STRENGTH REDUCTION OF REINFORCED CONCRETE COLUMNS SUBJECT TO CORROSION RELATED COVER SPALLING

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ABSTRACT

The performance of deteriorated reinforced concrete columns subjected to eccentric loads had been investigated in this research considering the effects of the concrete cover spalling location and the amount of spalling on the residual strength of rectangular and circular sections by developing axial load-moment interaction diagrams. The influence of several factors associated with material properties and amount of reinforcement on the improvement of the residual strength of such columns were investigated. Based on the theoretical analysis, it was found that the most critical case is where the concrete cover spalling occurs on the compression side of the column; therefore, this case was selected to carry out an experimental work to verify the accuracy of the developed model. The experimental failure load was found to be more than the calculated theoretical load with an acceptable tolerance which allows for safety if the equivalent rectangular stress block is used, while using the actual parabolic stress strain curve for concrete was found to be unsafe for some cases. The increase of the reinforcement area leads to an enhancement of the performance of the deteriorated columns by various proportions depending on the case and the loading conditions. Also, it was found that using a high grade of steel (72 ksi) significantly improves the strength of columns loaded with large eccentricities, while using concrete with higher compressive strength increases the capacity of the columns loaded with small eccentricities with e/h up to 0.4. For concrete stress-strain relationship, it was found that using the actual parabolic curve overestimate the strength of deteriorated
columns especially if the section is a compression-controlled section for both rectangular and circular sections. The comparison of the present study with previous theoretical and experimental studies reveals the applicability of the model for the strength evaluation of the deteriorated columns.

A numerical analysis was conducted by using ABAQUS-3D model to simulate the deteriorated columns and it was found that the finite element analysis results are in good agreement with the experimental results and it was possible to simulate the column behavior using FE models. Also, design equations were developed from a data analysis using STATISTICA to express the ultimate failure load for the deteriorated columns for all the studied columns, and it was found that more than 97% of the predicted values had excellent correlation with the observed values.
DEDICATION

This is a special dedication to my beloved father, Mr. Nizar Abdulhameed Khalid and my mother Ms. Balqees Rasheed Karo.
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CHAPTER I

INTRODUCTION

1.1 General

Deterioration of reinforced concrete structures resulted from mistakes in design or construction, climate conditions, chemical attack or damage due to impact, seismic and blast loads.

Corrosion of reinforcement is considered one of the major durability problems in reinforced concrete structures in cold weather regions. For highway bridges and other reinforced concrete structures, the corrosion of reinforcing steel in concrete has been a major problem with serious economic and safety implications. Predominantly, with the extensive use of de-icing salt, bridge decks and bridge piers are vulnerable to corrosion of steel reinforcement through the break-down of the passive layer around the reinforcement. When the expansive forces resulted from the accumulation of the corrosion products around the reinforcement exceeds the tensile strength of concrete, cracks in concrete cover initiate. The propagation of the corrosion causes the concrete cracks to be wider and leads eventually to the spalling of concrete cover.

Corrosion in steel bars affects a structure’s safety, which depends on the surrounding environmental conditions that mainly affect the corrosion rate, the location of the member in the building, and the type of the member (Mohamed El-Reedy, 2008).
Freezing and thawing beside corrosion in cold weather regions causes cracking of concrete cover. The damage occurs where the parts of structures are in direct contact with water or snow and when the surface remains wet for a long period of time. When water freezes to ice it occupies 9% more volume than that of water, so the concrete is considered as critically saturated if 91% of concrete pores are filled with water which develops internal cracks around coarse aggregate particles. The deterioration of concrete due to freezing and thawing is gradual since the formation of the internal cracks is a progressive phenomenon.

External columns in building and pier columns are considered vulnerable to change in environmental factors, and although, corrosion is considered a serviceability problem, the strength of columns should be investigated after their exposure to corrosion to determine the loss of strength due to concrete cover spalling. The designer needs to determine whether a column is still adequate to stand the applied loads or not, and to avoid unnecessary and expensive cost of rehabilitation and repair resulting from an over-conservative load rating of the deteriorated structures.

1.2 Objective

Regardless of the reasons that lead to concrete cover cracking of the deteriorated reinforced concrete columns, the load carrying capacity of such columns need to be investigated. Most studies related to the deteriorated columns were presented through experimental tests on the residual strength of corroded specimens. The theoretical study presented by Tapan and Aboutaha, 2008 evaluated the load carrying capacity of the deteriorated corroded columns through the calculation of the hoop stresses in the concrete cover after accounting the corrosion products, and then the cracked concrete section is determined.
The objective of this study is to evaluate the load carrying capacity of the deteriorated columns considering the concrete cover spalling in different locations of the column section taking into account the effect of the amount of the concrete cover spalling and the location of it on the residual strength of columns to determine the critical case which needs more attention in repairing of the deteriorated columns in addition to a parametric study to investigate the effect of several factors on the residual strength of the columns.

1.3 Research Methodology

An evaluation of the load carrying capacity of the deteriorated columns is presented in this study. Sets of axial load-moment interaction diagrams were developed assuming six cases of concrete cover spalling according to the location of the deteriorated column side. The amount of concrete spalling and different reinforcement ratio were considered in interaction diagram construction. Experimental work for columns with concrete cover spalling in the compression face was carried out to verify the theoretically developed axial load- moment interaction diagrams. Several specimens were subjected to accelerated corrosion and tested under compression loading.

A numerical analysis was performed by developing 3D finite element model in ABAQUS for the specimens tested in the laboratory to verify the theoretical and experimental results and to understand the performance and behavior of the deteriorated columns. Finally, a data analysis was made by using STATISTICA to develop an equation to predicate the ultimate failure load for several cases of deterioration.
CHAPTER II

LITERATURE REVIEW

2.1 Introduction

Reinforced concrete structures are influenced by the environmental conditions they are exposed to and that can lead to deterioration of these structures. For corrosion it is noted that most researchers focus on how to repair the corroded members by using different wrapping techniques or by using non-corroding reinforcing bars which can be more expensive in comparison with steel. It is important to investigate the residual strength of the deteriorated reinforced concrete members due to corrosion in order to decide how to repair them to reduce the repair cost.

In this chapter, the studies related to the effect of corrosion on the two components - concrete and steel- in reinforced concrete columns will be presented.

2.2 Corrosion of Reinforcing Steel Bars

2.2.1 Introduction

Because of highly alkaline environment provided by the concrete for the embedded steel, the steel is well protected against corrosion. In addition, the high quality concrete with low water/cement ratio which leads to low permeability decreases the penetration of factors inducing corrosion. However, corrosion can be an existing problem as a result of
inferior concrete quality or if the design doesn’t consider the environmental conditions adequately, sub-standard construction or because of the harsh environmental conditions.

2.2.2 Process and Mechanism of Corrosion

Corrosion is an electrochemical process involving the flow of charges (electrons and ions). At active sites on the bar, called anodes, iron atoms lose electrons and move into the surrounding concrete as ferrous ions (PCA, 2017). When corrosion of reinforcement occurs, the steel dissolves in the voids that contain water and gives up electrons; this is called anodic reaction, given as:

\[
Fe \rightarrow Fe^{2+} + 2e^- \quad (2.1)
\]

To preserve electrical neutrality, the electrons must be consumed elsewhere on the steel surface. If the electron will be accumulated on the other part of the steel reinforcement but cannot accumulate with huge numbers in the same location, there is another reaction that consumes electrons, which is known as the cathodic reaction (Broomfield, 2007; El-Reedy, 2008).

\[
\frac{1}{2} O_2 + H_2O + 2e^- \rightarrow 2OH^- \quad (2.2)
\]

The hydroxyl ions increase the local alkalinity and therefore strengthen the passive layer. To form rust, \((OH^-)\) must react with ferrous iron \((Fe^{2+})\) as a result of chemical equation (2.1) through a conductor (electrolyte). This reaction will produce ferrous hydroxide, which will react with oxygen and water again and produce ferric hydroxide (Broomfield, 2003). This chemical reaction is shown in Figure 2.1.

\[
Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2 \quad (2.3)
\]

\[
4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3 \quad (2.4)
\]

\[
2Fe(OH)_3 \rightarrow Fe_2O_3 + 3H_2O \quad (2.5)
\]
Ferrous hydroxide will increase the volume of the original steel bars by about two times or more (Fig.2.2) (Liu and Weyers, 1998). When iron turns to ferric hydroxide in the presence of water, its volume will increase more to reach about 10 times its original volume which produces very high expansive pressure that induces tensile stresses in the surrounding concrete. In this stage, cracks in concrete start until the concrete cover falls; rust, with its brown color, can clearly be seen on the steel bar (El-Reedy, 2008).
2.2.2.1 Carbonation Induced Corrosion

Carbon dioxide from the atmosphere reacts with the calcium hydroxide (and other hydroxides) in the cement paste and effectively neutralizing the pore solution.

Carbonation is detected as a reduction in the pH of the pore solution in the surface regions of the concrete and appears as a fairly sharp front, parallel to the surface.

\[
\text{CO}_2 + \text{H}_2\text{O} \rightarrow \text{H}_2\text{CO}_3 \quad (2.6)
\]

\[
\text{H}_2\text{CO}_3 + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 + 2\text{H}_2\text{O} \quad (2.7)
\]

The depth of carbonation increases with time and the rate of relative humidity (RH), the penetration of the CO\(_2\) into the concrete is highest at low RH but the reaction with the Ca(OH)\(_2\) takes place in solution and is, therefore, highest in saturated concrete. As a result of the combination of these two factors, that carbonation is most rapid in the 50% - 70% RH range, Figure 2.3 (Tuutti,1982,1980). (Rosenberg,1989) mentioned that although an intermediate RH provides the highest rate of carbonation, active corrosion of any significance does not occur in that humidity range (Hansson, 2012). Consequently, the most aggressive environment for carbonation-induced corrosion is alternate semi-dry and wet cycles (Tuutti, 1980). So, carbonation can be a major factor in the durability of concrete in regions subjected to periods of rainstorm followed by high temperatures.

When carbonation takes place, the pH of the hydrated cement paste surrounding the steel will decrease. As the pH of the pore solution drops below 9, the passive film protecting the steel bar becomes unstable and active corrosion initiates. The corrosion resulting from carbonation is generalized and relatively homogeneous and corrosion products don’t affect the concrete cover cracking because they tend to be more soluble in the neutral carbonated concrete and appear as a rust stain on the concrete.
2.2.2.2 Chloride Induced Corrosion

The capillary suction of chloride containing water such as seawater, water with dissolved deicing salts, is considered the fastest way for chloride to attack concrete (Bohni, 2005). Concrete can be attacked by chloride either from inside during mixing by using seawater, or additives containing calcium chloride to accelerate setting time, or it can be external effect through the seawater spray or salt used to melt ice (Al-Reedy, 2008). Unlike the carbonation, the corrosion resulting from chloride is nonuniform; this is because of the difference of the chloride ions concentration which is high in some areas around reinforcement leads to be active and other areas remain passive. The active region will produce electrons, anode, while the passive region will consume the electrons, cathode. The reaction can be expressed by the equation:

\[
\begin{align*}
\text{Fe} + 2\text{Cl}^- & \rightarrow \text{FeCl}_2 + 2e^- \\
\text{FeCl}_2 + 2\text{H}_2\text{O} & \rightarrow \text{Fe(OH)}_2 + 2\text{H}^+ + 2\text{Cl}^- 
\end{align*}
\]
AHLSTRÖM (2015), presented an experimental work to study the effect of the relative humidity on the corrosion of the steel bars at various chloride levels. In this study, it was found that for RHs from 75% to 100% showed that the corrosion rate was moisture dependent with a maximum in the interval 91% to 97% depending on the chloride concentration. For chloride concentration (2%) the maximum was at the RH (91%-94%), while at lower chloride concentrations (0.1% to 1.0%) the maximum was at RH (97%). Also, the exposure to cyclic RH (75%-100%) the corrosion rate at 0.6% chloride was considerably higher than the maximum value obtained at 97% RH static conditions. (Miyazato and Otsuki, 2010) studied the corrosion induced by both carbonation and chloride in mortar with bending cracks, they found that the w/c ratio affects the rate of corrosion for carbonation induced corrosion in which decreases by reducing the w/c ratio, while for chloride attack decreasing of w/c ratio doesn’t guarantee a decrease in corrosion rate, besides the corrosion rate induced by chloride is higher than that by carbonation.

2.2.3 Forms of Corrosion

The basis for the classification of the corrosion type is the appearance of the corroded metal. The awareness of the corrosion type and the environment that induced corrosion helps to find the best way to prevent it and how to repair the element if it occurred. The common forms of corrosion for reinforced concrete structures are discussed briefly in the following sections.

2.2.3.1 Uniform Attack Corrosion

Uniform corrosion (General) is characterized by a chemical or electrochemical reaction which proceeds uniformly over the entire exposed surface or over a large area,
usually described by the depth of attack over time, in terms of loss of metal thickness or as a rate of penetration over time. Uniform iron dissolution over the whole surface is related to microcell where anodic and cathodic reactions are immediately adjacent, leading to uniform corrosion which is generally caused by carbonation of the concrete or by very high chloride content at the rebar forming rust on the surface. Uniform corrosion is considered as a safe form of corrosion, due to the fact that it is predictable, manageable and often preventable.

2.2.3.2 Pitting Corrosion

This type of corrosion occurs under certain conditions, which leads to accelerated corrosion in certain (localized) areas rather than uniform corrosion throughout the element. Such conditions include low concentrations of oxygen or high concentrations of chlorides. Pitting corrosion is related to the formation of macrocells, where a net distinction between corroding areas of the rebar (anode) and non-corroding, passive surfaces (cathode) is found. In the worst cases, most of the surface remains protected, but tiny fluctuations degrade the film in a few critical areas. Corrosion at these points is amplified and can cause pits. Generally, a pit may be described as a cavity or hole with the surface diameter about the same as or less than the depth (Raupach, 2007).

2.2.4 Effect of Corrosion on Mechanical Properties of Steel Bars

2.2.4.1 Tensile Strength

As the tensile strength of reinforcing bars affects the flexural strength of reinforced concrete structures, the tensile strength of corroded steel bars in tension zone needs to be investigated for columns subjected to eccentric loads.
For steel bars exposed to atmosphere for 16 months, the effect of rusting on the yield and ultimate strength is insignificant, while there is a loss in weight increases with the period of exposure with maximum reduction at the end of the exposure (0.033lb/ft^2) with the corresponding reduction in diameter of (0.53%) (Masleuhuddin, et al., 1990). Their conclusion about the tensile strength is similar to that of (Uomoto et al., 1984) who obtained that the ultimate strength of the corroded bars was in the range of (90-95%) of the non-corroded bars.

