle. Subsequently, the corrosion rate increased suddenly just before the water evaporated completely and then decreased again. This indicates that in one contact with water, the reinforcement corrosion rate changes in a complex manner. In addition, the water evaporation rate and the effect of suppression of the reinforcement corrosion by the consumption of the dissolved oxygen changes depending on the environment. Therefore, it is difficult to estimate \( s_{dy1} \) accurately considering these conditions. Consequently, in the proposed durability design method, \( s_{dy1} \) was obtained by calculating the reinforcement corrosion rate and setting the time for reinforcement corrosion.

Tottori et al. (2004) reported that the corrosion rate of reinforcement was \( 2.3 \times 10^{-1} \text{ mm}/\text{y} \) when the carbonation depth was larger than the concrete cover thickness. Stefanoni et al. (2018) reviewed 53 papers that investigated the rate of steel corrosion in concrete and concluded that corrosion rate varies widely and is dependent upon the testing conditions. The corrosion rates ranged between 0.002 and 20 \( \mu\text{A}/\text{cm}^2 \). According to Faraday's law (Jones 1996), assuming that the atomic weight and valence of iron are 55.85 and 2, respectively, and that the density of the steel is 7.87 \( \text{g/cm}^3 \), 1 \( \mu\text{A}/\text{cm}^2 \) can be converted to \( 11.6 \times 10^{-3} \text{ mm}/\text{y} \). Using this relationship, the range of corrosion rates can be converted to between \( 2.3 \times 10^{-3} \) and \( 2.3 \times 10^{-1} \text{ mm}/\text{y} \), and the geometric mean of these minimum and maximum values is \( 2.3 \times 10^{-3} \text{ mm}/\text{y} \), which corresponds to the corrosion rate reported by Tottori et al. (2004). The rate was reduced to approximately one-tenth the original value when the remaining non-carbonated cover thickness was 5 to 13 mm. These authors also performed a literature survey and reported that the rate was approximately \( 2.3 \times 10^{-3} \text{ mm}/\text{y} \) when the carbonation depth was larger than the concrete cover thickness and approximately 1 to \( 6 \times 10^{-3} \text{ mm}/\text{y} \) when the specific resistance of mortar was 20 kΩm. Yonezawa et al. (2014) estimated the corrosion rate of reinforcement in carbonated concrete to be 0.15 to 0.65 mg/cm²/y for concrete kept indoor and 0.45 to 1.10 mg/cm²/y for concrete exposed to the external environment. The corrosion rate of reinforcement was noted to decrease with increase in the concrete cover thickness.

Furthermore, Mihara et al. (2002) studied the corrosion rate of a steel bar in a noncarbonated condition by studying the corrosion rate of the steel bar in a solution extracted from cementitious material. The rate was approximately 0.3 \( \mu\text{m}/\text{y} \), as evaluated from the generation rate of gas, and approximately 0.6 \( \mu\text{m}/\text{y} \), as evaluated by the weight change 30 days from the beginning of the test. If the corrosion rate is set assuming the carbonated condition, the rate will be excessively large when the carbonation depth is smaller than the concrete cover thickness; therefore, in our proposal, based on the experimental results pertaining to the noncarbonated condition, the corrosion rate of the reinforcement was set as \( 5.8 \times 10^{-3} \text{ mm}/\text{h} = 5.1 \times 10^{-3} \text{ mm}/\text{y} = 0.4 \text{ mg/cm}^2/\text{y} \).

In the scenario considered, if the carbonation reaches the reinforcement during the service life, the actual corrosion rate can be larger than the set value. However, if an appropriate cover thickness is assigned through the verification, and the expected concrete cover thickness is attained with an acceptable construction error, the carbonation will never reach the reinforcement in the service life. Furthermore, the large carbonation depth indicates that the concrete is subjected to long-term drying conditions, and the reinforcement is barely corroded under such conditions (ACI 2014; Broomfield 2006). Considering these aspects, the set value of the corrosion rate of the reinforcement could be considered suitable.

