Treatment of a stochastic service life prediction model to an evaluation of a distressed two-story RC building

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ABSTRACT: A service life prediction model is applied to a distressed two-storey building structure in Liberia for purposes of examining the model performance. The structure was estimated to be a 30 to 40 year old reinforced concrete (RC) structure due to be renovated for use as a residential and office building. Condition survey showed carbonation – induced reinforcement corrosion among structural members. The stochastic parameters used in the model analysis were taken from previous work (Ekolu, 2010). The stochastic model is environmentally sensitive and data from the condition survey was used to assess the model outputs. Basing on the model, the building's service life is estimated to have been in the range of 20 to 30 years. It is found that the model makes generally reasonable prediction of the current condition of the building. But this work also identifies some problems with the model that require research for performance improvement and imbuement of robustness for future use in service life estimation for concrete structures.

1 INTRODUCTION

This paper is an attempt to apply an existing service life prediction model on a reinforced concrete structure so as to evaluate the potential and accuracy attainable. While it is recognised that prediction of service life of a structure is essential for economic aspects associated with repair and maintenance, and risk analysis in certain structures especially the public infrastructure (Ekolu, 2010), accurate prediction of service life of any structure generally continues to be a complex subject of research. There are a number of models (RILEM, 1996; CIB, 2004) in the literature that have been suggested by various authors for different forms of applications, but hardly any of them may bear the acclaim of being universally acceptable or practically suited to various scenarios. Although a model can generate some results without incorporating variability into the analysis, the accuracy of the deterministic results can be easily distorted by variability of any of the parameters. For this reason, the concept of stochastic modelling becomes relevant and may even be necessary in service life prediction. These issues have been given careful consideration in determining the approach relevant to implementation of the model in the real life case of a dilapidated RC structure herein examined. The aims of this work are thus, to:

- Apply a stochastic method to a real case scenario for purposes of practical considerations
- Analyse results of the model output against field observations
- Examine model parameters towards sensitivity to outputs
- Assess the potential of the model for use in repair decisions
- Develop insights for future research in model development and application

2 BACKGROUND

The two storey RC building in Liberia, estimated to have been constructed in the 1970's, was architecturally designed to be a residential and office building. The owner needed to expand its facilities and intended to renovate the structure for use with its operations. Accordingly, engineering expertise was called upon to examine the condition of the structure and suggest the required repair and/or rehabilitation options. From the outset of the project, it was evident that severe spalling of the concrete cover had occurred leaving exposed steel reinforcement bars. Corroded steel bars were observed in the major structural members especially the slabs and beams. Severe cracking was also found in certain wall sections. With these alarming signs of degradation, it became necessary to conduct a detailed investigation in order to determine the existing capacity of the building. The engineering team then conducted the following assessments (Watch Tower, 2011):

- A condition survey entailing on-site inspection
- Structural analysis of the existing capacity

The aged building had no existing drawings and its history pertaining to different uses and change of ownerships was also not clear. Figure 1 gives the ground floor plan of the building showing, some sites of the inspection carried out. A total of seven (7) inspection sites were conducted within the building viz:- Veg Prep Rm 113 (site 1), Cooler Rm 114 (site 2), Commissary Rm 105 (site 3 and 4), Commissary Store Rm 103 (site 5), Lounge Rm 102 (site 6) and Home Overseer Rm 136 (site 7).