(Almusallam, 2001) studied the effect of the degree of corrosion on the mechanical properties of the reinforcing bars. He determined that the tensile strength calculated using the nominal area was less than that calculated using the actual area. As the degree of corrosion increased up to 75%, the difference between the two values increased up to 75% which also a fall below the (ASTM 615 Specifications) of 600 MPA for a degree of corrosion more than (11.5%).

A study was carried out by (Cairns et al., 2005) to investigate the mechanical properties of the corroded steel bars using numerical and experimental models. The numerical model adopted in the study was constructed through a spreadsheet; the bar was divided into short incremental lengths, the cross sectional area along the bar was measured. Then the elongation of each increment of bar length and hence the strains were calculated for each load increment. Because of the lack of information on corrosion topography, the model is not suitable for practical use and the following relationships were used:

\[
\begin{align*}
    f_y &= (1.0 - \alpha_y Q_{corr})f_{y0} \\
    f_u &= (1.0 - \alpha_u Q_{corr})f_{u0}
\end{align*}
\]  

(2.10)  

(2.11)
where, $f_y, f_u$: yield and ultimate tensile strength after time $t$ based on the original cross section, respectively.

$f_{y0}, f_{u0}$: yield and ultimate tensile strength of the non-corroded bar, respectively.

$Q_{corr}$: average section loss as a percentage of original cross section

$\alpha_y, \alpha_u$: empirical coefficient used as 0.012 and 0.011

It was indicated that for plain mild steel bars subjected to low corrosion rate up to (7%), the reduction in the yield strength was not greater than the reduction in the cross-section, while for heavily corroded bars, the reduction in the yield strength was less than the maximum reduction in the cross-section.

2.2.4.2 Ductility

(Almusallam, 2001) carried out an experimental work by casting two main groups of concrete specimens, one of them reinforced with steel rebar of diameter (6mm) and the other reinforced with steel rebar of diameter (12mm), each group was subjected to a different degree of accelerated corrosion ranging from (0.88-75%). The stress-strain curves for (6mm) bars with 0.88% and 13.9% were constructed as shown in (Figure 2.4), from these curves it was obtained that the elongation of bars with 0.88% corrosion is more than that with (13.9%) corrosion. However, the test results indicated a brittle behavior for bars with (12.6%) or more degree of corrosion.

(Cairns et al., 2005) carried out tensile tests on steel rebar, the original diameter of the reference bar is (12mm) and other bars with various degrees of damage machined using an end mill of 8mm radius. Figure 2.5 shows the load-displacement relationship obtained for these samples for. It can be observed that the ductility is significantly affected by the reduction in area after the yield limit.
Steel bars specimens were corroded using accelerated corrosion tests in salt spray environment, the stress-strain curves were determined for the non-corroded specimens and that exposed to corrosion after 10, 45 and 90 days by (Apostolopoulos, 2006). From Figure 2.6, it can be observed that the exposure to corrosion caused a significant reduction in ductility, the elongation for the non-corroded specimens was 16.91% while for that exposed to corrosion for 90 days was 11.79%. Also, the strain energy density which was calculated as the area under the stress-strain curves with the increase of the exposure time, the decrease in strain values is reduced by (16%, 33.5%) as the time of accelerated corrosion increases from (45 to 90) days. The decrease in strain energy density reduces the service life since it reduces the number of cycles to failure under low cycle fatigue as studied by (Apostolopoulos, 2006), Figure 2.7.
Figure 2.5: Plots of Load Versus Elongation from Tensile Tests on 12 mm-Diameter Bars With 8 mm-Radius Machined Defects of Various Depths (Cairns et al., 2005)

Figure 2.6: Typical Stress-Strain Curves for Non Corroded and Corroded Steel Bars (Apostolopoulos, 2006)

Figure 2.7: Effect of Corrosion on the Number of Cycles to Failure for All Applied Stress Levels (Apostolopoulos, 2006)
2.2.4.3 Buckling Length of Reinforcing Bars

Kashani, et al. (2013) carried out uniaxial compression testing on (57) corroded bars with different effective lengths. They have chosen five ratios of (L/D) which were (5,8,10,15, and 20). The specimens were exposed to pitting corrosion along the length of the bars resulting in a reduction in cross-section area. The results obtained from these tests showed that bars with (L/D=5) can be treated as tension bars and the effect of buckling can be ignored. For bars with (L/D=8) and (L/D=10), three types of behavior were observed depending on the amount of mass loss and uniformity of corrosion. For bars with highly pitting corrosion showed a smooth transition from linear elastic to nonlinear plastic which was longer compared to uncorroded bars. The post-buckling softening of these bars showed a similar trend to the uncorroded bars, but with a significant reduction in buckling stress. For bars with more uniformly distributed corrosion and small mass loss showed a similar behavior of the uncorroded bars with a small reduction in buckling stress. The last type of behavior was indicated for the bars with relatively uniform corrosion and high mass loss showed a faster transition from linear elastic to nonlinear plastic and a relatively high decrease in buckling stress. For bars with (L/D=15) and (L/D=20), it was observed that a quick transition from linear elastic to post-yield softening compared to shorter bars. Also, these bars showed a very sharp transition from linear elastic at the point of buckling to a very steep post-yield softening branch.

The inelastic buckling of steel bars was studied by other researchers, they indicated that the ultimate load capacity decreased while the (L/D) increased more than (5) and the load-deflection path could be unstable after the yielding load, (Mau,1990, Monti and Nuti,1992).
Bae et al. (2005) presented an experimental work and simple analytical model to investigate the inelastic buckling of 162 reinforcing bars tested under monotonic compression loads for this purpose. The experimental program consisted of tests of Grade 60 No. 8 and No. 10 steel reinforcing bars. The behavior of reinforcing bars in compression depended on their unsupported length-to-bar diameter ratio \((L/d)\), as well as their initial imperfections at midspan with respect to the bar size \((e/d)\). The two ends of a bar were assumed to be restrained against rotation and the bar diameter refers to the nominal bar diameter. A range of nine unsupported length-to-bar diameter ratios \((L/d=4, 5, 6, 7, 8, 9, 10, 11, \text{and} 12)\) and six initial eccentricities to-bar diameter ratios \((e/d=0.0, 0.1, 0.2, 0.3, 0.4, \text{and} 0.5)\) were used for each bar size. For No. 8 reinforcing bars, it was observed that load-carrying capacity and ductility decreased as the \(L/d\) ratio increased, while for No. 8 reinforcing bars, it was observed that load-carrying capacity and ductility were somewhat lower than that for No. 8. Depending on the experimental results, it was concluded that for a given \((L/d)\) ratio, an increase in the initial eccentricity \((e/d)\) ratio resulted in a decrease in the load-carrying capacity and ductility, and in the same manner for a given \((e/d)\) ratio, an increase in the slenderness \((L/d)\) ratio resulted in a decrease in the load-carrying capacity and ductility. For a given \((e/d)\) ratio and bar size, the maximum axial load occurred approximately at the same lateral deformation for all test specimens.

In general, it was found that for reinforcing bars of \(L/D\) less than or equal 6) the load-carrying capacity was maintained while specimens experienced large inelastic deformations. The researchers suggested avoiding the use of reinforcing bars of \(L/D\) greater than 6) since there was an instability problem once the maximum load was reached.
2.3 Deterioration of Concrete Cover

2.3.1 Loss of Concrete Cover Due to Freezing and Thawing

In cold climate region, the freezing and thawing is considered a major problem causes deterioration of the concrete. Cycles of freezing and thawing cause cracks in concrete running parallel to joints or corners of concrete surfaces because they are exposed to more standing water and then absorb more water. If the pores are critically saturated, water will begin to flow to make room for the increased ice volume. The concrete will rupture if the hydraulic pressure exceeds its tensile strength. For the rest of the concrete area, fine cracks scattered in a pattern along the concrete surface may also occur during freezing periods due to shrinkage of Portland cement paste. Further, the de-icing salt together with repeated freezing and thawing may cause failure of the concrete cover by surface scaling and loss of cross-sectional area which combined with steel corrosion may critically reduce the structure’s service life. In addition, the bond strength of an ordinary concrete reduced by 85.6% after 100 cycles of freezing and thawing (Behfarnia, 2010).

2.3.2 Loss of Concrete Cover Due to Corrosion

Corrosion of the reinforcing bars is indicated by the staining of the surrounding concrete. When the oxidation reaction takes place at the anode, it produces the product known as rust. Rust is brown or dark colored and is visible on the exterior surface of the concrete where cracks have formed because of the accumulation of the corrosion products on the bar surface and the rust has leached out through the crack, often by means of water penetration into the crack. As the pressure resulting from the corrosion products increases over time, the crack width increases and eventually leads to spalling of cover.
2.3.3 Studies Related to Cover Cracking

In this section, the experimental work presented by some of researchers to study the effect of corrosion on the concrete cover is reviewed.

Andrade et.al (1993), carried out an experiential work to study the time elapsed between steel depassivation and concrete cracking and to check the ability of the numerical model to make an accurate prediction. Four concrete specimens of dimensions (15x15x38cm) were cast and one reinforcing rebar was embedded in the specimens. The rebar was placed at the corner for the first specimen while for the others the rebar was placed in the center of one of the specimen sides. The specimens were corroded by applying current and adding 3% of CaCl$_2$ by weight of cement. The results indicated that the amount of bar radius decrease necessary to induce cracking was very low (about 20µm in the condition of the test with the assumption that 100% current efficiency must be taken into account, the visible crack widths corresponding to this value were (0.05-0.1mm). the researchers estimated the expected time periods needed to cause cracking depends on the corrosion rate, if the corrosion rate/year is (10µm), (2 and 10-15) years is needed to induce crack width (0.05-0.1mm) and (0.2-0.3mm), respectively, while if the corrosion rate/year is (100µm), (0.2 and 1-2) years is needed to induce crack width (0.05-0.1mm) and (0.2-0.3mm), respectively. Also, a general formula was proposed to calculate the propagation period as part of service life as:

$$ t_p = \text{Decrease in Bar Radius/Corrosion Rate} $$

The state of stress in the concrete due to bond forces from deformed reinforcing bars is analyzed by (Tepfers, 1979). The stresses are calculated for an elastic stage, a plastic stage, and an elastic stage with internal ring cracks. Depending on the concrete cover to
bar diameter \((c/D)\), Tepfers found that the ultimate load may be higher than that load causes cover cracking since the concrete has some plastic deformation. The appearance of the concrete cover cracks along the deformed bars in the bond zones for normal cover thickness can be determined by using the mean of the values obtained for the two stages (partly cracked case and plastic case), given by:

\[
P_{cr} = 0.3(0.5 \frac{4.33c}{D})f_{ct}
\]  

(2.12)

where: \(c\) is the concrete cover to the bar, mm, \(f_{ct}\) is a concrete tensile strength, MPa, \(D\) is the bar diameter, mm, \(P_{cr}\) is the pressure to induce cover cracking, MPa

Liu and Weyers, (1998) carried out an experimental work on simulated bridge deck slabs considering different factors, corrosion rate by adding different amount of chloride (0.6, 1.2, 2.4, 4.8, 9.6 and 12 lb/yd³), concrete cover depth (2 and 3in), reinforcing steel bar spacing (6 and 8in), and size (No.5 and No.6). It was observed that total of 10 slab specimens of the 44 slabs cracked within the 5-year experimental testing program, 3 specimens were with admixed chloride content (9.6lb/yd³), 2 in cover depth, No.5 reinforcement cracked after 1.84 year, the other 3 specimens were same as the previous set but with 3 in cover depth cracked after 3.54 year and the last 4 specimens were with admixed chloride content (12lb/yd³), 1 in cover depth, No.5 reinforcement cracked after 0.72 year. Also, a theoretical formula was proposed to calculate the time to cracking which was indicated as a function of corrosion rate and the critical weight of rust products which is dependent on the cover depth, properties of the concrete, type of corrosion products, and the size of the reinforcing steel. The authors indicated that the proposed model agreed with the experimentally observed time-to-corrosion cracking of the cover concrete.
Pantazopoulou and Papoula, (2001) proposed a theoretical model to demonstrate the mechanical consequences of corrosion-product buildup around the bar. Service life is estimated as the time required for through cracking of the cover, which is identified in the model by a sudden drop in the internal pressure exerted by the corroding bar eventually relaxing to zero. The problem was modeled with reference to Fig. (2.8), wherein the reinforcing bar of initial radius $R_b$ is embedded in concrete with $C_c$, the cover dimension, measured from the center of the bar to the nearest free surface of concrete and $R_c$ defines the crack front. Assuming uniform corrosion on the bar surface, the bar diameter is reduced, because of iron depletion to $R_{rb}$,

$$R_{rb} = (R^2_b - \Delta V_s/p)^{1/2} \quad (2.13)$$

$$\Delta V_s = 3.68 \times 10^{-11} \pi D_b \Delta t i_{corr} \quad (m^3/m) \quad (2.14)$$

Where, $i_{corr}$ is in $Ap/m^2$, $D_b$ is in m, and $\Delta t$ is in seconds

The oxide layer thickness that builds up around the bar is denoted as $t_r$, given by:

$$t_r = \sqrt{R^2_{rb} + \frac{\Delta V_r}{\pi}} - R_{rb} \quad (2.15)$$

And

$$\Delta V_r = \pi (R^2_r - R^2_{rb}) \quad (2.16)$$

Figure 2.8: Idealization of Cover Concrete as Thick-Walled Cylinder: (a) Cylinder Model; (b) (c) Definition of Terms; (d) Rust Deposited within Open Cracks (Pantazopoulou and Papoula, 2001)
Thus, for a given corrosion current density $i_{\text{corr}}$, the volume of dissolved iron $\Delta V_s$ and associated volume of rust product $\Delta V_r$ are calculated, then the thickness of rust layer $t_r$ is obtained. To calculate the time to a predefined level of damage (e.g., through cracking of the cover), the stress state generated in the cover due to the accumulation of $t_r$ must be resolved. Deformation capacity and residual strength in tension were gauged from the characteristic fracture properties of the material. The resulting boundary-value problem was solved in space and time using a finite-difference discretization. This analytical model was compared with experimental results presented by (Liu and Weyers, 1998) to check the applicability of the model and it was illustrated that the model is capable of reproducing successfully the experimental trends and gives reasonable estimates for the time and critical mass of rust associated with cover cracking.