The duration of the contact of water with the reinforcement at one permeation occurrence is difficult to estimate because it varies depending on the duration of water supply on the concrete surface, penetration rate of water into concrete, evaporation rate of the penetrated water, and consumption rate of the dissolved oxygen. Thus, in the proposed approach, it was assumed that the water supply to the concrete surface was a result of rainfall and the duration of water supply to the concrete surface was equal to the duration of the water being in contact with the reinforcement. Here, the frequency
distribution of the rainfall duration was based on an assessment that is introduced in the next subsection. The median value of this distribution was 10 h. Further, the water contact duration of one rainfall occurrence was set to 10 h multiplied by 1.2 for allowance. Thus, \( s_{01} \) can be calculated by multiplying the corrosion rate of reinforcement, \( 5.8 \times 10^{-8} \text{ mm/h} \) by 12, leading to a value of \( 70 \times 10^{-7} \text{ mm} \).

### 4.4 Number of times water comes into contact with reinforcement in one year

As mentioned previously, in the proposed approach, it was assumed that the water supply occurred as a result of rainfall. This section discusses the number of times water comes into contact with the reinforcement in one year, \( N_w \). For water to reach the reinforcement, a continuous rainfall duration higher than the time required for water to reach the reinforcement is necessary; therefore, \( N_w \) was set by analysing the number of rainfall occurrences whose durations were higher than a certain time based on past meteorological data, which consisted of the hourly precipitation record obtained from the website of the Japan Meteorological Agency (JMA 2019). Considering the variety of the total precipitation, the frequency of precipitation, and territorial characteristics, six cities (Sapporo, Tokyo, Osaka, Niigata, Takamatsu, Naha) in Japan were selected, and the precipitation record for 10 years (1 January 2007 to 31 December 2016) for these cities was analysed to estimate the annual variation.

Here, the precipitation record includes not only rainfall, but also snowfall. Our proposal is based on the basic understanding that the water and dissolved oxygen supplied by the wetting and drying of concrete causes reinforcement corrosion (Fig. 2); therefore, snowfall was not considered to be a part of precipitation in the analysis. Precipitation was judged as snow when the temperature at the beginning of the precipitation was less than 0°C because it is known that water becomes snow when the temperature is 0°C regardless of the relative humidity on the ground (Harpold et al. 2017). Following this assumption, for example, when rainfall changed into snowfall owing to decrease in the temperature, it was regarded as continuous rainfall. On the other hand, if snowfall changed into rainfall owing to increase in the temperature, it was regarded as snowfall.

\( N_w \) can differ depending on the definition. For example, in case rain is regarded to have stopped when the rainfall stops, the value of annual precipitation becomes the largest; the number of short precipitations increases, and the continuous precipitation duration becomes shorter. On the other hand, in case rain is regarded to have stopped when the rainfall stops for a certain period of time, the value of annual rainfall decreases but the duration of rainfall becomes longer. The former case leads to a conservative evaluation regarding when the water reaches reinforcement in a longer rainfall, such as for a structure with regular cover thickness or pore structure. Therefore, in the proposed, to provide a conservative evaluation for concrete with a general quality and cover thickness, the continuous rainfall was regarded to end when rainfall stopped for more than three hours.

Considering the data of each of the six cities, the relationship between the continuous precipitation duration \( T \) in the last 10 years eliminating snowfall and the annual precipitation of continuous rainfall longer than \( T \), \( N_w \), was determined. Figure 3 shows the total precipitation recorded for the last 10 years in the six cities and the envelope curve. \( N_w \) is calculated using Eq. (7) regardless of the location of the concrete structure.

\[
N_w = 262 \times \exp(-0.068T)
\]  

where \( N_{w17} \) is the number of occurrences of annual continuous precipitation longer than \( T \), and \( T \) is the duration of continuous precipitation (h); if the water takes time \( T \) (h) to permeate to the level of the reinforcement, then \( N_w = N_{w17} \). The envelope curve is noted to cover almost all precipitation data, including the annual variation of each city. Safer evaluation is ensured by using the safety factor \( \gamma_w \), which considers the variation of the design corrosion depth of reinforcement, as discussed in Section 4.5.