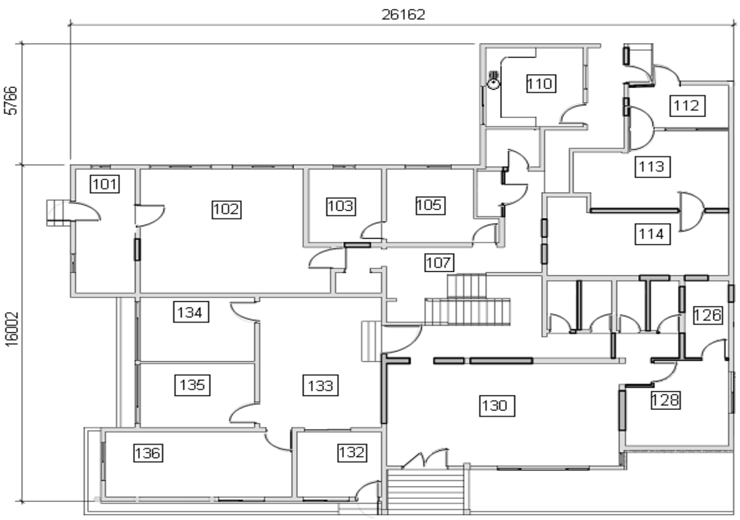

Figure 1 Floor plan of the two storey R.C building

3 CONDITION SURVEY

A condition survey on the building was done as outlined in most of the conventional literature on repair of structures, in order to identify the cause(s) and sources of the distress, determine the extent and severity of the distress, evaluate the appropriate remedial action(s), and prioritize repairs. The following survey actions were undertaken during the onsite inspection:

- Visual inspection was done specifically paying attention to the physical condition of the concrete. Observations were made on crack locations and sizes, spalling and delaminations in various structural members, surface staining and discolourations, exposed and corroded steel bars, honey combing (see Figures 2 and 3).
- Mechanical sounding was done with a two-pound hammer, which was used to strike the concrete surface while listening to the sound pitch. The simple test was conducted extensively over large areas of most concrete elements in order to detect delaminations. The surface was also scraped with a nail /screwdriver to see if the concrete was soft and powdery, or hard and resistant.
- Inspection of exposed rebars provided important information on the bar type and size (diameter), bar spacing, cover depth, and the severity of bar corrosion (see Figure 3).
- Non-destructive (NDT) tests for compressive strength was conducted by Rebound Hammer, in order to obtain the range of concrete strength(s) used in the structural members. No cores were taken for strength testing.
- Carbonation tests, conducted by phenolphthalein spray on freshly exposed concrete surfaces, gave indication of the carbonation depth in the various structural members. Establishment of the depth and extent of carbonation was considered to be essential, the mechanism being one likely cause of the observed re-bar corrosion attack.
- Dust samples were collected by drilling into the in-situ concrete. For each sample, a minimum of 25 grams of drilled dust were collected from selected sites locations. Other samples consisted of solid chunks of concrete broken off from structural members. The samples were supplied for laboratory analyses with interest in determining their chloride contents.
- A photographic record was also made, capturing the distress conditions and features observed in the structural components.

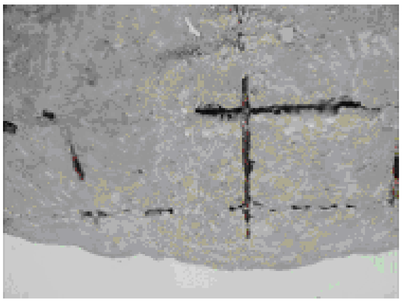

Figure 2 Slab soffits showing a thin concrete cover and corroded rebars

Figure 3 Severely corroded bar, completely debonded from concrete

The condition survey conducted as above discussed led to the following findings:

- Carbonation of the concrete was extensive with most of concrete tested showing a fully carbonated concrete cover thickness. This was observed in nearly all the major structural elements including slabs, external and internal beams, ring beams supported on masonry walls, columns. Phenolphthalein tests in several locations showed carbonation to have reached, and in some places, gone beyond the level of steel reinforcement. In sites 1 and 6, all cover concrete tested positively for carbonation except the immediate concrete behind the steel bar. During the tests, no carbonation depth measurements were taken. Since the tests had indicated that most or all the cover concrete of the structural members had carbonated, there was a clear loss of alkalinity responsible for protection of steel.