A theoretical model of the cover layer cracking in reinforced concrete structures due to corrosion expansion of reinforcement and uniform stress at infinity was proposed by (Li, et al., 2008). It was assumed that the corrosion on the surface of bar and stresses at infinity are uniform. Fig. (2.9) showed the model used in the study.

![Figure 2.9: Model for Estimation of the Stresses Under the Coupled Action of Corrosion Expansion and Uniform Stresses at Infinity (Li, et al., 2008)](image)

1 refers to reinforcing rebar
2 refers to corrosion products
3 refers to concrete
The tangential stresses in concrete subjected to corrosion expansion and uniform stresses at infinity were derived and expressed as

\[ \sigma_3 \theta = \sigma_1 \theta + \sigma_2 \theta \]  

(2.17)

Another formula was presented to estimate the time for first crack initiation, given as:

\[ t_c = \frac{(f_c - \sigma_{3\theta_{\text{max}}}) r_2}{0.0232 G_3 t p} \]  

(2.18)

Where, \( i \) is the current density, \( G_3 \) is the shear modulus of concrete and \( p \) is coefficient of expansion

When the applied stresses at infinity are low, no damage of concrete occurs only due to the uniform stresses, it was found from Eqs. (2.16) and (2.17) that the maximum value of tensile hoop stress \( \sigma_{3\theta_{\text{max}}} \) caused by the uniform stresses at infinity happens at the corrosion products–concrete interface for any \( \theta \) value when \( \sigma_1 = \sigma_2 \).

In addition, it was stated that \( \sigma_{3\theta_{\text{max}}} \) occurs at \( \theta = 0 \) along the corrosion products–concrete interface for \( \sigma_2 > \sigma_1 > 0 \) and at \( \theta = 90 \) for \( \sigma_1 > \sigma_2 > 0 \), and in both cases, the crack initiation may be accelerated. On the contrary, the crack initiation may be decelerated if \( \theta = 0 \) and \( |\sigma_2| > |\sigma_1| \), or \( \theta = 90 \) and \( |\sigma_2| < |\sigma_1| \), with noting that \( \sigma_2 \) and \( \sigma_1 < 0 \), indicating that the change of stress state at infinity can accelerate or decelerate the initiation of a crack.

The researchers also investigated the effect of the shear modulus ratio between concrete and reinforcing bar on the crack initiation and they have found that there is no significant influence of it on the crack.

A configuration model of corroded bar section was built by Yuan and Ji, (2009) by examining a cylinder specimen cut from a corroded specimen, the dimensions of the
cylinder specimens including the corroded steel bar are about 50 mm in diameter and 25 mm in height. Digital optical microscope was used for observing the distribution of the corrosion products layer along the perimeter of the steel bar. The observations showed that the corrosion products distributed only on the upper half circumference of the steel bar facing the concrete cover in the appearance of half ellipse, no corrosion was found at the side away from the concrete cover.

Based on the assumption that the distribution pattern of the corroded part of the steel bar section is proportional to the thickness distribution of the corrosion products layer, the expression of the model shown in Fig.(2.10) is given by:

$$d(\theta) = R - \frac{R(1-2\eta)}{\sqrt{(1-2\eta)^2 \cos^2 \theta + \sin^2 \theta}}$$

(2.19)

Where, R is the radius of non-corroded steel bar; da is maximum corroded thickness nearest the concrete cover ($\theta = 90$), dq is the corroded thickness at ($\theta$) under polar coordinate, $\eta$ is corrosion level in the section of the steel bar by weight.

![Figure 2.10: Model of Corroded Section Configuration (Yuan and Ji, 2009)](image)

In accordance to the observation, it was indicated that the Interface transition zone between non-corroded steel bar and concrete has a high porosity which offers space for expansion.
of corrosion products, the corrosion products diffuse into this region and hence the expansive pressure is developed.

Bohner et al.,(2010) developed a comprehensive analytical prediction model, which describes the time-dependent damage process of cracking and spalling under realistic conditions. The main target of the work is to determine the mechanical properties of the corrosion products by applying novel experiments and inverse analyses. The experimental work was represented by 275 concrete cylinders specimens of height (30cm) with centrically embedded reinforcing bars using different bar diameter (8, 16, 24mm) with different concrete cover (10, 20, 30, 40mm), the cylinder diameter was equal to (2 times the concrete cover +bar diameter). After 7 days curing, the specimens were corroded by adding chloride (2.5%) of cement mass to the fresh concrete or by an accelerated carbonation of concrete (1%) of volume.

Furthermore, laboratory tests were performed to quantify the material properties of the concrete (compressive and tensile strength as well as the modulus of elasticity and the fracture energy of the concrete. Further investigations using optical microscopy, Raman spectroscopy and computer tomography were carried out to identify the type, location and amount of corrosion products that migrate into voids and emerging cracks in the concrete. The intensity of the internal corrosion pressure and tangential strains for the reinforcement were measured.

The numerical model was carried out with the finite element software DIANA. The cross-section details and material properties determined in the laboratory tests were implemented in the model. The strains and stresses were carried out in the analysis using time step. The following formula was predicted to express the free expansion of the reinforcing bar:

\[ \text{Free Expansion} = \frac{\text{Bar Diameter}}{2} \times \text{Concrete Cover} \]
\[ \Delta r = -\frac{d_i}{2} + \frac{1}{2} \left( \frac{d_i}{2} \right)^2 - \frac{1}{4} \left( \frac{d_i}{2} - (d_i - 2x)^2 - \phi \cdot d_i^2 \cdot \left( 1 - \left( \frac{1 - 2x}{d_i} \right)^2 \right) \right) \]  

(2.20)

Where, \( \Delta r \) = free increase in reinforcing bar radius, mm; 
\( d_i \) = initial reinforcing bar diameter, mm; \( x \) = loss of reinforcing bar radius, mm 
\( \phi \) = volume ratio chosen to equal 3 for uniform corrosion and 2 for pitting corrosion

The time until cracking of the concrete cover was also analyzed by means of the different investigated for both uniform and pitting corrosion, as shown in Figure (2.11). The crack width of about \( (10^{-5} \text{ to} 10^{-4}) \) mm was corresponding to a cracked element. A general conclusion between the ratio of concrete cover and reinforcing bar diameter, c/d-ratio and the time to cracking of the concrete cover couldn’t be drawn so far. However, it was indicated that the time to cracking increased with the increase of the concrete cover depth and with the decrease of the reinforcing bar diameter.

![Figure 2.11: Time to Cracking for the Outmost Elements (crack width 10^{-5} to 10^{-4} mm) Investigated on Different Analyzed Models for both Pitting and Uniform Corrosion (Bohner et al.,2010)](image-url)
It was stated that the analytic model will be suitable for different and changing conditions regarding the geometry of concrete cover and reinforcing bar diameter, material properties and climatic conditions for the prediction of the damage development.

Jabbour and Pérez, 2010, presented an experimental study for circular spirally reinforced concrete columns subjected to accelerated corrosion and sustained constant compression load. The RC column specimens used in this study consist of a 260-mm diameter circular cross-section reinforced with 6-ϕ 16mm longitudinal bars and spirals ϕ11.3 mm. The concrete cover to spiral reinforcement was 20 mm. The specimens were divided into two groups, one group (CV) where vertical reinforcement was corroded and isolated from non-confining spiral reinforcement and the other (CS) where spiral reinforcement was corroded and isolated from the vertical reinforcement to study the effect of confinement. Four of columns were subjected to accelerated corrosion for four months while the other four subjected to accelerated corrosion for ten months.

It was observed that the crack pattern for columns group CV was longitudinal along the vertical reinforcement. On the other hand, for columns group CS, minor hairline smaller and more scattered cracks along the spiral reinforcement were observed.

Figures (2.12-a and b) show the crack width propagation with respect the time. From these figures, it can be noted that the crack width increases with time and the maximum lateral crack width for group CS 0.083 mm, while the maximum longitudinal crack width for group CV was measured to be 0.712 mm.

As a result, it was concluded that for reinforced concrete columns the influence of the corrosion of longitudinal bars is more critical and causes significant cracks in the concrete cover which leads to the spalling over time that affects the performance and serviceability
of the columns. The nonuniform volume expansion around the corroded reinforcing bars which causes stress field leads to deformation of the surrounding concrete was represented by a theoretical model in terms of displacement proposed by Xia et.al, 2012. The displacement model is expressed in a polar coordinate system as:

\[ u(\theta')|r' = R_o = \frac{1}{2} (\delta_{max} - \delta_{min})(1 + \sin^3 \theta' - \cos^2 \theta') + \delta_{min} \]  

(2.21)

\[ u(\theta')|r' = R_o \] is corrosion-induced displacement on the internal boundary of concrete; \( r' \) and \( \theta' \) are the polar radius and polar angle, respectively; and \( \delta_{max} \) and \( \delta_{min} \) are the maximum and minimum thickness of the layer of corrosion products that is responsible for the generation of tensile stress, respectively.

![Graphs](image)

**Figure 2.12:** a) Longitudinal Cracking Width over Time in Columns Group CV (Jabbour and Pérez, 2010)

The proposed theoretical displacement model was compared with the experimental data provided by (Yuan and Ji 2009). Based on the computed corrosion induced stress results, it was concluded that the tensile hoop stress and compressive radial stress increase...
with the decrease of concrete cover. Also, for a given structural component the tensile stresses around the corner bars are greater and much more serious than that around internal bars because of their position which is close to the outside environment and the corrosive agents will simultaneously diffuse through cover from two directions. Therefore, the corrosion-induced stress field around a corner bar can be obtained through the superposition of stress states around a middle bar as shown in Figure (2.13).

Figure 2.13: Computation of Corrosion-Induced Stresses around a Corner Bar through Superposition (Xia et.al, 2012)

2.4 Influence of Corrosion on Reinforced Concrete Structures

2.4.1 Introduction

The service life of the reinforced concrete structures is greatly affected by the corrosion of the reinforcing bars. The corrosion products accumulated around the corroded bars cause internal pressure to the surrounding concrete surface leads to cracking since the concrete has limited plastic deformation and weak tensile strength. Over time, the propagation of the cracks leads to spalling and causes deterioration which affects the serviceability of the structure, moreover, the concrete cover spalling and the effect of corrosion on the steel bar cross-section and ductility and loss of bond between reinforcing bar and concrete all result in reduction of the strength of the structure and must be checked under the applied load.
In this section, some of the studies related to the effect of corrosion on the members of reinforced concrete structures are reviewed.

2.4.2 Effect of Corrosion on the Reinforced Concrete Beams

The behavior of beams subjected to corrosion due to marine environment was studied by Umoto and Misra, (1984). The beams were cast as two sets; Type A dimensions (10x10x70cm), concrete cover (1 or 2cm) and reinforcement ratio (2.09 and 1.85%), Type B (10x20x210cm) with stirrups (6mm) spaced at 170mm, concrete cover (1cm) and reinforcement ratio (2.28%). An accelerated corrosion was used in the experiment using sodium chloride (NaCl) of content ranging (0-6.3 kg/m³) dissolved in the mixing water at the time of casting. A constant current of (167mA) was passed for (10days) for type A and (1A) for (14day) for type B.

Depending on the test results, the following observations were reported:

- The cracks along the stirrups appeared much earlier than the longitudinal bars since they have less concrete cover.
- For Type B beams, the measured crack width was 0.1mm after (2weeks), one of the specimens had been left in the air for (3months) caused the crack to propagate to 0.9mm.
- Regarding failure mode, flexural and shear cracks were connected to the cracks by corrosion at the maximum load for beams Type A which were singly reinforced, and the concrete spalled off. While for beams Type B which were doubly reinforced, the failure mode was flexural accompanied by buckling of the compression concrete.
- The loss in observed energy was calculated for beam Type B and it was found to be less than that of the non-corroded beam by (17, 34 and 46%) which indicates a loss in ductility while the corrosion progresses.
The reduction in strength for beams Type A ranged from (5%) to (33%) depending on the (NaCl) content. For non-corroded beams, the reduction in strength was (2%) for (NaCl) content (1.01 and 6.63kg/m$^3$), while the corroded beams showed a reduction in strength by (5%, 7% and 33%) in comparison with non-corroded beams with correspondence to (NaCl) content (0, 1.01 and 6.63kg/m$^3$), respectively.

Rodriguez et al., (1997) carried out an experimental work to study the corrosion effect on the performance of beams considering different variables. Six types of simply supported beams specimens with different detailing but same cross section (150x200x2300mm) were used in the experiments. For Type 1, only bottom bars were corroded, while for other types, longitudinal and shear reinforcement were corroded. Two types had different tensile steel ratio, other two had compressive steel ratio and the last two types had a different spacing of shear reinforcement. To accelerate the steel corrosion, calcium chloride was added as (3%) by weight of cement to the mixing water and a constant current density of about (100A/cm$^2$) current was applied.

The test readings were recorded when the applied load reached the service load and when the applied load reached the ultimate and caused the failure of the beam.

The following conclusions were drawn due to test results:

- The deflection at mid-span and crack width increase for corroded beams at the service load.
- The flexural and shear strength decrease at the ultimate load.
- Three types of failure were noticed for the specimens,

Type 1 failure by bending at the bottom reinforcement occurred in beams with a low ratio of tensile reinforcement.
Type 2 failure by bending at concrete occurred in beams with a high ratio of tensile reinforcement.

Type 3 failure by shear occurred in all corroded beams with a high ratio of tensile reinforcement and large spacing of shear reinforcement, due to the reduction of the effective depth at shear span resulting from the spalling of the top concrete cover.

2.4.3 Bond Strength for Corroded Reinforced Structures

Mehdi, 1969 studied the effect of corrosion and bar spacing on the bond strength between reinforcing bars and concrete. An experimental work was conducted on (115) eccentric bond pullout specimens. The specimens were exposed to seawater and the results indicated some reduction in the bond strength. Concrete samples (150x150x150mm) reinforced with (8, 10, 12 and 16mm) corroded steel bars were tested after exposure to atmosphere for (16 months), the test results showed that the bond stress values were about (1.7) times the design values according to ACI and (2.8) times the ultimate local bond stress values per CP 110, while the bond strength for (16mm) bar didn’t change, (Maslehuddin, et al., 1990). They attributed the increase in the bond stresses to the plain surface produced from the filling up the gaps between the lugs due to rust which leads to a mechanical interlocking problem. In addition, they noticed spalling of the oxide layer from the lugs which accelerates corrosion at lugs.

