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Assessment of concrete damage and strength degradation caused by reinforcement corrosion

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Abstract. Structural performance deterioration of reinforced concrete structures has been extensively investigated, but very limited studies have been carried out to investigate the effect of reinforcement corrosion on time-dependent reliability with consideration of the influence of mechanical characteristics of the bond interface due to corrosion. This paper deals with how corrosion in reinforcement creates different types of defects in concrete structure and how they are responsible for the structural capacity deterioration of corrosion affected reinforced concrete structures during their service life. Cracking in cover concrete due to reinforcement corrosion is investigated by using rebar-concrete model and realistic concrete properties. The flexural strength deterioration is analytically predicted on the basis of bond strength evolution due to reinforcement corrosion, which is examined by the experimental data available. The time–dependent reliability analysis is undertaken to calculate the life time structural reliability of corrosion damaged concrete structures by stochastic deterioration modelling of reinforced concrete. The results from the numerical example show that the proposed approach is capable of evaluating the damage caused by reinforcement corrosion and also predicting the structural reliability of concrete structures during their lifecycle.

1. Introduction

Deterioration of reinforced concrete (RC) structures due to reinforcement corrosion is a growing problem worldwide. It typically involves cracking and spallling of concrete cover and reduction in area of steel reinforcement and loss of bond between concrete and corroded steel. This eventually affects the service life of the concrete structures and also increases the resources required for the maintenance and rehabilitation over time [1-5]. Managing corrosion damaged RC structure has become a great challenge both financially and technically. RC structures damaged by reinforcement corrosion compromises structural safety and durability by affecting their performance. The cost associated with managing these corrosion damaged RC structure (repair, rehabilitation, demolition) is in billion dollars [6-9]. For optimum and cost effective infrastructure management, time-dependent reliability analysis is considered as the effective tool. In time-dependent reliability analysis of deteriorating RC structures, the quantification of the damage caused by reinforcement corrosion is essential. Among these types of damage, only cracking in the concrete cover is visible and can be measured without affecting the functionality of the RC structures in operation.

Cracking in concrete cover is an important parameter which helps in condition monitoring of the RC structures. It is necessary to predict the internal damages such as residual strength deterioration

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from the observable surface condition during the routine inspection or maintenance process. Therefore, it is always beneficial to establish a prediction method to quantitatively assess the structural performance by assessing cracking in the concrete cover. In order to evaluate the residual strength of corroded RC structures, considerable investigations have been undertaken in the past two decades, mainly focusing on the relationship between rebar mass loss and residual strength [1-4]. However, limited research has focused on the relationship between cracking in the concrete cover surface and residual strength and its effect on the structural reliability of these corrosion damaged RC structures.

This paper presents a time-dependent reliability analysis of corrosion affected RC structures. At first, the evaluation of the damages caused by reinforcement corrosion such as rebar mass loss, cracking in concrete cover and residual strength loss is presented analytically. Then, a stochastic deterioration model based on a gamma process is adopted to assess the structural reliability. The results from the numerical example show that the proposed approach is capable of evaluating the damage caused by reinforcement corrosion and the structural reliability of corroded RC structures.

2. Damages caused by reinforcement corrosion

The progress of corrosion directly affects the performance and hence the remaining service life of a corroding RC structure [7]. The quantitative description of these damages associated with performance deterioration of corrosion damaged RC structures is the first step in structural reliability analysis of these structures. Therefore, in this section quantitative assessment of the damages caused by reinforcement corrosion is discussed.

2.1. Corrosion induced cover cracking

The corrosion products formed during corrosion process are expansive in nature, which causes two to six times volume increase as compared with the original steel [10]. The increase of volume per unit length due to bar corrosion $\bullet V$ can be obtained from the volume of rust minus the volume of the original rebar of a diameter R_b consumed. This increment of volume per unit length of rebar creates a radial displacement at the rebar-concrete interface u_{bx} which can be estimated from

$$u_{bx} = \frac{\bullet V}{2\bullet R_b} = \frac{1}{2}\left(\bullet_{vol} \bullet 1\right) R_b X_p \tag{1}$$