### 4.5 Design reinforcement corrosion depth per year

Substituting the values and equations obtained in Sections 4.3 and 4.4 into Eq. (6), \( s_{dy} \) can be calculated using Eq. (8).

\[
s_{dy} = 1.9 \times 10^{-4} \times F_w \times \exp (-0.068T_p)
\]  

where, \( F_w \) is the coefficient used to take into account the local environmental condition, and \( T_p \) is the time required for the water permeated into concrete to reach the reinforcement (h).

The coefficient, \( 1.9 \times 10^{-4} \), in Eq. (8) was obtained by multiplying \( s_{dy} = 7.0 \times 10^{-7} \) and the value 262 in Eq. (7) and truncating the number to two decimal places. In the verification, the same value was used for \( T \) in Eq. (7) involving \( T_p \), \( F_w \) is an indicator set to consider the dif-

![Fig. 3 Total precipitation record in six cities during the last 10 years and the envelope curve.](image-url)
where, therefore, a simple classification was employed. If structures exist, and their classification is complex; Table 1

| Type | Classification and example of concrete member | \( F_w \) |
|------|-----------------------------------------------|------------|
| 0    | Always dry                                    | 0.7        |
|      | - Concrete member is not subjected to water supply and dew condensation water except in cases in which waterproofing is lost. |            |
| I    | Experiences wetting and drying cycles         | 1.0        |
|      | - The concrete member is often subjected to water; however, the member dries after water supply is stopped. |            |
|      | - General concrete structures such as bridges and those not classified as 0, II and III subjected to water supply for more than 12 hours continuously and experiences frequent wetting and drying cycle. |            |
|      | - A portion subjected to long-term water supply due to leaked water from superstructures etc. even after water supply is stopped. |            |
|      | - Columns or piers in rivers, subjected to wetting and drying cycle. |            |
| III  | Always wet                                    | 0.7        |
|      | - The member is always wet; it experiences few frequent wetting and drying cycles and oxygen supply to reinforcement is limited. |            |
|      | - Portion is always under water including ground water. |            |

Here, by substituting \( A = \sqrt{(r \times \gamma \times \cos \theta)(2 \times \eta)} \), Eq. (10) is obtained.

\[
L = A \times \sqrt{t_i}
\]

(10)

Assuming that this equation is applicable to concrete, \( T_p \) for \( c_d \) in the durability design can be expressed as in Eq. (11) using the design water penetration rate coefficient \( q_{de} \).

\[
c_d = q_d \times \sqrt{T_p}
\]

(11)

where, \( q_d \) is the design water penetration rate coefficient (mm/\( \sqrt{h} \)).

From Eq. (8) and Eq. (11), the equation to calculate \( s_{dy} \) can be obtained as follows.

\[
s_{dy} = 1.9 \times 10^{-4} \times F_w \times \exp (-0.068 \times c_d^2/q_d^2)
\]

(12)

### 4.7 Design water penetration rate coefficient

To calculate \( q_{de} \), the water penetration rate must be defined. Thus, using the reported experimental data, the water penetration rate was determined to obtain \( q_{de} \). Based on the data of experiments that involved the measurement of the water penetration with time, the water penetration rate was calculated using Eq. (13) (Sakai et al. 2017).

\[
A = 5 \times (W/B)^2 \times \beta_1^2
\]

(13)

where \( W/B \) is the water to binder ratio, and \( \beta_1 \) is the coefficient used to consider the curing condition and effect of the environment.

The comparison between \( A \) calculated using Eq. (13) and that calculated from the data in the literature (Koshikawa and Ogihara 1991; Sakai and Kishi 2016; Sakai et al. 2017; Suzuki and Ueda 2015; Tsuchiya et al. 2008; Ueda and Suzuki 2016a; Zhang et al. 2011) is shown in Fig. 4. The test conditions used in these studies are summarized in Sakai et al. (2017). Thus far, the number of studies in literature that have reported the rate of increase of the water penetration depth with time is limited; to the best of the authors’ knowledge, the data reported here constitute all the literature that the authors could find. The specimens were subjected to each curing condition after the demoulding, which was done 24 h after casting. The value of \( \beta_1 \) was set to one for water curing, two for sealed or high humidity curing, and three for curing in the air. The details regarding the curing conditions and \( \beta_1 \) are listed in the literature by Sakai et al. (2017). In the figure, most of the data can be noted to be within the range of ±150% of the A calculated using Eq. (13), though some data calculated from the experimental results are not in this range. These data correspond to specimens subjected to severe drying, such as drying at 105°C or for several years, before the water penetration test is carried out. Therefore, this gap is likely caused by the absence of a factor to consider the effect of drying. To this end, when the concrete was subjected to severe drying, it was assumed that the water retained by the
curing was evaporated, and \( \beta_1 \) was set to 3, which is the same as that for a specimen cured in air, regardless of the actual curing condition. The result obtained is shown in Fig. 5, and all data can be noted to be within the range of \( \pm 150\% \) of A calculated using Eq. (13). Eq. (13) can be applied to concrete made with ordinary Portland cement (OPC), OPC replaced with blast furnace slag up to a volume ratio of 50\%, OPC replaced with ash up to a volume ratio of 15\%, and concrete with \( W/B = 0.4 \) to 0.7.