(Al-Sulaimani et al., 1990) carried out pullout tests on (5.9 in) cubic concrete specimens with (0.39, 0.55, and 0.79 in) and 12 beam specimens (5.9 x 5.9 in) in cross section and (39.4 in) in length. Each beam was reinforced with one (0.47 in). An accelerated corrosion was applied to the specimens to study its effect on bond strength. For the pullout tests, the loaded-end and free-end slips were recorded for each load level. Beam
specimens were tested as simply supported beams under a two-point loading with a total span of (35.4 in) and shear span of (11.8 in). For each load level, deflections at midspan and at load points, as well as free-end slips at both ends of the bar, were recorded.

It was observed that the bond strength increased with corrosion increasing to about 1%, then the bond strength decreased with the increase of the corrosion level to become negligible at a degree of corrosion of (7.5%). For beam specimens, the bond strength increased with corrosion increasing to 0.5%, then the bond strength decreased gradually with the increase of the corrosion level. The researchers attributed the initial increase in bond to the increase in roughness of the bars with the growth of a firm layer of corrosion products and with the increase in the corrosion level the deterioration in reinforcing bar ribs and hence the reduced concrete confinement of the bar influenced the bond behavior. Both pullout and beam tests showed a sharp jump in the value of the free-end slip with the opening of a longitudinal crack indicating a sudden loss of bar confinement. The critical slip in the post-cracking corrosion stage for the maximum corrosion level studied was about 10 times the value of the non-corroded situation.

(Wang and Liu, 2004) proposed an analytical model to predict the effect of corrosion on the bond strength, the pressure due to expansive action of the corrosion products was estimated, the bond strength was calculated taking into account the reduction of bar confinement caused by cover cracking, change of friction coefficient between the steel and the concrete, and reduction of the friction force on the bearing face and the deterioration of the ribs of the deformed bars. The ultimate bond strength of corroded reinforcements was formulated as a function of splitting bond strength, corrosion pressure prior to the loading and the angle of the face of crushed concrete. A comparison between
the analytical model and previous experimental work carried out by (Al-Sulaimani, et al., 1990) showed a good agreement.

Bajaj (2012) studied the effect of corrosion on the mechanical properties of reinforced concrete. Bond strength for different corrosion levels was one of the investigations considered in this study. Two series of cylindrical specimens of diameter 2 in with reinforcing bar 0.375 in and concrete cover 0.8125 in, for first series the specimens, were of length 12 in with embedded length 12 in, while for the second series the specimens were of length 9 in with embedded length 5 in. An accelerated corrosion test was conducted on each specimen to achieve the desired degree of corrosion after immersing it partially in 3.5% NaCl solution by weight for 72 hours.

Figure (2.14) shows the bond strength values the degree of corrosion, it is observed that series 2, up to 2% corrosion level, the failure mode was yielding of the steel at the threaded portion, while for higher corrosion levels, the failure mode was splitting of concrete. Thus, the bond strength increased up to 2% corrosion level and then it decreased by the increase of the corrosion level to lose 40% of its value in correspondence to corrosion level (6%).

![Figure 2.14: Bond Strength vs. Degree of Corrosion (Bajaj,2012)](image-url)
Similar observations were noticed for series 3, but the bond strength for series 3 specimens was more than that of series 2 specimens for all the corrosion level. The specimen length to embedded length ratio for series 2 and series 3 was 1 and 1.8, respectively. Also, the crack width for series 2 was higher than series 3 for a particular corrosion level. This higher crack width resulted in more reduction in confinement effect, which resulted in low bond strength.

2.4.4 Effect of Corrosion on the Reinforced Concrete Slabs

An Experimental work was carried out by Almusallam et al. (1996), to investigate the structural behavior of simply supported one way-slab subjected to a uniform distributed load. The specimens were with dimensions (12 X 28 X 2.5 in.), five No.2 at 2.25 in c/c spacing main reinforcement was used and 3/8 in clear cover was used. To accelerate reinforcement corrosion, direct current was impressed on the bars, using a direct current (DC) rectifier. The specimens were partially immersed in a 5% sodium chloride solution. The slabs were tested in flexure using a simply supported system with a total span of (24 in.). A uniformly distributed load was applied; the load and deflection data were recorded. Due to the test results, the following conclusions were drawn:

- An increase in the ultimate strength of slabs with slightly corroded bars was indicated prior to the formation of cracks. However, a sharp reduction in the ultimate strength was measured in slabs with increasing reinforcement corrosion. The reduction in the ultimate flexural strength of slabs with 5% reinforcement corrosion was 25%, while it was 60% in the slabs with 25% reinforcement corrosion.
The increase in the ultimate strength of the slabs in the pre-cracking state was attributed to an increase in the roughness of the bars due to the growth of firm rust on their surface, which increase the bond strength.

The slabs with non-corroded steel bars and that with low corrosion level up to 1.5% showed a considerable ductility. With the increase of corrosion level, a progressive loss in the ductile behavior of the slabs was noted.

A flexural mode of failure by yielding of steel in slabs with both non-corroded and corroded steel in the pre-cracking stage was noted. In slabs with corroded bars, in the post-cracking stage, a bond-shear type of failure was indicated due to the spalling of the concrete cover caused by reinforcement corrosion.

It was found that the ultimate strength of unreinforced slabs is similar to that reinforced by 60% corroded bars, while the slabs with 75% corrosion showed larger deflection values before failure in comparison with unreinforced slabs which indicated even highly corroded steel reinforcement improves the brittle nature of plain cement concrete slabs.

Chung et al. (2004), performed experimental flexural tests to investigate a correction factor for development length for slab specimens with corroded reinforcing bars. The specimens’ dimensions were (500x1100x90mm) reinforced with (φ10mm spaced at 100mm c/c). An accelerated corrosion process was applied by adding 3% of cement weight of chloride admixture. Different levels of corrosion ranged (1%-15%) of the origin steel bars cross section was used. Different values of development lengths ranged (1-44 cm) were adopted for the specimens. A flexural test was carried out by applying two point loads to the specimens. From the test results, it was observed that for low corrosion level
(0-3%), a large number of cracks with small width was noted and the failure mode was flexural. While for high corrosion levels, less number of cracks and wide width was noted. The deformation increased rapidly that showed brittle behavior as that of plain concrete.

Based on the test results of this study, the researchers suggested a correction factor for ACI development length given by the following equation:

\[ \ell'_d = \ell_d \frac{1}{\delta} \]  

Where, \( \delta \) is correction factor equal to:

\[ \begin{align*}
1.0 & \quad \text{for } C_0 \leq 2 \\
2.09 C_0^{-1.06} & \quad \text{for } C_0 > 2
\end{align*} \]  

\( C_0 \) is the corrosion level in percentage.

Gao et al. (2016a, 2016b) conducted an experimental work on eighteen RC slabs exposed to accelerated corrosion with different test conditions. The slab specimens were with dimensions (12x3x28) in. A 5 % NaCl solution along with the impressed current was applied during the corrosion process and the corrosion level was (10%). A two-day wetting and one-day drying cycle was followed for 21 days. The specimens were divided into three groups, without pre-existing cracks and sustained load, with pre-existing cracks but no sustained load, and specimens with pre-existing cracks and sustained load. The relationship between the mass loss, longitudinal and transverse crack width and the corrosion time was studied. It was found that the maximum mass loss occurred at the middle of the slab span. For the crack width, it was observed that the crack width increased with time for both longitudinal and transverse cracks, the specimens subjected to pre-existing cracks and sustained load exhibit wider transverse cracks and had the maximum longitudinal crack width. A comparison was made between the un-corroded and corroded specimens related
to the failure mode and load-deflection relationship as shown in Figure (2.15-a and b) respectively. It was indicated that the un-corroded slabs exhibited ductile failure mode through the observation of several flexural cracks while the corroded slab specimens exhibited one wide crack running in the transverse direction indicating a brittle failure mode. The corroded specimens were observed to suffer larger deflections at smaller loads than those for the corresponding un-corroded specimens and the reduction in the load carrying capacity was found to be (23-27%) for the corroded specimens with pre-existing cracks and sustained load.

![Figure 2.15: Crack Pattern and Load-Deflection Behavior for Un-Corroded and Corroded Specimens (Gao et al, 2016)](image)

2.4.5 Effect of Corrosion on the Reinforced Concrete Columns

Concrete columns with reinforcing bars subjected to marine environment were tested by Uomoto and Misra (1984) to investigate how the corrosion changes their behavior. The columns specimens were of dimensions (10x10x740cm), concrete cover (2cm) with (6mm) ties spaced at 75mm, concrete cover (1cm) and the reinforcement ratio (3.1%). Accelerated corrosion was used in the experiment using sodium chloride (NaCl)
of content (0, 1.01 and 6.63 kg/m³) dissolved in the mixing water at the time of casting. A constant current of (45mA and 180mA) was passed for (10days) as a maximum.

It was observed that the cracks formed along the main reinforcement were more than cracks along the ties. The reduction in strength for the corroded specimens was (20%-30%) of the non-corroded specimens due to spalling of the concrete cover.

Revathy et al.(2009) tested circular concrete columns under uniaxial compression loading to investigate the effect of corrosion degree on the strength capacity and ductility. The specimens were 150 mm in diameter and 900 mm in height provided with six bars of 8 mm diameter as longitudinal reinforcement. Each specimen also contained 6 mm diameter stirrups with a spacing of 115 mm c/c.

Two different degrees of corrosion damage of 10 and 25% were induced through an accelerated corrosion and by adding 3.5% NaCl solution. All specimens were tested under compression loading, the displacement values and ultimate capacity were recorded.

Figure 2.16 shows the effect of corrosion degree on the ultimate load and ductility. It was observed that the reduction in the ultimate strength of the corroded specimens by 3% and 12% for the corrosion degree 10% and 25% respectively was observed. For the ductility of the columns, the deflection was decreased by 1.5% and 9% for columns subjected to 10% and 25% corrosion damage level, respectively which indicates a reduction in ductility of the columns.
Wang and Liang (2008) presented an experimental work to determine the load capacity of reinforced concrete columns with partial length corrosion. The specimens were 1300mm long with a rectangular cross-section of 200mm×200mm reinforced with four 18mm diameter deformed bars. The shear reinforcement consisted of 8mm diameter plain stirrups spaced at 50mm in the column ends and at 100mm in the center part.

3.5% of calcium chloride (CaCl2) by weight of cement to the mixing water during concrete mixing and an accelerated corrosion was subjected. The specimens were divided to two sets, first set the corroded partial length (350 or 700mm) was in the tensile zone, while the corroded partial length was in the compression zone for the other set.

Seven specimens were subjected to eccentric load with e/h about (0.25) and five specimens were subjected to eccentric load with e/h about (0.75).

The following observations were recorded for the tested specimens:

- Failure mode for the RC columns with different corrosion levels and partial lengths was approximately the same for both sets in which the tensile concrete was cracked firstly, with the increase of the load the tensile steel yielded and later one compressive steel bar
except for the specimens with large eccentricity and high level of corrosion that the failure cracks of the test specimens developed from or located nearby or merged with the longitudinal corrosion cracks within the partial length corrosion. The specimens with large eccentricity and high corrosion level in tensile zone tend to behave in a brittle mode.

- For the corroded columns with large eccentricity, comparing with the corresponding non-corroded RC columns, greater reduction of load capacity was noted for the columns with partial length in the tensile zone; while low reduction of load capacity was obtained in the columns with partial length in the compression zone. For columns with large eccentricity and high corrosion level in the tensile zone, the reduction in ultimate strength was found to be 23.4% of that of non-corroded column as shown in Figure 2.17. The researchers attributed that to the existence of the longitudinal corrosion cracks within the partial length and the reduction of the confinement action provided by the stirrups, the longitudinally mechanical integrality of the column was destroyed, and a weak zone was formed lead to that reduction in strength.

- For the corroded columns with small eccentricity, comparing with the corresponding non-corroded RC columns, greater reduction of load capacity was obtained for the columns with partial length in the compression zone with maximum reduction in strength by 34.2%; and relatively low reduction of load capacity was obtained in the columns with partial length in the tensile zone.

A similar study was carried out by Azad and Al-Osta (2014), to study the effect of corrosion on the eccentrically loaded columns; the specimens were with rectangular section (220x220mm) reinforced by 4ϕ20 and (180x180mm) reinforced by 4ϕ18 subjected to accelerated corrosion for (6-13) days. Eccentric loads were applied to the columns of
different e/h (0.16, 0.33 and 0.52). The experimental failure loads of the corroded columns were compared with that of the non-corroded specimen. For the same corrosion level, it was noted that the reduction in strength for columns with smaller e/h=0.16 was more than the reduction in strength for the columns for e/h=0.52. The reason of this behavior is that the cracking of concrete cover effectively reduces the section’s compressive strength for small eccentricity, as the area of the concrete cover constitutes a sizeable portion of the gross concrete section. For a larger e/h, the bars on the tension side are in tension; the cracking of concrete cover on tension side is therefore not as critical as the section fully in compression.

Similarly, for the comparison between the experimental values of failure loads of corroded columns and theoretical failure load of non-corroded columns shown in Figure 2.18, the columns with larger eccentricity showed a less reduction in strength than that of smaller eccentricity. This implied that the calculation of the theoretical strength depending only on the reduction of the reinforcing area doesn’t represent the actual strength of the corroded columns.

From the experimental observations, a formula for a reduction factor was predicted as follows:

$$\alpha = 1.323 \left[ \frac{(e/h)^{0.14} \left( \frac{D'}{D} \right)^{1.48}}{I_{corr} T} \right]^{0.352} \quad (\alpha \leq 1) \quad (2.23)$$

Where, D' is the diameter of the corroded reinforcing bar;

D is the diameter of the non-corroded reinforcing bar;

e is the eccentricity of the applied load; h is the depth of cross-section;

I_{corr} is the corrosion current density, mA/cm²; and T is the duration of corrosion, days
To check the applicability of the proposed formula, the experimental failure loads of corroded specimens for Wang and Liang (2008) were compared with the theoretical values using the reduction factor. It was seen that, with the exception of two specimens, the predicted strength values differ less than 10% from the test results.