where X_p is the corrosion level defined by the ratio of the mass loss of the corroded rebar to the original mass of the rebar and $\bullet_{vol}$ is the volume ratio of the corrosion product formed to its parent metal lies between 1.8 to 6.4 [10]. In this paper corrosion has been considered as uniform, therefore, reduction in rebar radius from the initial state R_b when uniform attack penetration x occurs can be evaluated from $R_{bx} = R_b - x$. The evolution of cracks in concrete cover is discussed in the analytical investigations by Chen and Alani [6], where the equivalent cracks width over the time was defined as the cumulated crack width over the cover surface. The intact cover concrete is treated as elastic material and the cracked concrete is considered as anisotropic in nature [6, 11]. From the anisotropic property and the bilinear softening law of the cracked concrete, normalized cumulative crack width over the concrete cover surface is obtained by considering boundary conditions and by ignoring the Poisson's effect associated with the hoop strain of the completely cracked concrete, expressed here as

$$W_{cx} = \frac{\frac{E}{f_t} u_{bx} \bullet a\left[R_b + R_c\left(l_o \bullet R_c\right)\left(l_o \bullet R_b\right).\bullet\left(R_c, R_b\right)\right]}{b\left(l_o \bullet R_b\right)\left[1\bullet R_c\left(l_o \bullet R_c\right).\bullet\left(R_c, R_b\right)\right]} \tag{2}$$

where l_o is the material constant given by $l_o = n_c l_{ch}/2\bullet b$ in which n_c is the number of cracks taken as 3 or 4 for the concrete around the rebar and l_{ch} is the characteristic length defined as $l_{ch}=EG_f\big/f_t^2$ by Hillerborg et al. [12] ; and $\bullet\left(R_c,R_b\right)$ is the crack factor associated with the material properties and radial distance r between rebar surface R_b and concrete cover surface R_c, defined as.

$$\bullet(R_c,R_b) = \frac{R_c \bullet R_b}{l_o(l_o \bullet R_c)(l_o \bullet R_b)} + \frac{1}{l_o^{\,2}}\ln\frac{R_c}{R_b}\frac{\left|l_o \bullet R_b\right|}{\left|l_o \bullet R_c\right|} \qquad (3)$$

2.2. Residual strength deterioration

The flexural strength deterioration due to reinforcement corrosion was investigated by Nepal and Chen [7], where the analytical method was proposed to evaluate the residual strength of RC beam with corroded reinforcement by considering different failure modes of concrete and steel. In case of un-corroded perfectly bonded RC beam the strain compatibility condition exists, as given by design codes. Therefore, the initial flexural resistance of RC beams can be evaluated by using design codes. For the corroded RC beam when ultimate bond strength is insufficient to prevent anchorage failure, the tensile force generated in the corroded tensile steel can be obtained from

$$f_{stx} = 2n_b \bullet R_{bx} l_d T_{ubx} \qquad (4)$$

where n_b is the number of the bottom tensile steel and l_d is the development length which can be evaluated from design code. T_{ubx} is the ultimate bond strength of corroded rebar and is obtained from Nepal and Chen [9]. The strain compatibility of a RC beam with corroded reinforcement can be considered between un-bonded and bonded condition [13]. Assuming the deformation of concrete is mainly due to plastic deformation occurring within the plastic equivalent region, new strain compatibility of the corroded beam can be expressed as

$$\frac{\bullet_{stx}}{\bullet_{ccx}} = g_x \frac{d_x \bullet Y_x}{Y_x}, \quad \frac{\bullet_{scx}}{\bullet_{ccx}} = g_x \frac{Y_x \bullet d'_x}{Y_x} \qquad (5)$$

where the plastic equivalent region is defined as $L_{eq} = 9.3Y_x$ [14]. Parameters in equation (5) are defined as: $\bullet_{ccx}$ is ultimate strain of concrete; $\bullet_{stx}$ and $\bullet_{scx}$ are strains of tensile steel and compression steel, respectively; Y_x is the neutral axis depth from the edge of compression zone; d_x is the effective depth of beam and d_x' is the distance from the centroid of the compression steel to edge of the compressive fibre corresponding to corrosion level X_p; and g_x is the interpolation factor which can be obtained by considering the bond strength value of perfectly bonded and un-bonded condition of the RC beam. By utilizing the concept given by Cairns and Zhao [15] that the corroded RC beam still follows the condition of equilibrium of resultant tensile and compressive forces acting at the beam section, the residual flexural strength can be evaluated by

$$M_{ux} = f_{ccx}\left(d_x \bullet 0.4Y_x\right) + f_{scx}\left(d_x \bullet d_x'\right) \qquad (6)$$

where f_{ccx} and f_{scx} are the compressive forces acting at the centroid of compression zone and the centroid of the compressive steel of the corroded beam, respectively.