The curing condition that corresponds to the standard curing mandated in the construction section of JSCE-SSCS (demoulding at the age of three days for concrete involving OPC) was assumed to be between air curing (\( \beta_1 = 3 \)) and sealed curing (\( \beta_1 = 2 \)), and \( \beta_1 \) was set to 2.5. Using \( \beta_1 = 2.5 \), Eq. (13) can be simplified and the equation to calculate \( q_p \) can be expressed as Eq. (14).

\[
q_p = 31.25 \times (W/B)^2 \quad (14)
\]

where \( q_p \) is the predicted value of water penetration rate coefficient (mm/\( \sqrt{h} \)).

The design value of the water penetration rate coefficient \( q_d \) can be calculated using the following equation.

\[
q_d = \gamma_c \times \beta \times q_k \quad (15)
\]

where \( \gamma_c \) is the factor used to account for the variation in concrete material (in general, it may be set as 1.3), \( \beta \) is the coefficient used to consider the curing condition and the effect of environment, and \( q_k \) is the characteristic value of the water penetration rate coefficient (mm/\( \sqrt{h} \)).

**Table 2 Values of coefficient \( \beta \) to consider curing condition and effect of environment.**

| Environmental condition | \( \beta \) |
|--------------------------|-----------|
| Curing condition corresponds to standard curing mandated in the construction section JSCE-SSCS**. | 1.0 |
| In case the abovementioned standard curing cannot be applied. | 1.5 |
| Better curing than the abovementioned standard curing condition. | 0.7 |

*If the structure is exposed to external environment involving severe conditions such as temperature of higher than 40°C and that involving wetting and drying cycles, \( \beta = 1.5 \) regardless of the curing condition.

**Standard Specifications for Concrete Structures of the Japan Society of Civil Engineers

***Curing condition corresponds to the sealed condition until the age of 28 days.

Here, the value for \( \gamma_c \) is taken from that used in the verification of carbonation in JSCE-SSCS design section. \( \beta \) is set as seen in Table 2 considering Eq. (13), Eq. (14) and the value of \( \beta_1 \).

Here, \( \beta_1 = 1.0 \) corresponds to \( \beta = 0.2 \); however, it is not clear whether the high resistance of the concrete cover against water penetration, which is induced by water curing, is ensured for a large duration. Therefore, in

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Fig. 4 Comparison between experimentally obtained water penetration rate A reported in the literature and A calculated using Eq. (13), without consideration of the effect of severe drying.

Fig. 5 Comparison between experimentally obtained water penetration rate A reported in the literature and A calculated using Eq. (13), with consideration of the effect of severe drying.
Table 2, β = 0.7 is set as the minimum value. However, β = 0.2 can be used if high resistance against water penetration is ensured.