- The depth of cover was found to be generally small and highly variable from one site to another. Sites 1 and 2 had cover depths of 10 to 15 mm. The highest cover depth of 25 mm was found in site 5. In sites 6 and 7, certain sections of slabs and beams had a concrete cover thickness as low as 3 mm.
- Nearly all the reinforcement bars that were exposed for inspection were found to have corroded. Some of the severely corroded bars were completely de-bonded from the concrete with flakes of rust products coming off easily upon scraping with a knife. It was estimated that the severely corroded bars had lost as much as 30 to 50% of their cross-sectional areas.
- Water infiltration was specifically observed in certain areas. In site 6, the concrete slab was damp even after removal of the surface finishes consisting of 10 mm slurry, 20 mm sand, 70 mm screed, 15 mm sand, 25 mm tile screed and 25 mm terrazzo tiles. The sand on the surface of the 100 mm thick slab acted as a reservoir of moisture storage. It seemed that the balconies provided the source for water ingress into the floor slab concrete, with moisture getting trapped in the lower sand layer of the floor. The paint peeling off observed underneath the slab in site 7 was also attributed to the possible water infiltration in that section of the building.
- The compressive strengths, estimated using the Rebound Hammer NDT tests, showed results falling in the range of 18 to 23 MPa throughout the structure. The structure appears to have been constructed with concrete of the characteristic strength of 20 MPa.

4 LABORATORY TESTS

4.1 Chemical analysis

Dust samples drilled from the structural members and some chunks of concrete broken from the structure were analysed for chloride concentration. Interest was given to chlorides, the agents being the other possible cause of corrosion with or without carbonation. A total of six (6) samples were tested for water-soluble and acid-soluble chloride contents, basing on ASTM standards C 1218 and C 1152 respectively. The chloride contents were found to be in the range of 0.001% to 0.003% for water soluble chlorides and 0.005% to 0.009% for acid soluble chlorides, by mass of concrete. These results indicated that the chloride concentrations are very low, about 10 times below 0.05% which in practice is considered to be the threshold, beyond which chloride corrosion will occur. It was concluded that chloride attack was unlikely to be involved in the corrosion of steel reinforcement in this structure.

4.2 Optical microscopy

While it was evident from site inspection that the concrete had severely carbonated, information supplied from the site indicated that certain sections of the 30 to 40 year old building exhibited presence of colour changes between the outer concrete layer and interior concrete of structural members. Colour modification from the normal gray to brownish orange colour was evident from the surface to a certain depth of the concrete cover as seen in the fractured concrete shown in Figure 4. The observed colour change was quite puzzling as it is uncommon that such colour modifications could occur naturally due to carbonation. Concretes typically do not exhibit colour changes upon carbonation. It was of interest therefore to determine the mechanism associated with the colour change.

One piece of a concrete sample of about 60 x 60 x 35 mm length taken from structure, and containing both, the discoloured outer region and the normal gray interior concrete was used to undertake the phenolphthalein indicator test and petrographic examination by optical microscopy. Both tests confirmed that the naturally discoloured region was precisely the carbonation depth. The characteristic features for carbonation can be seen in the micrograph of Figure 5 which was typical of the discoloured concrete region. The dense paste structure is a result of infilling by discrete calcite crystals visibly notable as speckled texture, as opposed to an uncarbonated gray interior concrete, which was porous and generally free of calcite.

5 STRUCTURAL ASSESSMENT

The suspended 100 mm thick floor slab spanning 5.8 m in the short direction was evaluated having been observed to be in a structurally severe condition. The deflection of the slab was determined to have reached 35 mm. When all the finishes including layers of sand, screed, and terrazzo tiles were removed, the slab recovered only 5 mm. This failure to attain a significant elastic recovery was interpreted as being indicative of having approached or reached yield point. This suspicion was reinforced by the springy slab movements observed as one walked over it.

Figure 4. Field evidence of colour change (to orange) in the outer layer of concrete compared to the original gray colour seen in the interior concrete

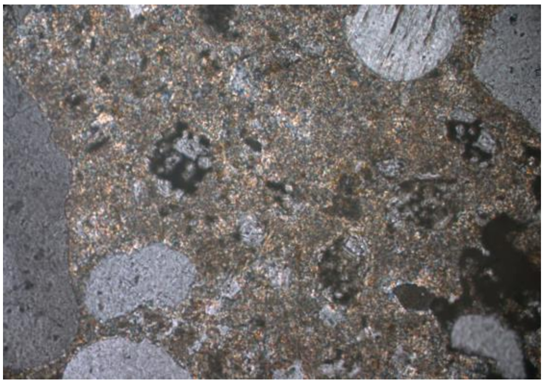

Figure 5. Discoloured region showing dense carbonated paste, associated with discrete calcium carbonate crystals

The slab, which was reinforced with R12 mild steel bars at a spacing of 300 mm, was assessed as a two-way slab subjected to a dead load of 3 kN/m^2 and a live load of 1.5 kN/m^2. From design calculations for the ultimate limit state (ULS), the slab should span no more than 4.1 m. The analysis showed that the as-built capacity of the slab was only one-half of the ULS design requirements. Similar conclusions were reached on serviceability design requirements.