2.3. Structural reliability analysis

The gamma process has been often adopted for structural deterioration modelling [6, 16, 17]. The gamma process is a stochastic process with independent non-negative increments having a gamma distribution with a given average of deterioration rate. Structural resistance degradation caused by reinforcement corrosion is a continuous and non-negative phenomenon [5, 7]. Therefore, the gamma process is suitable for the stochastic modelling of structural resistance deterioration in corrosion affected RC structures during their lifecycle. In this gamma process deterioration model, cumulative resistance deterioration J is considered as a random quantity with the gamma distribution, and has the shape parameter $\bullet_x > 0$ and scale parameter $\bullet > 0$. Then, the probability density function of this random quantity J, i.e. the structural resistance during the lifecycle at time t and corrosion level X_p $(X_p > 0)$, can be formulated as

$$f_{J_x}(J) = Ga\left(J, \bullet_x, \bullet\right) = \begin{cases} \dfrac{\bullet^{\bullet_x}}{\bullet \bullet_x} J^{\bullet_x \bullet 1} e^{\bullet \bullet J}, & for\ J \bullet 0 \\ 0\ , & elsewhere \end{cases} \tag{7}$$

where $\bullet(\bullet_x) = \int_0^{\bullet} v^{\bullet_x \bullet 1} e^{\bullet v} dv$ is the gamma function for shape parameter $\bullet_x > 0$. The scale parameter $\bullet$ could be estimated from statistical estimation methods such as a Maximum Likelihood Method by maximizing the logarithm of the likelihood function of the increment of the parameter [17] and the shape function $\bullet_x$ can be obtained from $\bullet_x = \bullet J_x$ in which J_x indicates the average deterioration rate associated with the reinforcement corrosion such as flexural strength deterioration in ultimate limit state. Assuming J_L as the maximum allowable limit of the structural deterioration, from the definition of probability of failure and by integrating probability density function given in equation (7), the structural reliability associated with structural resistance deterioration is given by

$$R(t) = 1 \bullet F(t) = 1 \bullet P_r\left[J_x \bullet J_L\right] = 1 \bullet \int_{J=J_L}^{\bullet} f_{J_x}(J) dJ = \frac{\bullet(\bullet_x) \bullet \bullet(\bullet_x, J_L \bullet)}{\bullet(\bullet_x)} \tag{8}$$

where $\bullet(\bullet, z) = \int_{v=z}^{\bullet} v^{\bullet \bullet 1} e^{\bullet v} dv$ is the incomplete gamma function for $z \bullet 0$ and $\bullet > 0$.

3. Numerical example

A simply supported RC beam of 5m span of a bridge exposed to an aggressive environment as defined by Eurocode 2 is now utilised to demonstrate the applicability of the proposed method for assessing the structural performance and a time-dependent reliability analysis during its service life. The beam is doubly reinforced with the cross-sectional width b =300 mm and effective depth d =560 mm Four steel rebar with diameter D_b =20 mm are provided as the tensile reinforcement and two rebar of diameter D_{sc} =16 mm are provided as the compressive steel with clear cover thickness C =40 mm along with the stirrup of diameter D_{st} =6 mm at spacing of 100 mm. The concrete has a characteristic compressive strength f_{ck} =40 MPa, the yield strength of original reinforcing steel f_{yk} = 460 MPa with modulus of elasticity E_{st} =200 GPa. The characteristic compressive strength of concrete is used for estimating other relevant properties of concrete i.e. tensile strength f_t=4.6 MPa; modulus of elasticity E_c=37 MPa [18, 19].The concrete fracture energy G_F=200 N/m is adopted and ultimate cohesive crack width w_u=1.48 mm and critical crack width w_{cr}=0.23 mm are estimated from CEB-FIP [19] for given compressive strength and assumed maximum aggregate size of 20 mm. The total number of crack n_c = 4 is adopted here. The concrete creep coefficient $\bullet_c$ = 2.0 and Poisson's ratio $\bullet$ = 0.18 and the volume ratio $\bullet_{vol}$ of the corrosion products is taken as 2.0 [10]. The RC beam has minimum service life of 60 years and is operated in aggressive environments with mean annual corrosion current per unit length i_{corr} =1 μA/cm^2.