Figure 2.17: Experimental and Calculated Values Mu–Nu Interaction Diagram for the RC Columns (Wang and Liang 2008)

Figure 2.18: Interaction Diagram for Failure Load for Specimens (Azad and Al-Osta 2014)
Tapan and Aboutaha (2008 and 2011) proposed a strength evaluation method for a corroded bridge pier column by developing moment-axial load interaction diagrams using damaged material properties. The stress and area of corroded reinforcement are calculated as:

\[ f = (1 - 0.005Q_{corr}) f_y \]  
\[ A_s = (1 - 0.01Q_{corr}) A_{so} \]

\[ Q_{corr} = 0.046 \frac{l_{corr}}{D_b} \]

Where,

\( f \) and \( f_y \) are the yield strength of corroded and non-corroded reinforcement, respectively

\( A_s \) and \( A_{so} \) are cross-sectional area of corroded and non-corroded reinforcement, respectively, \( Q_{corr} \) is the amount of corrosion of reinforcement (%)

\( l_{corr} \) is the corrosion rate of reinforcement (µA/cm), \( D_b \) is the diameter of non-corroded reinforcement, and \( t \) is the time elapsed since the initiation of corrosion (years)

For the cracked concrete cover section, the researchers adapted the model proposed by (Pantazopoulou and Papoulia 2001, Wang and Liu 2004) which is based on a volume increase of the corrosive products. The solution of a thick-walled cylinder under uniform internal pressure has been used to model the internal pressure exerted by the corroded bar on the surrounding concrete. The concrete cover cracking is assumed to occur when the maximum hoop stress is equal to the tensile strength of concrete. The amount of corrosion to cause cover cracking is calculated as 2.25% for cover to longitudinal reinforcement diameter ratio equals to 1 (C/ D = 1), and 5.25% for C/D = 2.5. In this paper, the critical case (C/D=1) was selected for the analysis.
Six cases were assumed due to the location of the corroded bars with different corrosion rates to develop the interaction diagrams as shown in the Figure (2.19).

The interaction diagrams for all six cases indicated that the axial and flexural strength decrease as the amount of corrosion increases and there is a significant reduction in strength for the amount of corrosion 10%. For case I, the reduction in axial and flexural capacities under balanced condition was found to be more than the reduction in compression or tension controlled regions. For the Case II, where it is assumed that corrosion takes place at the extreme layer of tension bars, the reduction in axial load carrying capacity under pure compression is the same as for Case I. As the corrosion amount increases, the reduction in load carrying capacity under balanced condition and pure moment is significantly higher than Case I. For Case III, the reduction in axial and flexural capacity under balanced condition is higher than the reduction in compression or tension controlled region, but it is lower than Case I and II. For all corrosion levels the reduction in pure moment capacity for Case III is less than Case II. For the Case IV, where all bars are assumed to be corroded, it is obvious that the reduction in load carrying capacity is much more than all other cases investigated.

For Case V, and VI, the reduction in the load carrying capacity is almost the same, but the reduction at tension controlled region is more for Case V as the amount of corrosion increases beyond 10% corrosion level. The reduction in the compression controlled region is almost equal in both two cases for all four deterioration stages since tension bars have smaller stresses at the compression controlled region, corrosion of those bars have little effect on load carrying capacity. In summary, the reduction in load carrying capacity depends on the location of the deterioration and amount of corrosion.
Figure 2.19: Interaction Diagrams for Pre-Defined Deterioration stages, (Tapan and Aboutaha, 2011), where deterioration is on the:

a) compression side of the column section.
b) tension side of the column section.
c) on the left side of the column section.
d) all sides of the column section.
e) compression and left side of the column section.
f) tension and left side of the column section.
CHAPTER III

STRENGTH PREDICTION OF DETERIORATED RC COLUMNS: THEORETICAL ANALYSIS

3.1 Introduction

One of the obvious consequences of the corrosion of reinforcing bars in RC structures is the cracking of the concrete cover which causes loss of concrete section especially in the area surrounding the corroded reinforcing bars, and over time leads to cover spalling. Columns are compression members designed mainly to resist axial loads. As the concrete is considered an effective material to resist compressive stresses, the concrete cover spalling significantly affects the strength of the columns.

Figures (3.1) and (3.2) show examples of concrete cover spalling in deteriorated columns of rectangular and circular cross-sections, respectively.

In this chapter, a theoretical analysis to determine the strength reduction of axially loaded columns and development of the axial load-moment capacity due to concrete cover spalling is presented for both rectangular and circular cross-sections.
Figure 3.1: Concrete Cover Spalling in Deteriorated Rectangular Columns (Aboutaha 2004)

Figure 3.2: Concrete Cover Spalling in Deteriorated Circular Columns
3.2 Cover Spalling for Axially Loaded Columns

When a symmetric reinforced concrete column is subjected to a concentrated axial load, the two components of the cross section, concrete and steel, contribute to carry the applied load using the nonlinear response of both materials. The nominal column strength is simply defined in ACI-Code using the following equation:

\[
P_n = 0.85f'_c(A_g - A_{st}) + f_yA_{st}
\]  

(3.1)

Where,

\(A_g\) is the gross area

\(A_{st}\) is the total area of steel reinforcement

\(f'_c\) is the compressive strength of concrete

\(f_y\) is the yield strength of the steel bars

By assuming spalling of concrete cover, the gross area of the column cross section will be reduced to:

\[
A_{g,\text{red}} = A_g(1 - S\%)
\]  

(3.2)

Where, \(S\) is the cross section loss due to cover spalling as a percentage of the gross section

Equation (3.1) can be rewritten as:

\[
P_n = A_g[0.85f'_c(1 - S\% - \rho) + f_y\rho]
\]  

(3.3)

Figure (3.3) shows the nominal axial load capacity of a column cross section-rectangular or circular- versus the corrosion section loss for different steel reinforcement ratio.
Figure 3.3: Nominal Axial Load Capacity vs Corrosion Section Loss

From the above figure, it can be observed that the axial load capacity decreases with the increase of the percentage section loss for a given reinforcement ratio ($\rho$). The percentage reduction in axial load capacity decreases with the increase of ($\rho$) for a certain section loss. For example, for 20% loss of the gross section, the reduction in axial strength is 18.02% for ($\rho$) equal to 1%, while it is 14.89% for ($\rho$) equal to 4%. With the increase of the section loss, the difference in the strength reduction between any successive reinforcement ratio increases to be 10.32% for ($\rho$=1% and 4%) for section loss (50%).

3.3 Strength of Eccentrically Loaded Columns

The pure compression loads in the axially loaded members rarely occur. Columns resist moments in many cases such as moments due to eccentric loads carried by the brackets, moments due to the unbalanced moment at the end of the beams supported by the columns and moments resulted from the lateral loads (Wight and MacGregor, 2009, Nilson et al., 2004).
In the following sections, the effect of the concrete cover spalling on the strength of the eccentrically loaded columns is explained and determined.

3.3.1 Material Properties

The mechanical properties of the concrete and reinforcing bars in addition to the size of the cross section characterize the performance of the reinforced concrete member. The response of the materials to the applied loads is represented by the stress-strain relationship for both concrete and steel as discussed in the next subsections.

3.3.1.1 Concrete

The strength of concrete is affected by many factors: the water/cement ratio, properties of aggregates, type of cement, properties of supplementary cementitious materials if used, age of concrete, environmental conditions during concrete curing, dimensions and shapes of specimens, and loading rates (Wight and MacGregor, 2009, Nilson et al., 2004, McCormac and Brown, 2014, Gu et al., 2016). Concrete strongly influences the behavior of column since it has a high strength in compression. Typical stress strain curve for different values of concrete grade is shown in Figure (3.4). The stress and strain relationship is linearly elastic from zero up to about one-third to one-half the ultimate strength, then becomes nonlinear to reach a maximum strain of (0.0015-0.002) corresponding to the maximum strength in the ascending branch. The concrete ultimate strain ($\varepsilon_{cu}$) decreases with the increase of its ultimate strength. The presented theoretical work based mainly on the simplified method of ACI for the calculation of the compressive strength resulted from concrete. The assumed ultimate strain at the extreme compression value is taken as 0.003 for normal weight concrete and the ultimate strength is typically taken as $(0.85 f'_{c})$. Concrete tensile strength is neglected because the analysis goal is to
obtain the maximum strength capacity of the column. However, the difference in the theoretical analysis results if the simplified ACI equation or the real parabolic stress strain curve of concrete is used in calculation will be presented later in Chapter V.

![Typical Stress Strain Curves for Concrete](image)

Figure 3.4: Typical Stress Strain Curves for Concrete (Wight and MacGregor, 2009)

3.3.1.2 Steel

Steel reinforcement in columns is used to resist both tensile and compressive stresses. The steel stress-strain curve is identical in tension and compression. Typical stress-strain relationship is shown in Figure (3.5-a). It is observed that the stress-strain relationship is elastic and linear up to the yield point. After that, a nonlinear relationship is recognized to the ultimate stress then the strain is continuous to increase at a constant stress.
ACI code allows using a bilinear curve shown in Figure (3.5-b) for mild steel which is used in the calculation of the column strength capacity in this study.

The steel stress is given as:

\[ f_s = E_s \varepsilon_s \quad \text{if } f_s < f_y \]  \hspace{1cm} (3.4-a)

\[ f_s = f_y \quad \text{if } f_s \geq f_y \]  \hspace{1cm} (3.4-b)

Where,

\( f_s \) = Stress in steel, ksi.

\( f_y \) = Specified yield stress, ksi.

\( E_s \) = Modulus of elasticity taken as 29,000 ksi.

\( \varepsilon_s \) = Strain in steel in/in
3.3.2 Interaction Diagrams for Reinforced Concrete Columns

Whether the column is subjected to moment or eccentric load, same procedure is used to construct the load-moment interaction diagram since the applied axial load and moment can be replaced with an equal load with eccentricity. Although it is possible to derive a family of equations to evaluate the strength of columns subjected to combined bending and axial loads, these equations are tedious to use (Wight and MacGregor, 2009). For this reason, interaction diagrams for columns are generally computed by assuming a series of strain distributions, each corresponding to a particular point on the interaction diagram, and computing the corresponding values of $P$ and $M$.

Once enough such points have been computed, the results are summarized in an interaction diagram. For conventional reinforced concrete members, strains and stress changes can be determined in any typical section along the span using equilibrium equations, stress-strain relations, and strain compatibility.

Figure (3.6) shows a) a typical cross section for a rectangular column, b) strain distribution in which the concrete strain in the extreme compression fiber is assumed to be 0.003 and the strain at each fiber is proportional to the distance from the center of the fiber to the neutral axis and c) the stresses in concrete and steel using the equivalent rectangular block for the concrete stress.

The interaction diagram can be constructed through using the following steps:

- The depth of the neutral axis location (c) is assumed.
- For the simplification of calculation, ACI Code permits the use of an equivalent rectangular concrete stress distribution rather than using a closely representative stress-strain curve. The width of the rectangular stress block is constant and represented by a
uniform compressive stress equals to \((0.85f'_c)\). The depth of the rectangular stress block (a) is determined.

\[
a = \beta_1 c
\]

\[
\beta_1 = 0.85 - 0.05 \frac{f'c - 4000}{1000} \quad f'_c \text{ in psi}
\]

But not more than 0.85 and not less than 0.65

- The strain in each steel layer is determined as illustrated in Figure (3.6-b), then the stresses in these layers are computed.

- The internal forces for concrete and each steel layer are determined by multiplying the stresses by the areas acting on. Then, by summing all the forces the nominal axial load capacity \((P_n)\) is obtained. The nominal internal moment capacity \((M_n)\) is obtained by summing the moment of the forces for concrete and reinforcement about the centroid of the cross-section as illustrated in Fig.(3.6-c).

- The above steps are repeated for another assumed depth of neutral axis. For a set of (c) values, a corresponding set of points of \((P_n, M_n)\) which represent points on the interaction diagram.

Figure 3.6: Interaction Diagram Calculation Process
a) typical cross section for a rectangular column,
b) strain distribution  c) stresses and forces at nominal strength
Five points of pair \((P_n, M_n)\) are sufficient to construct the interaction diagram including a point for pure axial load \((e=0)\) and pure moment \((e=\infty)\). A typical interaction diagram is shown in Fig. (3.7), the radial lines represent a particular eccentricity. Three modes of failure can be recognized for a RC column subjected to axial load and moment due to the applied eccentricity. If the eccentricity is large, a small value of the depth of neutral axis will be resulted and then the strain in the tensile reinforcement will be greater than the yield strain which means that the steel will yield before the concrete crushes resulting in tension failure. The balanced failure occurs when the yielding of steel and crushing of the concrete takes place at the same time. Only one specific value of eccentricity and depth of neutral axis define the balanced failure as follows:

\[
c_b = \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y} d
\]  

(3.7)

Where, \(c_b\) is the balanced neutral axis

\(d\) is the effective depth of the cross section

\(\varepsilon_u\) is taken as 0.003

Small eccentricities cause to increase the depth of the neutral axis, when it becomes larger than \((c_b)\), the strain in the tensile reinforcement will be less than the yield strain which causes the column to fail in a brittle manner since the crushing of concrete occurs first. This type of failure is defined as tension failure.

For design, it should be taken into account that the cross section is adequate to resist the applied loads in which the load path doesn’t reach the limit curve to prevent the failure.
3.4 Strength Prediction of the Deteriorated Columns Subjected to Eccentric Loading

To estimate the load carrying capacity of RC columns subjected to corrosion in terms of concrete cover spalling, the interaction diagrams for rectangular and circular cross sections are developed for deteriorated columns assuming different locations of cover loss using Excel spreadsheets.

3.4.1 Interaction Diagram of Deteriorated Rectangular Sections

To develop the interaction diagram, the size of the cross-section and properties of the materials (concrete and steel) should be defined, the area of the steel reinforcement and the spacing between the steel layers should be specified and the depth of the concrete cover spalling is assumed. All possible location of concrete cover cracking along the cross-section faces due to corrosion is considered to investigate the column strength including six cases of concrete cover spalling, the cases are as follows:

Case I  Cover spalling at the column compression face.
Case II  Cover spalling at the column tension face.
Case III  Cover spalling at the column left side or right side.

Case IV  Cover spalling at all column faces.

Case V  Cover spalling at the compression and left sides of the column.

Case VI  Cover spalling at the tension and left side of the column.