6 STOCHASTIC MODEL ESTIMATION OF SERVICE LIFE

Using information determined from field survey and assessment of the R.C structure, the data acquired was treated to stochastic service life estimation. It identified carbonation as the primary mechanism responsible for the observed degradation of the building. Accordingly, a carbonation (only) prediction model was considered in the analysis, allowing service life estimation directly as follows (RILEM, 1996):

$$\mu(T_{sc}) = \frac{C^2}{C_{env} C_{air} a (f_{ck} + 8)^b} + \frac{80C}{Dr}$$

Where: $\mu(T_{SL})$ = mean service life (years), C= depth of concrete cover (mm), D= the diameter of the reinforcement bar, r = rate of corrosion of the reinforcement bars. t = time of exposure (years), C_{env} = environmental coefficient, C_{air} = air content coefficient, f_{ck} = characteristic cube compressive strength, f_{cu} = Mean cube compressive strength, a,b = constants dependant on binder type

The first component of the model estimates the time to de-passivation of the steel as a result of carbon-dioxide diffusion through the whole cover thickness. The second component of the model estimates the time required for propagation of corrosion to the level at which cracking of the concrete cover occurs.

The values for coefficients of variation used in the analyses were either taken from Ekolu, 2004 or adopted from the literature. The classifications and parameters undertaken for the model analysis are as follows:

1. All the seven (7) sites within the structure were classified as ‘sheltered’ from rain.
2. The structural concrete was assumed to be made of ordinary Portland cement (OPC) concrete, although no concrete analysis was conducted to determine the actual type of binder used in the concrete.
3. The mean compressive strengths measured on site were used in the model as opposed to using the characteristic strength value. The strengths from the various structural members generally varied from about 18 to 23 N/mm^2. No adjustments for carbonation effects on strength were considered.
4. The mean concrete cover thickness values determined from field measurements were used in the model.

In Table 1 is given a list of the model parameters and the associated coefficients of variation assumed for each parameter and used in service life prediction for the building structure.

Table 1 Model variables and their coefficients of variation

Cenv	Cair	D (mm)	fck (MPa)	a	b	r	Cover (mm)
1	1	12	Specific to site	1800	-1.7	1	Specific to site
0.5	0.4		0.2			0.5	0.2

By considering only the first component of the model, estimation of the time to carbonation of the cover thickness can be made, that is, without considering corrosion propagation. The time to depassivation of steel was taken as the limiting criteria defining the end of service life. Results of this analysis are shown in Figure 7 for the seven sites investigated in the RC building. It can be seen that carbonation of the cover thickness progressed quite rapidly in the early ages of the structure. This is consistent with the structurally low strength, low quality concrete. The model also predicts with 80% probability that the full cover thickness of the structural members carbonated within the first 20 years of the structure's life. The model results agree with field observations from onsite inspection and survey showing that the cover thickness of a majority of structural members had carbonated, reaching and in some cases going past the level of steel reinforcement (see Section 3). While the results of model prediction generally corroborate field observations, the absence of accurate information on the exact age of the building makes it difficult to precisely relate model predictions to the observed service life condition of the structure.

It is typical to allow about five years as propagation period from the time of depassivation of steel. Considering the observed severity of corrosion of steel bars showing associated loss of about 30 to 50% in steel area, it is possible or even likely that corrosion propagation had been in progress for quite a long time and a propagation period of about 10 years may appear reasonable. Combining the two components leads to service life estimation of 30 years for the building structure.