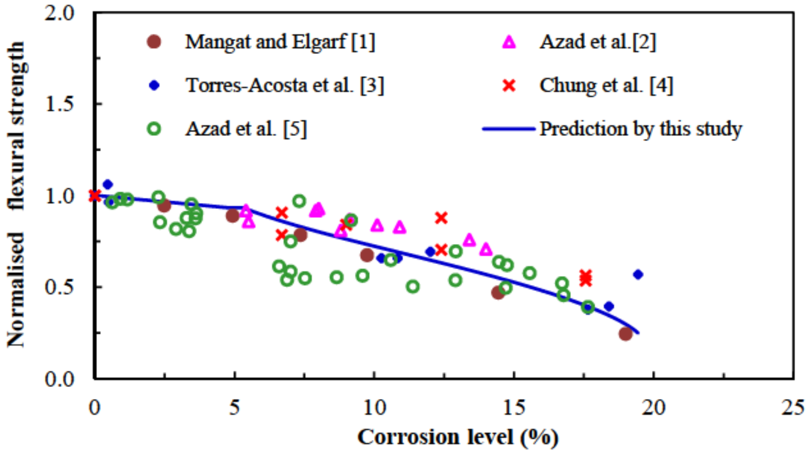

Figure 1. Analytical prediction of residual flexural strength versus corrosion level.

The results in figure 1 show the predicted results of residual flexural strength as a function of corrosion level. The predicted results are then compared with previous experimental investigations obtained from various sources. In figure 1, the residual load capacity is represented by the normalised flexural capacity, which is calculated by dividing the flexural capacity of corroded element by the capacity of the non-corroded element. It is found that the trend of predicted flexural strength deterioration match well with the published experimental data available from various sources. At the initial stage, the flexural strength of the corroded element deteriorates slowly almost in linear trend. When corrosion level reaches about 5%, considerable deterioration occurs due to the reduction in bond strength and corresponding anchorage failure, which occurs before yielding of the steel and the surrounding concrete.

Structural damage assessment and performance predictions using monitored data is critical to determine cost-effective infrastructure management strategies [20,21]. During the routine inspections of concrete bridges, cracking in concrete cover is the most important information recorded for condition rating. Based on the condition ratings collected during inspections, Bridge Management Systems (BMSs) are developed for optimum allocation of limited resources available [22]. Depending on the size of the cracks, these defects in concrete cover due to corrosion can be classified in different categories such as spalling; minor and major cracking etc. The defects in concrete and corresponding rebar loss are described in figure 2. Here the hair line crack is represented by crack ≤0.05 mm; minor cracking 0.05-0.1 mm; major cracking 0.1-0.4 mm and spalling 0.4-1.0 mm. When hair line crack appears at the concrete cover there is about 2% loss in rebar and as the crack size progress the reduction in rebar continuously increases, reaching approximately 13% when the spalling of the concrete cover takes place.

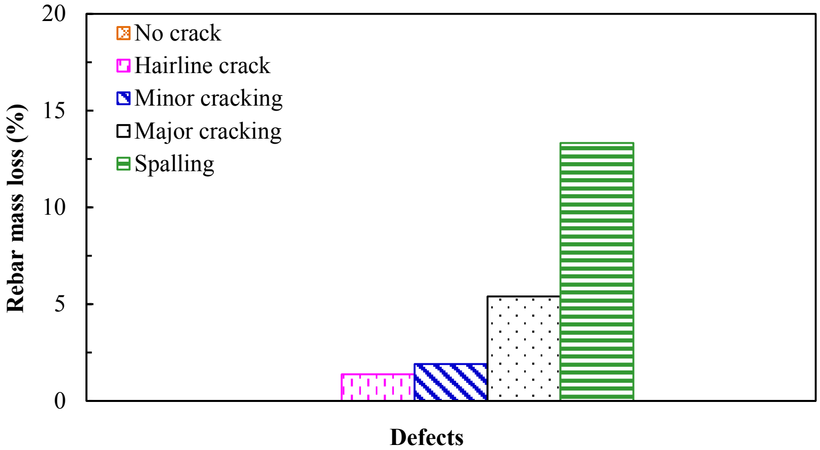

Figure 2. Reduction in rebar mass versus defects.