Substituting Eq. (13) into Eq. (10) and reorganizing in terms of β, L can be expressed using Eq. (16).

\[ L = 31.25 \times \beta \times (W/B)^2 \times \sqrt{t_i} \]  

(16)

By using this equation, the depth of water penetration into concrete can be estimated. The effect of W/B and the curing condition on the water penetration depth is shown in Figs. 6 and 7, respectively. In the verification of water penetration using Eq. (12), L is not calculated explicitly. However, it is important to understand the effect of concrete quality on the time dependence of the water penetration depth. It is known that the relationship between water penetration into concrete and the square root of time does not cross the origin and has an intercept (McCarter et al. 1992). However, even though it depends on the experimental conditions, the intercept was approximately 10 mm and the calculation of \( L \) can be considered reasonable. \( \beta \), W/B, and \( F_w \) are variables representing the effect of curing, mixing design and environmental conditions, respectively. Each of these variables are effectively independent. However, when concrete experiences severe drying, \( \beta \) becomes large regardless of curing condition; therefore, \( \beta \) can be affected by environmental conditions. Only a very severe drying environment such as one lacking a water supply for years or that experiences drying at 105°C can increase \( \beta \). Such conditions are not realistic when the corrosion due to water penetration must be considered; therefore, these variables can be regarded as independent.

5. Verification of proposed durability design method

5.1 Preliminary calculation using the proposed equations

This section describes the calculation and comparison of the concrete cover thickness that satisfies the verification requirements of water penetration and carbonation, by using the introduced equations. The coefficients and replacement rates listed in Table 3 were used. The values of W/B, \( F_w \) and \( \beta \) used in the calculation are listed in Table 4. Figure 8 shows the effect of W/B and the type of cement on the cover thickness. In the proposed durability design method, because the water penetration rate is determined using W/B, the calculated concrete cover is the same regardless of the cement type. In the proposed durability design method, the concrete made with OPC has almost the same concrete cover as that set considering the durability design for carbonation, and the concrete cover becomes smaller when W/B is higher than 0.5. This indicates that the concrete cover can be reduced in a design considering the water penetration.

In the durability design for water penetration considering concrete using Portland blast-furnace slag cement of type B (BB) and Portland fly-ash cement of type B (FB), the concrete cover becomes smaller than that cal-

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**Table 3** Value of parameters used in the estimation of the cover thickness.

| Type             | Symbol | Description                  | Value |
|------------------|--------|-------------------------------|-------|
| Water penetration| \( \gamma \) | Structural factor            | 1.0   |
|                  | \( \gamma_w \) | Safety factor for \( x_d \) | 1.15  |
|                  | \( \Delta c_e \) | Construction error in cover | 5     |
|                  | \( \gamma_c \) | Concrete material factor      | 1.3   |
| Carbonation      | \( \beta_e \) | Environment factor            | 1.6   |
|                  | \( \gamma_c \) | Concrete material factor      | 1.0   |
|                  | \( \gamma_{sb} \) | Safety factor for \( y_d \) | 1.15  |
|                  | \( \delta \) | Remaining non-carbonated cover| 10    |
| Both             | \( t \) | Service life                  | 100   |

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**Table 4** Value of parameters used in the estimation of the water penetration rate.

| Symbol | Description                              | Value |
|--------|------------------------------------------|-------|
| W/B    | Water to binder ratio                    | 0.4 - 0.7 |
| \( F_w \) | Coefficient to consider local water supply | 0.7 - 1.3 |
| \( \beta \) | Coefficient to consider curing and condition and environment | 0.2 - 1.5 |
culated using the durability design for carbonation when $W/B$ is equal to or larger than 0.4. For example, when the calculated concrete cover in the durability design for carbonation is 63 mm for FB and 55 mm for BB with $W/B = 0.55$, the cover is reduced to 38 mm in the proposed durability design. It is assumed that the standard curing described in Section 4.7 is applied on site. So far, for some cases, the durability design for carbonation led to excessive cover thickness when blended cement was used because a larger carbonation depth has been reported in concrete involving blended cement (Horiguchi et al. 1994; Lye et al. 2015, 2016). The durability design for water penetration can avoid such problems, and the performance of the blended cement can thus be appropriately evaluated.

**Figure 9** shows the effect of $F_w$ on the concrete cover. $F_w$ was set to 1.0 for a concrete structure in a general environment, 0.7 for a condition involving less water supply, and 1.3 for a condition in which water supply occurs frequently. It can be noted that $F_w$ has a significant influence on the concrete cover.