Figure (3.8) shows the interaction diagram calculation process for Case I where the cover spalling occurs at the compression face of the column. In this case, the effective depth of the cross-section decreases by the amount of the cover spalling and the moment capacity of the section is determined about the plastic centroid which moves towards the tension fiber of the cross-section and the section becomes unsymmetrical. For the calculation of the internal force contributed by the concrete, the width of the cross-section is the same as the original column cross-section, while the moment arms of the steel and concrete internal forces to the plastic centroid are not the same as the original column cross section. The internal forces are computed as follows noting that the depth of the neutral axis is assumed as the distance from the bottom fiber of the cracked layer defined before.

\[ C_c = 0.85f'_c b \beta_1 c \]  \hspace{1cm} (3.8)

\[ T_i, C_{si} = A_{si} f_{si} \]  \hspace{1cm} (3.9)

\( C_c \) is the compressive force resulted from concrete block

\( T_i, C_i \) is the tensile or compressive force resulted from steel bars depending on their location in relation to the neutral axis, respectively.

\( f_{si} \) is the steel stress calculated from equation (3.4-a or b)
Figure 3.8: Interaction Diagram Calculation Process for Case I

For Case II where the cover spalling occurs at the tension face of the column shown in Figure (3.9), the effective depth of the cross section is the same as the original cross-section, and similarly to Case I the moment capacity of the section is determined about the plastic centroid which moves towards the compression fiber of the cross-section. Also same as Case I, the internal force contributed by the concrete is calculated using the same width of the original cross-section, while the moment arms of the steel and concrete internal forces to the plastic centroid are not the same as the original column cross section.

Figure 3.9: Interaction Diagram Calculation Process for Case II

For Case III where the cover spalling occurs at the right or left face of the column, the effective depth of the cross section remains the same as the original cross section. The internal force contributed by the concrete is calculated using the reduced width of the original cross section by the amount of the cover spalling, while the moment arms of the
steel and concrete internal forces are the same as the original cross section. Figure (3.10) shows the interaction diagram calculation process for Case III.

For Case IV where the cover spalling occurs at all column sides, the deteriorated section remains symmetrical about both axes and the effective depth of the cross section is reduced by the amount of the cover spalling. The internal force contributed by the concrete is calculated using the reduced width of the original cross section which is twice the amount of the cover spalling, while the moment arms of the steel and concrete internal forces are less than that of the original cross section due to the reduction of the whole cross-section depth. Figure (3.11) shows the interaction diagram calculation process for Case IV.
Finally, for Case V and VI as shown in Figures (3.12, 3.13), the calculations of the internal forces and the moment about the plastic centroid are the same as Case I and II, respectively. The internal force contributed by the concrete is computed as Case III that the width of the equivalent rectangular section is reduced by the amount of the cracked cover depth. In these two cases the column cross section will be unsymmetrical about both axis, so the moment carrying capacity will be reduced about minor and major axes and affecting the resistance to the applied biaxial moment.

![Cover Spalling at Extreme Compression Layer and Right or Left Column Side](image1)

**Figure 3.12: Interaction Diagram Calculation Process for Case V**

![Cover Spalling at Extreme Tension Layer and Right or Left Column Side](image2)

**Figure 3.13: Interaction Diagram Calculation Process for Case VI**
3.4.2 Interaction Diagram for Circular Sections

The interaction diagrams for a circular section can be constructed by using the strain compatibility solution (Wight and MacGregor, 2009). The compressive force resulted from the concrete is obtained by multiplying the stress \((0.85f_c)\) by the area of the segment within the depth of the stress block \((a)\), given as follows:

\[
A = h^2\left(\frac{\theta - \sin \theta \cos \theta}{4}\right)
\]  

(3.10)

The distance from the center of the segment to the centroid of the column cross-section denoted by \((y')\) is calculated as follows:

\[
y' = \frac{h \sin^3 \theta}{3(\theta - \sin \theta \cos \theta)}
\]  

(3.11)

Where: \(h\) is the diameter of the cross-section, \(\theta\) is sector angle in radian and calculated as:

\[
\theta = \cos^{-1}\left(\frac{2 - a}{h}\right) \quad \text{if } a \leq h/2, \theta < 90^\circ
\]  

(3.12-a)

\[
\theta = 180 - \cos^{-1}\left(\frac{a-h}{h}\right) \quad \text{if } a > h/2, \theta > 90^\circ
\]  

(3.12-b)

Using strain compatibility solution, the strain at the steel layers is determined as follows:

\[
\varepsilon'_s = \frac{\varepsilon_u}{c} (c - d')
\]  

(3.13-a)

\[
\varepsilon_s = \frac{\varepsilon_u}{c} (c - d)
\]  

(3.13-b)

\[
\varepsilon_{si} = \frac{\varepsilon_u}{c} \left(c - d' - \sum_{j=1}^{i} s_{j-1}\right) \quad i = 2, \ldots, m
\]  

(3.13-c)

\[
\varepsilon_{si} = \frac{\varepsilon_u}{c} \left(c - d' - 0.5\gamma h - \sum_{j=n-i+1}^{n-m} s_{j}\right) \quad i = m + 1, \ldots, n - 1
\]  

(3.13-d)

Where: \(\gamma h\) is the distance between the centers of the outside layers of bars.
d’ is the distance from the center of the first layer of bars to the extreme compression fiber.

m, n are the number of steel reinforcement layers in half circular cross section and the total number of steel layers, respectively.

\[
s_1 = 0.5\gamma_h (1 - \cos \zeta) \tag{3.14-a}
\]

\[
s_2 = 0.5\gamma_h (\cos \zeta - \sin (90 - 2\zeta)) \tag{3.14-b}
\]

\[
s_3 = 0.5\gamma_h (\sin 90 - 2\zeta) \tag{3.14-c}
\]

Where: \( \zeta = \frac{2\pi}{n_b} \) in radian

\( n_b \) is the total number of steel reinforcement

\( s_1, s_2 \) and \( s_3 \) are as shown in Figure (3.14)

The stresses at the steel layers are calculated using Eq. (3.4) and then the resulting forces of the concrete block and steel bars are computed. Finally, the moment of the cross section is calculated by summing the moment of each segment about the centroid.

The process of interaction diagram determination is shown in Figure (3.14)

![Interaction diagram Process for Circular Columns](image)

Figure 3.14: Interaction diagram Process for Circular Columns

For deteriorated circular sections, it was assumed that the concrete cover spalling occurs uniformly around the cross section. As for rectangular sections, all the properties of the materials and cross-section size should be defined. The value of concrete cover cracking depth is assumed, and the new diameter of the cross-section is calculated as:
\[ h_{cs} = h - 2d_{cs} \] 

(3.15)

where: \( h_{cs} \) is the diameter of the deteriorated circular column, and 
\( d_{cs} \) is the depth of the cracked layer.

Then, the same procedure is used to develop the interaction diagram for the deteriorated circular column by substituting the new diameter value in equations (3.10 to 3.13). Figure (3.15) summarizes the process of the interaction diagram development of the deteriorated circular section.

Figure 3.15: Interaction Diagram Process for Circular Columns
CHAPTER IV

EXPERIMENTAL WORK

To verify the theoretical analysis proposed for the deteriorated RC columns, experimental work was carried out on columns with a rectangular section for one case where the concrete cover spalling occurs at the compression side of the columns with rectangular sections. Also, few samples for columns subjected to accelerated corrosion were tested.

The test specimens for the columns with concrete cover spalling were divided into six sets according to dimensions, eccentricity and concrete cover spalling and one set of columns subjected to accelerated corrosion.

The experimental work for the column specimens in details is presented in this chapter.

4.1 Test Specimens

Six sets of column specimens were tested; all the specimens were of square cross-section with dimensions (7in x7in). The sets were divided as shown in Table (4.1).

Set#1 was conducted as a reference without concrete cover spalling to determine the column strength capacity for three values of eccentricity (0, 0.875 and 1.75) in, Set#2 was conducted to study the effect of concrete cover spalling on the reduction of the column strength with half concrete cover spalling if the axial load is applied by the same previous eccentricity value and the effect of the full concrete cover spalling on the column strength for the same values of small eccentricity was evaluated by testing Set#3. The axial load
with a large value of eccentricity (3.5, 5.5 and 7), \((e/h=0.5, 0.785 \text{ and } 1)\) was applied for Set#4, 5 and 6. Set#4 was conducted as a reference to determine the column strength capacity without any concrete cover spalling while the influence of the cover spalling on the axial load-moment capacity for the columns was studied by performing tests on Sets#5 and #6 if half or all of the concrete cover spalled, respectively.

### Table 4.1: Columns Specimens Details

<table>
<thead>
<tr>
<th>Set No.</th>
<th>No. of Specimens</th>
<th>Length, in</th>
<th>e/h</th>
<th>Concrete Spalling Depth/Concrete Cover Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>30</td>
<td>small</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>30</td>
<td>small</td>
<td>½</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>30</td>
<td>small</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>36</td>
<td>large</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>36</td>
<td>large</td>
<td>½</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>36</td>
<td>large</td>
<td>1</td>
</tr>
</tbody>
</table>

### 4.2 Concrete Mix Design

The materials used for concrete mix to get normal strength concrete for all specimens were ordinary (ASTM C150 Type I) Portland cement, #8 coarse aggregate, fine aggregate, water, and admixtures. Table (4.2) shows the mix design used for the concrete columns.
Table 4.2: Mix Design of Concrete Columns

<table>
<thead>
<tr>
<th>Mix Proportion of Concrete (per 1 cubic yard)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type I Cement (lb)</strong></td>
</tr>
<tr>
<td>564</td>
</tr>
</tbody>
</table>

4.3 Steel Reinforcement

Four standard #4 steel bars were used as the main longitudinal reinforcement for all specimens. The yield strength given for #4 bars is (60 ksi). #3 steel bars were used for the lateral reinforcement with spacing as shown in Figures (4.1). Ties at the ends were placed at a smaller spacing to increase column confinement and contribute in avoiding local buckling. Although the tie spacing according to ACI-Code is 7in for the prepared specimens, the maximum space used between ties is (4 in) to increase the moment capacity and ductility of the columns which was proved by (Sheikh and Yeh, 1990 and Saatcioglu, Salamat and Razvi, 1995). The longitudinal bars were welded to steel plates of (7inx7inx0.25in) at the ends of the columns. The spacing between the longitudinal bars is (4in) that the concrete cover is (1.5 in) from all sides. The steel reinforcements were supplied by Akron Rebar Company (Akron, Ohio). Axial tension tests were performed to determine the material properties such as yield strength, tensile strength and ultimate strength. The tests were conducted on #4 bars (with a nominal diameter of 0.5 in.). For axial testing, the bar specimens were trimmed to a length of 17 inches, and the cross-section at the middle region of each bar was reduced to 0.3 inches approximately as per ASTM A370 standards to ensure that failure would occur in that region (as shown in Figure 4.2).
Figure 4.1: Details of the Columns Specimens
a) Specimens with concentrated and small eccentric loads
b) Specimens with large eccentric loads
c) Cross section details

Note: All dimensions are in inches
Axial tension testing was conducted using an Instron (UTM-HYD type with Model#1000HDX) axial-torsion testing machine. The end portions of the rebar specimen were clamped between the jaws of the machine, and the middle region of the reinforcing bar was fitted with a 2-inch extensometer to capture the strain data. Displacement was applied at a strain rate of 0.005 in./in./min. until failure of the specimen. Figure (4.3) shows the setup for the axial testing.

Figure (4.4) shows the developed stress-strain curves for the bars, the results showed linear stress-strain profile until 62 ksi.
The average yield strength was found to be 62 ksi and the average ultimate strength is 110 ksi.

4.4 Casting Procedure

Before specimens casting, the steel bars were prepared for the application of strain gages, all the reinforcing bars in tension and compression regions were ground at the middle of the rebar length using a $\frac{3}{4}$"-thick grinding disc to grind the ribs on the bar. The surface was then smoothed using finer grit sanding tools to ensure that no undulations would remain on the surface. The grinding of the bar surface is shown in Figure (4.5). Then the surface of the bar was cleaned following a procedure suggested by Micro-Measurements. Once the surface was cleaned the strain gage was fixed to the prepared surface. After that, the strain gages were soldered to lead wires as shown in Figure (4.6).
The strain gage data were acquired through a data acquisition system during testing. A wooden mold shown in Figure (4.7) was painted with form oil was used to cast the specimens in the laboratory. All specimens were cast horizontally to minimize the variation of concrete strength in addition to the ease of casting.

The concrete was mixed according to ASTM standards in a mixer shown in Figure (4.8). After mixing, the concrete was placed into respective molds for each set and compaction was achieved by means of external vibration. Also, concrete cylinders of 4in×8in were cast for determining the compressive strength of concrete for each series. The
specimens were removed from the molds after one day and were cured for 28 days in the humid room.

Figure 4.7: The Formwork Used for the Specimens.

Figure 4.8: Concrete Mixer

For set#2 and set #5 with half of concrete cover spalling, the concrete cover within the middle one third of the column length was removed after (2 hours) of casting, same as for set #3 and set #6 in which all the concrete cover was removed as shown in Figure (4.9) for set#5 and set #6.
Before testing, the columns subjected to large eccentric loads were attached to steel bracket consist of five plates made of A-36 carbon hot-rolled steel supplied from Alro Steel (Cuyahoga Falls, Ohio). The side plates are of thickness (0.5in), while the others are (0.25in) in thickness. The plates were welded at the college Machine Shop, the bracket is shown in Figure (4.10) with clear spacing 0.5in from all sides which filled with grout to make sure that the columns adhered to the steel brackets. The grout has a high early strength up to 4300 psi after (8 hours) and of an ultimate strength 16000 psi.

Figure 4.9: Specimens After the Removal of Concrete Cover for sets#2 and #3.

Figure 4.10: Steel Bracket Details
4.5 Testing Procedure

The columns were tested using (INSTRON Universal Test Machine) shown in Figure (4.11) to apply axial compressive load with different eccentricities. The machine capacity is (236 kips). The columns were simply supported using rollers attached to the testing machine at top and bottom of columns. The steel and concrete strain gages were conducted to a system data that for a certain value of the load, the corresponding strain was taken. In addition, a dial gauge was used to measure the mid-height deflection during loading. To provide the pin-ended boundary condition for the columns, a roller with an additional loading plate made of steel A572 of dimensions (7x7x1.5) in was used to transfer the load from machine to the specimen as shown in Figure (4.12).