Influence of different types of aforementioned defects in concrete cover on the structural behaviour of corroded RC structure is presented in figure 3. From the results, till minor cracking in the concrete cover there is no significance change in residual flexural strength. As the defects reach to spalling stage, flexural strength decreases significantly. This clearly shows that, defects in concrete cover have significant effect on residual strength of corroded RC structures. In comparing these results of residual strength, the reduction in residual strength in unconfined concrete is relatively higher than in confined concrete. For instance when the defect in the concrete cover is spalling, the residual flexural strength of the confined element maintains about 60% of its initial strength, whereas in unconfined element it only maintains 40% of its initial strength. This is due to the absence of transverse reinforcement (stirrups) in unconfined concrete. Hence, the results from figure 3 and figure 4 show that, at the same stage of defects in the concrete cover, unconfined concrete is more susceptible than confined concrete.

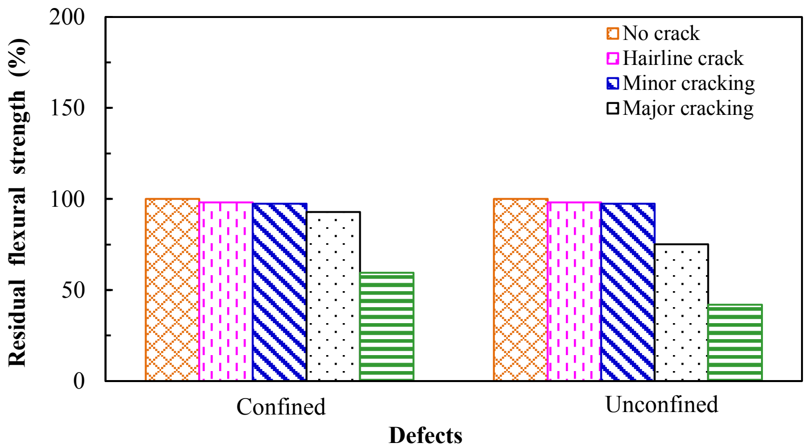

Figure 3. Normalised residual flexural strength versus defects for confined and unconfined concrete.

The structural reliability of confined and unconfined concrete in terms of flexural strength deterioration is given in figure 4. Here, different allowable flexural strength deterioration limits, i.e. J_L = 20%; 25% and 30%, respectively, have been considered during the analysis. Here again, at any stage of cover cracking structural reliability continuously decreases for both unconfined and confined concrete, showing higher probability of failure for a lower allowable deterioration limit. Furthermore from the time-dependent reliability analysis shown in figure 4, it is clear that the unconfined concrete has considerably lower structural reliability than the confined concrete when the same predefined allowable limit and concrete cover crack width are considered.

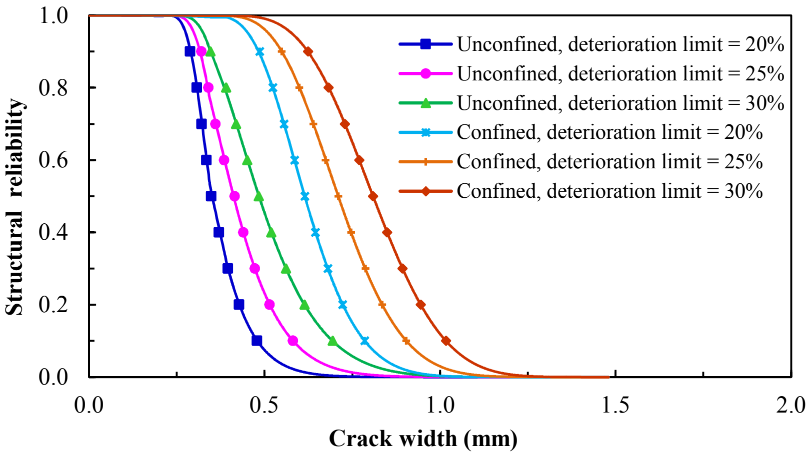

Figure 4. Structural reliability versus surface crack width for various allowable flexural strength deterioration limits of unconfined and confined concrete.