**Figure 10** shows the effect of $\beta$ on the concrete cover. It has been indicated that setting $\beta$ to 0.2 for concrete cured under water, to 0.7 for concrete that is seal cured or cured in high humidity, and to 1.5 for concrete exposed to the air 24 h after casting leads to a satisfactory prediction of the water penetration rate (Sakai et al. 2017). Applying these values in the design, as shown in the figure, the value of $\beta$ of 1.5, 0.7 and 0.2 lead to a cover thickness of 53 mm, 30 mm, and 14 mm, respectively, when $W/B$ is 0.55.

### 5.2 Comparison of the proposed durability design method with the existing method

In the proposed durability design method, the water penetrated into the concrete leads to the occurrence of a small amount of corrosion to the reinforcement, and the reinforcement corrosion depth is calculated as the accumulation of this process. To justify the proposed durability design method, one method is to compare the obtained results with experimental results pertaining to the monitoring of the water penetration and reinforcement corrosion over a long period; however, no such report exists yet.

However, the premise of the durability design method is for it to be used in practical design; therefore, the most critical aspect is to determine the appropriate cover thickness. Moreover, if this durability design method is replaced with that for carbonation, it should reflect the actual situation of degradation in concrete structures. Therefore, in this section, the validity of this durability design method is discussed by comparing it with that for carbonation.

First, the tendency of reinforcement corrosion in existing structures and the results pertaining to each durability design method are compared. As introduced in Section 1, the existing durability design for carbonation assumes that the reinforcement corrosion occurs at a higher rate when the concrete is dry; however, this is in contrast to the actual situation, in which faster corrosion occurs in concrete that comes into contact with water. In the durability design for water penetration, the reinforcement corrosion is slower in dry concrete subjected to less water supply; therefore, the actual situation is reflected, and appropriate information is transmitted for the maintenance. In addition, when a water supply exists, the occurrence of wetting and drying cycle leads to faster corrosion compared to that for concrete in a constant wet condition. This aspect is not considered in the existing durability design for carbonation; however, it is consid-
In the proposed durability design method via \( N_w \) and \( F_w \). Faster corrosion with wetting and drying cycles is attributable to the combined effects of liquid water and dissolved oxygen. \( N_w \), the annual number of instances wherein the water reaches the reinforcement, was employed in the proposed method to express this tendency rationally. \( F_w \) was also employed to consider the change in \( N_w \) due to the local environmental conditions with the expectation of attracting attention to the importance of frequency and duration of water supply. As a result, the tendency of the corrosion in the existing structures is expressed more appropriately.

Second, we consider the obtained cover thickness. To apply a durability design method in an actual design, it is important to determine the appropriate cover thickness for ensuring the durability. The cover thickness given by the proposed durability design method is, as shown in Fig. 8, smaller than that given in the durability design for carbonation when OPC is used, \( W/B \) is equal to or larger than 0.55, and standard curing is employed. Even if \( W/B \) is equal to or larger than 0.55, the resistance of concrete against the water penetration is high if an appropriate curing method is employed. In the considered case, the proposed method is applicable to concrete having a value of \( W/B \) equal to or less than 0.7. The maximum \( W/B \) specified in the sections on design and construction in the JSCE-SSCS is 0.65, therefore, the cover thickness obtained in the durability design of water penetration is acceptable for practical design. Figure 8 shows that a larger cover thickness is obtained when \( W/B \) is 0.4 compared to that obtained by the durability design for carbonation. This can be an underestimation of the quality of concrete with such low \( W/B \) but according to the experiment using such concrete, water rapidly penetrated up to a depth of 10 mm (Koshikawa and Ogihara 1991). This indicates that the cover thickness is not necessarily overestimated. In the practical design, to set the concrete cover in accordance with the design section of JSCE-SSCS, the minimum cover thicknesses must be determined considering, in addition to the durability design, the diameter of the reinforcement and the maximum size of gravel. Therefore, the cover thickness given by the durability design method cannot be excessively high for an actual structure with \( W/B \) of 0.4. As shown in Fig. 9, the cover thickness obtained with \( F_w \) of 1.3 is larger than that obtained by the durability design for carbonation when \( W/B \) is from 0.4 to 0.7. This indicates that the cover thickness cannot be smaller than that set in the durability design for carbonation when the concrete is often subjected to water. As a result, the cover thickness obtained in the proposed durability design method can be made smaller by setting \( F_w \) equal to or smaller than 1.0 than that given by the durability design for carbonation.