The full model with both components was also used to assess the influence of corrosion rate and bar size. In applying the service life prediction model, it was found that the second term was dominant and controlling, the two most sensitive variables being the corrosion rate and the diameter of the steel bar. Basing on experimental data, Tutti, 1982 suggested that the corrosion rate in carbonated concrete structures sheltered from rain can be evaluated at 90% relative humidity (RH) and proposed, r = 12 μm/year for 90%RH and r = 1 μm/year for 80%RH. The carbonation rates suggested for European cities range from 2.5 to 9 μm/year for concrete sheltered from rain (RILEM, 1996). Figure 8 shows the significant influence of change in corrosion rate on service life prediction for a fixed bar diameter of 12 mm. Clearly, the distribution curves are extremely sensitive to the changes in corrosion rate. For example, an increase of corrosion rate from 1 to 2 μm/year reduces services life estimation by more than one-half, while rates as high as 9 μm/year do not seem to give meaningful estimations. Similarly, the bar diameter have a marked influence on the estimations as shown in Figure 9. Increasing the bar size from 12 mm to 16 mm reduces service life estimation by about 10 years.

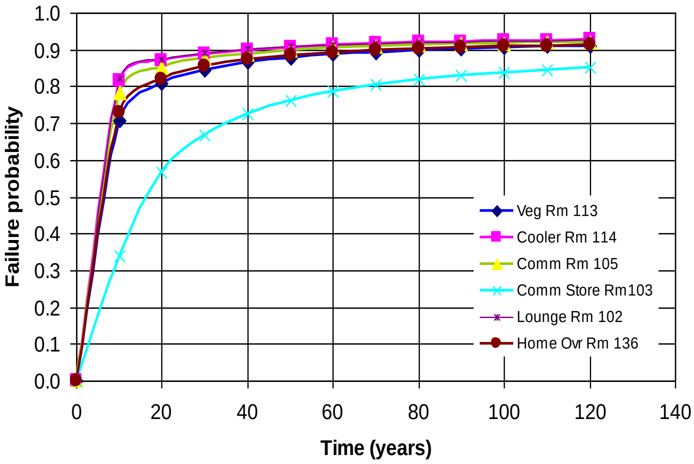

Figure 7 Probability distribution function for carbonation of the structural members

The high sensitivity of the service life predictions of the model to corrosion rate and bar sizes raises concerns since slight changes can greatly alter results. The problem is worse in the case of corrosion rates, which are generally difficult to establish for anyone environmental location. In African regions, there are no established or known corrosion rates that may be used in applications such as service life models. For the building structure examined in this paper, the question of the correct rate for use in the model analysis cannot be answered at this point and further research is needed. The corrosion rate of 1 μm/year appears to be too low while 9 μm/year is too high. Intermediate values between 2 to 4 μm/year appear to be appropriate. From the observed condition of the structure, the 10% failure probability has certainly been exceeded. If a 50% failure probability is assumed, the corrosion rate of 2 μm/year places the service life estimation of the structure within the range of 30 to 50 years. The survey of the structure was conducted in 2010 for a building estimated to have been constructed in the 1970's. Comparing the service life predictions and actual field observations, it can be surmised that the model outputs are meaningful.

7 CONCLUSIONS

A stochastic method was applied to estimation of service life of an old, distressed two storey RC building, employing environmentally sensitive carbonation model. The model predictions appear to place the current service life of the structure in the range of 30 years, while taking note of the broad range of assumptions made for parameters used in the model execution. These predictions also appear to be consistent with the suspected age of the structure. Issues arise concerning the efficacy of the model itself which has been found to be overly sensitive to changes in corrosion rate and bar diameters. These and other aspects determined from this work highlight the future research needs for improvement of the model.