4. Conclusions

This paper presents a new approach for evaluating the damages caused by reinforcement corrosion together with its effect on structural reliability. On the basis of the results obtained from the numerical example, following conclusions are drawn: a) The proposed approach is capable of evaluating structural behaviour and defects of corrosion damaged RC structures; b) Flexural strength decreases significantly after 5% mass loss due to significant reduction in bond strength loss; c) Further progress of corrosion causes significant reduction in rebar size which in turn widens the crack in concrete cover, and consequently reduces residual strength of bond and flexural strength; d) The reliability of the corroded structure decreases with progress of defects in concrete. Further investigations are required to include the effect of external loading on the performance of corroded RC structure serving in aggressive environment.

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References

[1] Mangat P S and Elgarf M S 1999. Flexural strength of concrete beams with corroding reinforcement *ACI Structural Journal* **96** 149-158.
[2] Azad A K, Ahmad S and Azher S A 2007. Residual strength of corrosion-damaged reinforced concrete beams *ACI Materials Journal* **104** 40-47.
[3] Torres-Acosta A A, Navarro-Gutierreza S, and Teran-Guillen J 2007. Residual flexure capacity of corroded reinforced concrete beams *Engineering Structures* **29** 1145-1152.
[4] Chung L, Najm H and Balaguru P 2008. Flexural behavior of concrete slabs with corroded bars *Cement and Concrete Composites* **30** 184-193.
[5] Azad A K, Ahmad S and Al-Gohi B H A 2010. Flexural strength of corroded reinforced concrete beams *Magazine of Concrete Research* **62** 405-414.
[6] Chen H P and Alani A M 2013. Optimized maintenance strategy for concrete structures affected by cracking due to reinforcement corrosion *ACI Structural Journal* **110** 229-238.

[7] Nepal J and Chen H P. Evaluation of residual strength of corrosion damaged reinforced concrete structures *Fourth International Symposium on Life-Cycle Civil Engineering* 2014, Tokyo, Japan.

[8] Nepal J, Chen H P and Alani A M 2013. Analytical modelling of bond strength degradation due to reinforcement corrosion *Key Engineering Materials* **569** 1060-1067.

[9] Nepal J and Chen H P. Gamma process modelling for lifecycle performance assessment of corrosion affected concrete structures *World Congress on Advances in Civil, Environmental, and Materials Research* 2014, Busan, Korea.

[10] Papakonstantinou K G and Shinozuka M 2013. Probabilistic model for steel corrosion in reinforced concrete structures of large dimensions considering crack effects. Engineering Structures **57** 306-326.

[11] Chen H P and Xiao N 2012. Analytical solutions for corrosion-induced cohesive concrete cracking *Journal of Applied Mathematics* Article ID 769132.

[12] Hillerborg A, Modeer M and Petersson P E 1976. Analysis of crack formation and crack growth in concrete by means of fracture mechanics and finite elements *Cement and Concrete Research* **6** 773-781.

[13] Wang X H and Liu X L 2010. Simplified methodology for the evaluation of residual strength of corroded reinforced concrete beams *Journal of Performance of Constructed Facilities ASCE* **24** 108-119.

[14] Au F T K and Du J S 2004. Prediction of ultimate stress in un-bonded prestressed tendons *Magazine of Concrete Research* **56** 1-11.

[15] Cairns J and Zhao Z 1993. Behaviour of concrete beams with exposed reinforcement *Proceedings of the ICE - Structures and Buildings* **99** 141-154.

[16] Van Noortwijk J M and Frangopol D M 2004. Two probabilistic lifecycle maintenance models for deteriorating civil infrastructures *Probabilistic Engineering Mechanics* **19** 345-359.

[17] Van Noortwijk J M 2009. A survey of the application of gamma processes in maintenance *Reliability Engineering and System Safety* **94** 2-21.

[18] EC2 2004. *Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings (BS EN 1992-1-1:2004)*, European comitee for standardization, Brussels.

[19] Comité Euro-international du Béton - Fédération International de la Précontrainte (CEB-FIP) 1993. *CEB-FIP Model Code 1990 Design Code*, Thomas Telford, London.

[20] Chen H P and Bicanic N 2010. Identification of structural damage in buildings using iterative procedure and regularisation method *Engineering Computations* **27**(8) 930-950.

[21] Chen H P, Alani AM 2012. Reliability and optimised maintenance for sea defences *Proceedings of the ICE-Maritime Engineering* **165**(2) 51-64.

[22] Liu M and Frangopol D M 2005. Time-dependent bridge network reliability: Novel Approach *Journal of Structural Engineering ASCE* **131** 329-337.