As shown in Fig. 10, the concrete cover obtained using the proposed approach became larger than that obtained using the durability design for carbonation when the appropriate curing method was not employed \((\beta = 1.5)\) with \( W/B = 0.4 \) to 0.7. These results indicate that the proposed durability design method for water penetration is more suitable compared to that for carbonation when applied for actual design. The proposed durability design method was incorporated in JSCE-SSCS in 2017.

6. Conclusions

The conclusions derived from this study can be summarized as follows.

(1) A durability design method for water penetration was proposed based on the fact that the reinforcement corrosion in concrete structure is affected more by water than by carbonation.

(2) In the durability design method, the reinforcement is assumed to be corroded little by little by water with dissolved oxygen reaching the reinforcement, and the reinforcement corrosion depth is calculated as the accumulation of this piecewise corrosion. In this estimation, a constantly wet or dry condition leads to slower corrosion, whereas a condition involving wetting and drying cycles leads to faster corrosion of the reinforcement. In the durability design for carbonation specified in the current JSCE-SSCS, the corrosion in a concrete structure occurs at a higher rate in the dry condition. In the proposed durability design method, the concrete under dry conditions has a slower corrosion rate, which is in agreement with the actual situation.

(3) In the proposed durability design method, as an indicator of the concrete quality, besides the water to binder ratio, a coefficient to account for the curing condition and the environment effects is employed. Moreover, the conditions of supply of water to the concrete are categorized, and a coefficient to consider these condition is employed to ensure that the durability design takes into account the effect of the water supply conditions.

(4) To calculate the number of times the water contacts the reinforcement annually, an equation to calculate the annual frequency of precipitation using the duration of continuous precipitation was proposed. In this equation, a short break in the precipitation was ignored, and snowfall was not considered to be a part of precipitation.

(5) An equation was proposed to calculate the water penetration depth into concrete as a function of the time of contact of the concrete with water, water to binder ratio of concrete, curing condition, and environmental factors.

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Notations

| Symbol | Definition |
|--------|------------|
| $c$    | concrete cover (mm) |
| $c_d$  | design value of concrete cover used in durability design (mm) |
| $\Delta c_d$ | construction error in concrete cover (mm) |
| $d_{crack}$ | limit of corrosion amount at which corrosion cracking occurs (mg/cm²) |
| $F_w$ | coefficient to consider the local environmental condition |
| $L$  | penetration depth (m) |
| $N_w$ | number of times water contacts the reinforcement in one year |
| $N_{wcr}$ | number of times annual continuous precipitation occurs for a duration longer than $T$ |
| $q_d$ | design water penetration rate coefficient (mm/√h) |
| $q_c$ | characteristic value of the water penetration rate coefficient (mm/√h) |
| $q_p$ | predicted value of water penetration rate coefficient (mm/√h) |
| $r$  | pore radius (m) |
| $s$  | reinforcement corrosion depth (mm) |
| $s_{crack}$ | rebar corrosion depth at which corrosion cracking occurs (mm) |
| $s_d$ | design reinforcement corrosion depth (mm) |
| $s_{d1}$ | reinforcement corrosion depth owing to one contact of water with the reinforcement (mm) |
| $s_{dy}$ | $s_d$ per year (mm/√h) |
| $s_{lim}$ | limit of reinforcement corrosion depth (mm) |
| $t$  | service life considering reinforcement corrosion due to water permeation (y) |
| $T$  | duration of continuous precipitation (h) |
| $t_p$ | penetration time (s) |
| $T_p$ | time taken for the water permeated into the concrete to reach the reinforcement (h) |
| $W/B$ | water to binder ratio |
| $\gamma_c$ | factor to account for variation in concrete material |
| $\gamma_e$ | safety factor to account for variation in the design carbonation depth $y_d$ |
| $\gamma_f$ | factor to account for structural properties of concrete |
| $\gamma_d$ | design carbonation depth (mm) |
| $\beta$ | coefficient to consider curing condition and the effect of environment |
| $\gamma$ | surface tension of the liquid (N/m) |
| $\theta$ | contact angle (°) |
| $\eta$ | viscosity of the liquid (Pa·s) |