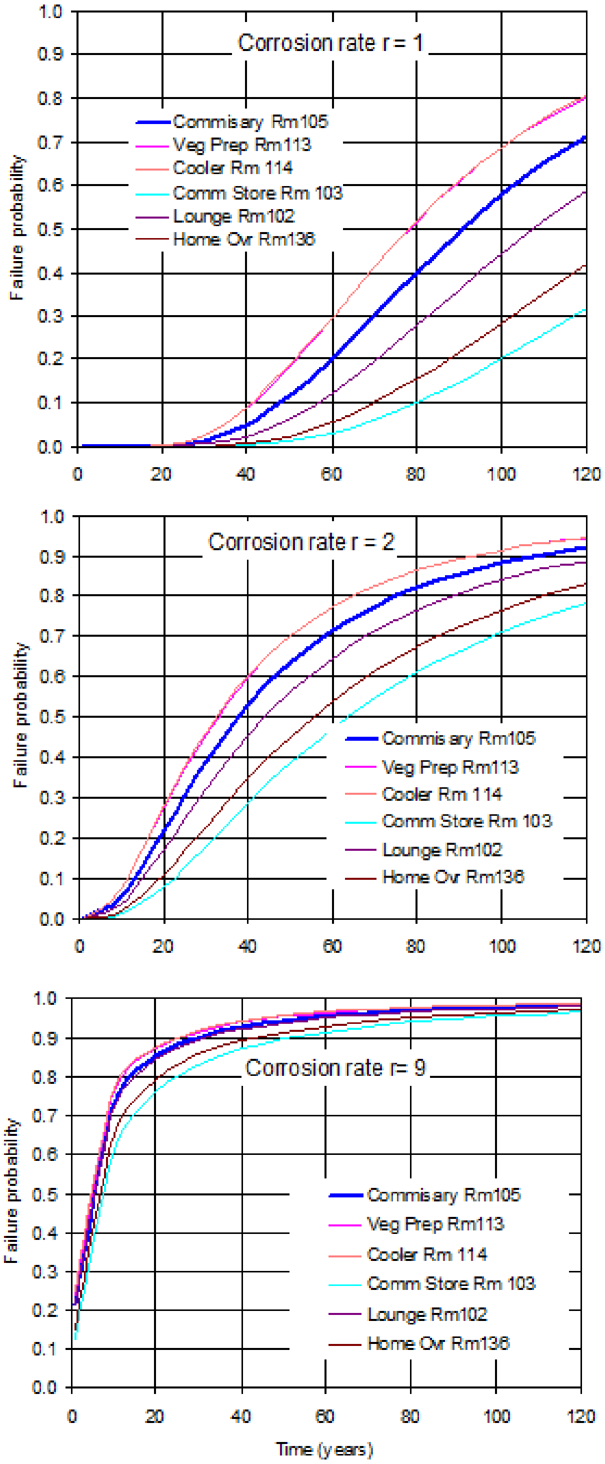

Figure 8 Probability distribution functions for varied corrosion rates and fixed bar diameter of 12 mm

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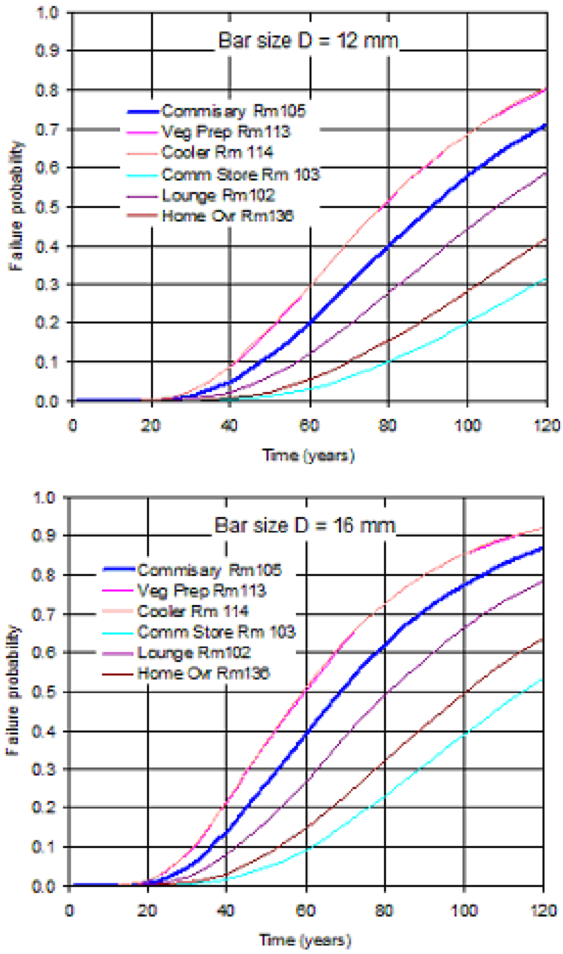

Figure 9 Probability distribution functions for a fixed corrosion rate r = 1 and varied bar diameters

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