Figure (4.13-a) shows the test setup for Sets#1, 2 and 3, and the test setup for Sets#4, 5 and 6 for columns loaded with large eccentricities is shown in Figure (4.13-b).
The compressive strength of the concrete cylinders was tested after (7, 14, 21 and 28) days, Figure (4.14) shows the concrete compressive strength- age typical curve. Also, the cylinders were tested at the same day of the columns test in order to determine the
strength on the day of the test using a standard compressive testing machine as shown in Figure (4.15) and typical failure mode of concrete cylinders is shown in Figure (4.16). The concrete compressive strength ranged between (4400-5400) psi.

![Concrete Compressive Strength vs. Age](image1)

**Figure 4.14: Concrete Compressive Strength vs. Age**

![Compressive Strength Testing Machine](image2)

**Figure 4.15: Compressive Strength Testing Machine**
4.6 Experimentation on Corrosion of Columns

Tests on one set of columns were carried out to estimate the difference of the strength reduction value between the columns with concrete cover spalling only and that subjected to accelerated corrosion. The set includes three specimens of length 30 in. The specimens were cast at the same time as the casting the other specimens with same concrete mix design. The specimens’ cross section and reinforcement details are same as shown in Figure (4.1-a and c). The specimens were subjected to small eccentricity ratio with three different values of \((e/h)\) which are (0, 0.125 and 0.25).

4.7 Accelerated Corrosion Process

To study the effect of corrosion on the performance of the structural members, they are exposed to an accelerated corrosion to conduct laboratory tests instead of exposure to real atmospheric conditions since it is a time-consuming. The accelerated corrosion process is achieved by passing an electric current through the test specimens. For a specific corrosion amount, the time and current for the accelerated corrosion process are calculated.
using Faraday’s law to achieve a specific corrosion level. From previous research, it was found that the minimum percentage of corrosion amount that affects the performance of RC columns is 25%. In this study, a total corrosion time of 21 days was considered in the tests conducted in this study. During the 21-day testing period, a cycle of two days of wetting followed by one day of drying was followed, and the current was calculated for a two-week period (equivalent to 1,209,600 seconds) using Faraday’s equation:

$$\Delta m = \frac{MIt}{zF}$$

where $\Delta m$ is the mass of steel consumed (grams); $M$ is the atomic weight of the metal (56 grams (0.1232 lb) for Fe); $I$ is the current (amperes); $t$ is time (seconds); $z$ is the ionic charge (which is equal to 2); and $F$ is Faraday’s constant (96,500 amperes/second).

Table 4.3 presents the current calculated for a period of 14 days to achieve a corrosion level of 28%.

Table 4.3 Current Required for 14-Day Accelerated Corrosion Process

<table>
<thead>
<tr>
<th>$z$</th>
<th>$F$</th>
<th>$M$ (g)</th>
<th>$t$ (sec.)</th>
<th>Unit Weight (lb/ft)</th>
<th>Percentage loss (%)</th>
<th>Rebar Exposed Length (in.)</th>
<th>$I$ (amperes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>96,500</td>
<td>56</td>
<td>1,209,600</td>
<td>0.669</td>
<td>28</td>
<td>10</td>
<td>0.2</td>
</tr>
</tbody>
</table>

4.8 Corrosion Test Setup

The three specimens were subjected to accelerated corrosion in the compression region to compare their strength with the corresponding specimens with concrete cover spalling. To accomplish that, a salt solution tank made of plastic plates was glued on the compression face at the middle third of the columns of length (10in). A 5% NaCl solution was prepared and used in the tanks and a stainless-steel plate was placed in the tank to act as a cathode. A wire from the specimen was connected to the positive terminal of a DC
power supply, and a wire from the stainless-steel plate was connected to the negative terminal. The calculated current of 0.2A was applied during the wetting cycle of the corrosion process. Figure 4.17 shows a schematic of the setup as well as the flow of chloride ions during the corrosion process.

The salt solution was removed, and the surface of the specimen was cleaned on the third day, and the specimen was left to begin the drying cycle. Every fourth day, the salt solution was refilled in the tanks, and the process was repeated until the test duration reached a total of 21 days. Figure 4.18 shows the setup of a set of specimens undergoing corrosion.

![Figure 4.17 Schematic of the Corrosion Test Setup](image1)

![Figure 4.18 Setup of Specimens Undergoing Accelerated Corrosion.](image2)
4.9 Test of Corroded Specimens

At the end of the corrosion period, the corroded columns were tested by the same test machine used for the other experiments shown in Figure 4.11. The three specimens were subjected to axial load with different eccentricities (0, 0.875 and 1.75) in, the eccentricity was applied by controlling the rotational support at the end of the column. The columns were loaded up to the ultimate carrying capacity and the mid-height displacement was measured by dial gauge. Figure 4.19 shows the strength test setup for the corroded columns.

Figure 4.19 Corroded Specimens Test Setup
CHAPTER V

FACTORS INFLUENCING STRENGTH CAPACITY OF DETERIORATED RC COLUMNS

5.1 Introduction

Concrete cover spalling in RC columns may be caused by the corrosion of steel reinforcement, freezing and thawing or by exclusive and impulsive loading. However, there are several factors affecting the column strength capacity associated with the concrete cover spalling. The theoretical approach discussed in chapter III to determine the strength capacity of deteriorated columns for both rectangular and circular cross-sections, assuming six different cases of concrete cover loss for rectangular cross sections depending on the location of the cover spalling was used to develop the interaction diagrams considering the influence of some factors on the strength of deteriorated columns and is presented in this chapter.

5.2 Effect of Depth of Concrete Cover Loss

5.2.1 Rectangular Cross Sections

Figure (5.1) shows the properties of the column cross section adapted for the interaction diagram construction. The concrete compressive strength $f'c$, was chosen as 5000psi and the yield strength of steel $f_y$ is 60 ksi. The size of the cross-section is 24in in width and 36in in height. The concrete cover measured from the center of the nearest steel
reinforcement layer is 3.0 in and reinforcement ratio $\rho$ is (1%), assuming major axis bending.

\[ f'_c = 5ksi, f_y = 60ksi \]
\[ E_s = 29000ksi \]

Figure 5.1: Column Cross Section Properties under Study

To predict the reduction in column strength related to the depth of concrete cover loss, three values of the amount of cover spalling (one third concrete cover, two-thirds concrete cover and all the concrete cover) was selected for each case.

Figures (5.2-5.7) show the developed interaction diagrams for each case for a given reinforcement ratio ($\rho=1\%$). The strength capacity in these curves is presented in non-dimensional terms for axial load capacity, $K_n$ and moment capacity, $R_n$ given as:

\[
K_n = \frac{P_n}{Agf'_c} \quad (5.1)
\]
\[
R_n = \frac{P_ne}{Ahf'_c} \quad (5.2)
\]

Where: $P_n$ is the nominal applied axial load capacity, kips

$Ag$ is the gross area of the column cross section, in$^2$

$f'_c$ is the concrete compressive strength, ksi

$h$ is the height of the cross-section, in

$e$ is the eccentricity measured from the center of the uncracked section, in
Figure 5.2: Interaction Diagram for Deteriorated Column with Cover Loss at Compression Side, Case I

Figure 5.3: Interaction Diagram for Deteriorated Column with Cover Loss at Tension Side, Case II
Figure 5.4: Interaction Diagram for Deteriorated Column with Cover Loss at Left or Right Side, Case III

Figure 5.5: Interaction Diagram for Deteriorated Column with Cover Loss at All Sides, Case IV
Figure 5.6: Interaction Diagram for Deteriorated Column with Cover Loss at Compression Side and Right or Left Side, Case V

Figure 5.7: Interaction Diagram for Deteriorated Column with Cover Loss at Tension Side and Right or Left Side, Case VI

The discussion for these curves will be presented in Chapter V.
5.2.2 Circular Sections

As the concrete cover loss decrease the area of the column section, the effect of the depth of the concrete cover loss on the strength capacity of the columns that have circular section was also studied-assuming uniform cover cracking around the section. The procedure derived in section (3.4.2) was applied to generate the interaction diagrams by making Excel spreadsheet to predict the effect of the concrete cover loss on the column strength capacity. Three values of concrete cover loss depth \((d_c/3, 2d_c/3 \text{ and } d_c)\) were selected in the analysis. Figure (5.8) shows the properties of the adopted circular cross-section to develop the interaction diagram. The concrete compressive strength \(f'_c\) is 5000psi and the yield strength of steel \(f_y\) is 60 ksi, the diameter of the cross-section is 26in. The concrete cover measured from the center of the nearest steel reinforcement layer \((d_c)\) is 2.6 in and the reinforcement ratio \(\rho\) is (1%).

![Diagram of column cross-section properties](image)

No cover spalling  cover spalling=dc/3  cover spalling=2dc/3  cover spalling=dc

Figure 5.8: Column Cross Section Properties under Study

Figure (5.9) shows the developed interaction diagram for the deteriorated circular column for different values of \((e/h)\) and will be discussed in Chapter V.
5.3 Effect of Reinforcement Ratio on the Strength Reduction of Deteriorated Columns

The steel reinforcement ratio has a significant effect on the strength of the RC columns, concentrically or eccentrically loaded. For eccentrically loaded columns, the amount and distribution of the steel reinforcement affect the failure load of the column for a certain eccentricity value caused by whether the steel reinforcement provides tension or compression resistance. For deteriorated columns, the location and amount of the concrete cover spalling may cause to change the action of the steel reinforcement from tension to compression vice versa.

To assess the effect of the steel reinforcement ratio on the strength capacity of the deteriorated columns, four values of reinforcement ratio, ρ (1%, 2%, 3% and 4%) were selected considering three values of cover spalling amount, \((1/3d_c, 2/3d_c \text{ and } d_c)\) to develop the axial load-moment capacity interaction diagrams for both circular and rectangular cross sections with six cases of concrete cover loss.

Figure 5.9: Interaction Diagram for Deteriorated Circular Column with Different Amount of Cover Spalling
Figure (5.10) shows the interaction diagrams for non-deteriorated column for rectangular section while the effect of the reinforcement ratio value on the strength capacity for deteriorated column for a certain value of cover spalling depth, $d_{cs}$ equals $1/3d_c$, $2/3d_c$ and $d_c$ for the assumed six cases of rectangular cross sections are shown in Figures (5.11-5.19). The properties of the column section shown in Figure (5.1) were used in the analysis, but different steel reinforcement ratio.

Figure 5.10: Effect of Reinforcement Ratio on the Non-Deteriorated Column Strength
Figure 5.11: Effect of Reinforcement Ratio on the Deteriorated Column Strength Case-I-
Figure 5.12: Effect of Reinforcement Ratio on the Deteriorated Column Strength Case-II-
Figure 5.13: Effect of Reinforcement Ratio on the Deteriorated Column Strength Case-III.
Figure 5.14: Effect of Reinforcement Ratio on the Deteriorated Column Strength Case-IV-
Figure 5.15: Effect of Reinforcement Ratio on the Deteriorated Column Strength Case-V-
Figure 5.16: Effect of Reinforcement Ratio on the Deteriorated Column Strength Case-VI-
The circular column section shown in Figure (5.8) was adopted in the analysis to predicate the load carrying capacity of the deteriorated column for four values of reinforcement ratio. Figure (5.17) shows the effect of the reinforcement ratio on the non-deteriorated circular column, while Figure (5.18) shows the deteriorated column interaction diagram considering the effect of the reinforcement ratio for a certain value of concrete cover spalling depth. \( \gamma = 0.8 \)

Where \( \gamma = \frac{(h-2dc)}{h} \)

\( h \) is the diameter of the column section

Figure 5.17: Effect of Reinforcement Ratio on the Non-Deteriorated Circular Column Strength
Figure 5.18: Effect of Reinforcement Ratio on the Strength of Deteriorated Circular Column
5.4 Effect of Concrete Compressive Strength

Columns are structural members designed to carry compressive axial load with or without moments and as the concrete is considered a strong material in compression, the effect of different values of concrete strength on the load carrying capacity of the columns exposed to concrete cover loss was examined through the development of axial load-moment interaction diagrams for rectangular and circular sections.

The concrete stress-strain relationship was investigated by many researchers and different models were proposed by (Hognestad’s 1951, Desayi and Krishnan 1964, Kent and Park 1971, Popovics 1973 and Tsai 1988) to represent the actual stress-strain curve through experimental and theoretical studies.

In this study, the compressive force resulted from the concrete in the loaded section was obtained based on the ACI simplified method originally proposed by (Whitney, 1940) using rectangular stress block and that based on the parabolic stress-strain curve proposed by (Desayi and Krishnan, 1964).

5.4.1 Rectangular Sections

The size and details of the cross-section adopted for this analysis are same as that used for experimental work as shown in Figure (5.19) assuming concrete cover loss at the compression face of the column ( Case I) and two values of the amount of cover loss, $d_{cs}$ equals $(1/2d_c$ and $d_c)$. 
As is illustrated Figure (5.20), the internal force resulted from concrete is calculated using rectangular stress block as:

\[ C_c = 0.85 f'_c b \beta_1 c \]  
(5.3)

And by parabolic stress strain relation as:

\[ C_c = \alpha f'_c bc \]  
(5.4)

The moment resulted from the concrete is calculated using rectangular stress block as:

\[ M_c = 0.85 f'_c b \beta_1 c (d - \frac{\beta_1 c}{2}) \]  
(5.5)

And by parabolic stress strain relation as:

\[ M_c = \alpha f'_c bc (d - \beta c) \]  
(5.6)

Where:

\[ \alpha = \frac{\varepsilon_o}{\varepsilon} \left[ \ln \left( \left( \frac{\varepsilon}{\varepsilon_o} \right)^2 + 1 \right) \right] \]  
(5.7)

\[ \beta = 1 - \frac{2 \left( 1 - \frac{\varepsilon_o}{\varepsilon} \tan^{-1} \left( \frac{\varepsilon_o}{\varepsilon} \right) \right)}{\ln \left( \left( \frac{\varepsilon}{\varepsilon_o} \right)^2 + 1 \right)} \]  
(5.8)

\( \varepsilon \) at the ultimate value was experimentally determined to be approximately:

\[ \varepsilon_u = 0.00421 - \left( \frac{f'_c - 1450}{5800} \right) \times 0.00138 \]  
(5.9)
The axial load-moment capacity interaction diagrams using both equations for the calculation of the concrete stress for different values of concrete compressive strength are shown in Figures (5.21-a,b, and c). The yield strength of the steel reinforcement was assumed (60ksi).
Figure 5.21: Effect of the Concrete Compressive Strength on the Rectangular Column Section, Case-I

a) With no cover loss
b) With cover loss equals (1/2d_c)
c) With cover loss equals (d_c)
5.4.2 Circular Sections

The size and details of the circular section used to investigate the influence of some factors on the performance of the deteriorated RC columns are used in this section also for different reinforcement ratio values (1, 2, 3 and 4%). For the analysis, the circular section is divided into a number of segments of very small thickness, \( t \) as shown in Figure (5.22). The area, strain, stress, force and moment for each segment are calculated and then the total axial load and moment capacity are determined as follows:

\[
A_{ci} = 2t \sqrt{\left(\frac{h}{2}\right)^2 - \left(\frac{h}{2} - \left(i - \frac{1}{2}\right)t\right)^2} \tag{5.10}
\]

\[
\varepsilon_{ci} = \varepsilon_{cu} \left[1 - \left(i - \frac{1}{2}\right)\frac{t}{c}\right] \tag{5.11}
\]

\[
f_{ci} = \frac{2f'_c\varepsilon_{ci}}{\varepsilon_0 \left[1 + \left(\frac{\varepsilon_{ci}}{\varepsilon_0}\right)^2\right]} \tag{5.12}
\]

\[
P_i = f_{ci}A_i \tag{5.13}
\]

\[
M_i = P_i \left[\frac{h}{2} - \left(i - \frac{1}{2}\right)t\right] \tag{5.14}
\]

Where, \( h \) is the diameter of the column section, \( t \) is the thickness of the segment and \( c \) is the depth of the neutral axis.

\( A_{ci}, \varepsilon_{ci}, f_{ci}, P_i, M_i \) are the area, strain, stress, force and moment about the centroid of the cross section for each segment, respectively.

\( \varepsilon_{co} \) is the maximum strain in concrete taken as 0.002.

Cross Section  \( \text{strain} \)  \( \text{stress} \)

Figure 5.22: Analysis of Circular Sections by Segments
The column strength capacity corresponding to different values of e/h using the actual concrete stress-strain curve or the equivalent stress block is shown in Figure (5.23) for columns with and without concrete cover spalling for different reinforcement ratio.
Figure 5.23: Axial Load Capacity vs e/h for Circular Columns with and without Cover Spalling Using Parabolic and ACI Concrete Stress
5.5 Effect of Yield Strength of Steel Reinforcement

To investigate the effect of steel reinforcement grade on the improvement of the strength of the deteriorated columns for both compression and flexural capacity, three values of yield strength (60, 66 and 72) ksi were selected for a certain value of $f_c$ (5700 psi) and different values of $(e/h)$. Figure (5.24-a, b and c) shows the axial load-moment capacity interaction diagrams for non-deteriorated and deteriorated columns.

The elastic-plastic (bilinear) model for steel stress-strain curve was used in the analysis for both compressive and tensile reinforcing bars.
Figure 5.24: Effect of the Concrete Compressive Strength on the Rectangular Column Section, Case-I-
   a) With no cover loss
   b) With cover loss equals (1/2d_c)
   c) With cover loss equals (d_c)
CHAPTER VI

RESULTS AND DISCUSSION

This chapter presents and explains both theoretical and experimental results obtained for the evaluation of the strength of columns subjected to different values of eccentricities and concrete cover spalling.

6.1 Theoretical Analysis Results

The interaction diagrams for the columns with concrete cover spalling were constructed using the procedure explained in sections (3.4.2 and 3.4.2) for rectangular and circular sections, respectively. There are different factors affecting the value of the reduction in strength of the deteriorated columns, some of these factors were considered in the theoretical analysis and presented in Chapter V.

The reduction in strength is obtained either by plotting the eccentricity/section depth (e/h) line where the eccentricity is calculated for specific values of (Pn, Mn) pair for the non-deteriorated column, the intersection points of (e/h) line with the interaction curves of the deteriorated section for each case will represent the values of residual strength, or it can be found by selecting a certain value of axial load or moment as a percentage of the nominal strength and drawing a straight line on the interaction diagram curves, hence the reduction in strength can be calculated through the differences between the intersection points of this line with the interaction curves for each case.
The following sections will present and discuss the results obtained from these interaction diagrams for rectangular and circular sections.

6.1.1 The location of the Concrete Cover Loss
For rectangular sections, the location of the cover spalling affects the percentage values of the reduction in strength according to column failure mode (tension, balance and compression). Therefore, six cases of deteriorated sections were assumed considering different locations of concrete cover spalling. In the following sections, the theoretical analysis results for the column strength capacity for each case and a certain value of reinforcement ratio ($\rho=1\%$) is discussed.

6.1.1.1 Concrete Cover Spalling at the Extreme Compression Column Side, Case I

From the interaction diagram shown in Figure (5.2) for this case, it is observed that the pure axial load capacity decreases to (97.5%, 95.1% and 92.6%) of the original strength for non-deteriorated column corresponding to the values of the amount of concrete spalling ($d_{cs}$) equals to ($\frac{1}{3}d_c$, $\frac{2}{3}d_c$ and $d_c$), respectively, while the pure moment capacity decreases to (96.5%, 93.8% and 91.8%) for $d_{cs}$ equals to ($\frac{1}{3}d_c$, $\frac{2}{3}d_c$ and $d_c$). The reduction in compressive and moment strength in balanced condition is (5%, 10% and 15%) for $d_{cs}$ equals to ($\frac{1}{3}d_c$, $\frac{2}{3}d_c$ and $d_c$), while the reduction in strength at the compression controlled region (i.e. small values of $e/h$) is equal to (4-11.5%) as ($d_{cs}$ increases from $\frac{1}{3}d_c$ to $d_c$) which is less than for that in tension controlled zone (large $e/h$) equal to (4.5-15.4%). In tension controlled zone, the depth of the neutral axis is already small and concrete loss in that region leads to a reduction in compressive force contributed by concrete block more than that in the compression zone especially for ($e/h$) less than 0.1.

6.1.1.2 Concrete Cover Spalling at the Extreme Tension Column Side, Case II

The interaction diagram for this case is shown in Figure (5.3), No reduction in pure moment capacity is noticed for any of amount of cover spalling. For ($d_{cs}$) equals to ($\frac{1}{3}d_c$), the reduction in the axial load strength was found to be (3-5%), while it was found to be
(4-6.5%) of the original strength for \(d_{cs}\) equals to \((2/3d_c)\) and the largest reduction in load capacity was observed to be when \(d_{cs}\) equals to \(d_c\) ranges from (5.7-10%). For the moment capacity, the maximum reduction in strength was obtained to be (3.9%, 7.6% and 10%) as \(d_{cs}\) increases by \((1/3d_c, 2/3d_c \text{ and } d_c)\) for \((e/h=0.4)\). The small percentage of strength reduction is attributed to the essential ignoring of concrete strength in the tension region; hence the loss in that region doesn’t strongly affect the column strength and the reduction in the cross-section height and the shifting of the plastic centroid causes that small reduction in strength.

6.1.1.3 Concrete Cover Spalling at the Left/Right Column Side, Case III

It is observed from Figure (5.4) that shows the interaction diagram for case III that the maximum reduction in strength is (4, 7.3 and 11%) for \(d_{cs}\) equals to \((1/3d_c, 2/3d_c \text{ and } d_c)\) and it increases with the decrease of \((e/h)\) value, i.e. it increases while transiting from tension controlled zone to compression controlled zone. The reason for the more reduction in strength for small eccentricities is the decrease of the cross-section width which leads to the reduction in internal force contributed from the concrete compression block.

6.1.1.4 Concrete Cover Spalling at All Column Sides, Case IV

The axial load-moment capacity interaction diagram for case IV is shown in Figure (5.5). The increase of the amount of concrete cover loss \(d_{cs}\) from \((1/3d_c, 2/3d_c \text{ and } d_c)\) decreases the axial load and flexural capacity by (13, 25 and 34%) for all values of \((e/h)\). The reduction in strength is uniform because of the equal concrete cover loss around the cross-section and the concrete cross section remains symmetric about both axes. The strength reduction in this case is much more than all other cases.
6.1.1.5 Concrete Cover Spalling at All Column Sides, Case V

For this case, it is noted from Figure (5.6) that the reduction in strength increases with the increase of the depth of the concrete cover loss, when \( d_{cs} \) equals to \( 1/3d_c \), the axial load and moment capacity are reduced by (6-7\%) and minimum residual strength is 8.2\% at \( e/h=0.4 \), similarly the strength capacity ranges from 83.3\% to 88.6\% of the original strength when \( d_{cs} \) equals to \( 2/3d_c \) and the most critical case at \( e/h=0.4 \), while for \( d_{cs} \) equals to \( d_c \) the residual strength value ranges from 80\% to 82\% of the original strength.

6.1.1.6 Concrete Cover Spalling at All Column Sides, Case VI

The reduction in strength for Case VI was obtained to be (3.5-8\%) when \( d_{cs} \) equals \( 1/3d_c \), (7-16\%) when \( d_{cs} \) equals \( 2/3d_c \) and the largest reduction in strength is observed when \( d_{cs} \) equals \( d_c \) (9.5-18.5\%). Similar to Case II, the maximum reduction in strength occurs at \( (e/h=0.4) \) and the reduction in both axial load and flexural capacity in the compression controlled zone is more than that in tension controlled zone because of the concrete loss in the left side which decreases the width of the cross-section, consequently decreases the internal compressive force resulted from concrete. The interaction diagram for this case is shown in Figure (5.7).

6.1.1.7 Circular Columns

The interaction diagram for circular columns with \( (\rho=1\%) \) is shown in Figure (5.9). it was assumed that the concrete cover loss occurs around the cross section and uniformly. Therefore, the percentage of the reduction in strength for both axial load and flexural capacity was found to be high (8-13\%) for \( d_{cs} \) equals \( 1/3d_c \), (15-24\%) for \( d_{cs} \) equals \( 2/3d_c \) and (21-34.5\%) when \( d_{cs} \) equals \( d_c \). The smaller reduction in strength was noticed to be for
(e/h = 0.8 to 1) and for e/h less than 0.8, the reduction in strength is approximately the same with a difference of ±2%.

6.1.2 Steel reinforcement Ratio Effect

Since steel bars in columns act as compressive and tensile resistance with a portion depends on the loading condition (e/h value), the effect of the reinforcement ratio (ρ=1% to 4%) on the residual strength of the deteriorated columns was investigated for rectangular and circular columns as shown in figures (5.10 through 5.18). The percentage reduction in axial load and flexural capacity for the columns with concrete cover spalling equals to d_c, which is considered the critical case, for each reinforcement ratio is shown in Bar charts Figures (6.1-6.7).

Figure 6.1: Residual Strength Capacity of Rectangular Columns Case I
a) Axial load  b) Moment
Figure 6.2: Residual Strength Capacity of Rectangular Columns Case II
(a) Axial load  b) Moment

Figure 6.3: Residual Strength Capacity of Rectangular Columns Case III
(a) Axial load  b) Moment

Figure 6.4: Residual Strength Capacity of Rectangular Columns Case IV
(a) Axial load  b) Moment
Figure 6.5: Residual Strength Capacity of Rectangular Columns Case V
   a) Axial load  b) Moment

Figure 6.6: Residual Strength Capacity of Rectangular Columns Case VI
   a) Axial load  b) Moment

Figure 6.7: Residual Strength Capacity of Circular Columns
   a) Axial load  b) Moment
It can be observed that the increase of the reinforcement ratio doesn’t have the same influence on the improvement of the deteriorated column performance. For rectangular section Case I, the increase in reinforcement ratio to (2%, 3% and 4%) causes to increase the residual strength by an average (4.5, 6.3 and 9%), respectively in the tension controlled zone for axial load capacity, while less strength improvement is noticed for small values of e/h to be (2 to 4%). Regarding the flexural strength, the increase of the reinforcement ratio enhances the column strength in tension controlled zone by (7-10%) but less improvement is noticed in compression controlled zone to be (2-5%). For large eccentricities, the gain in strength as $\rho$ increases is attributed to the increase of the steel area in the tension zone which leads to increase the internal force and hence the ultimate load capacity.

For Case II, it is observed that the increase in steel area improves the axial and flexural capacity by a small amount in tension and compression controlled zone by an average value (4-6%), however, the increase of the flexural capacity is found to be slightly more in small eccentricities. For Case III, the increase of the reinforcement ratio causes to improve the strength capacity of the deteriorated column by a small amount ranges from (2-3%). In this case, the reduction in strength is resulted from the decrease of the column section width but no change in the plastic centroid location; hence the location of the neutral axis at a certain value of e/h shifts by a small distance due to the concrete cover loss. For Case IV, when $\rho=2\%$, the reduction in strength decreases by (5-7.7%) in tension controlled region and decreased by less amount (3-5%) in compression controlled region, while for $\rho=3\%$ the strength gain is found to be (6-8.7%) for e/h (1-0.5) and it decreases from (7.4-5.4%) with the decrease of e/h from (0.3 to 0), respectively. The largest
difference in residual strength is observed at e/h =0.3 and 0.5 which is equal to (11 and 12%), respectively for ρ=4%. For Case V, it is noted that the increase of the steel area doesn’t affect the column residual strength by a large amount, which ranges from (82-87%) for ρ=2%, (83.5-87.5%) for ρ=3% and (85.6-90%) for ρ=4%. The strength gain is slightly more for the last case in rectangular column section with the increase of the reinforcement ratio from (1%, to 2%, 3% and 4%). The residual strength for 1% ranges from (81.6%-90.8%), for 2% it ranges from (84.3%-91.8%), while it ranges from (85.1%-92.7%) for ρ=3% and the largest strength gain is found to when ρ=4% at e/h=0.05 in comparison with ρ=1% by a difference 7%.

For circular column section, the effect of increasing the reinforcement ratio- shown in Figure (6.7)- from 1% to 2% leads to increase the residual axial and flexural strength/the strength of original section in tension controlled zone more by a percentage equals (8-12%) for that obtained with ρ=3% and 4%. On the contrary, it is found that as ρ increases to 3% and 4%, the residual axial and flexural strength in the compression controlled zone increases with a difference ranges from 8-11% of that observed when ρ equals 1% and 2%.

6.1.3 Concrete Compressive Strength Effect

The interaction diagrams of columns with or without concrete cover spalling shown in Figure (5.21) reveal that using the parabolic stress-strain curve for concrete rather than ACI simplified method in the calculation of the internal forces increases the theoretical strength capacity for all values of (e/h). It is observed that the $P_{ACI}$ in comparison to $P_{parab}$. has the same trend for all cases with or without cover spalling and the three values of the ultimate compressive strength $f'c$ which is found to be close in case of applying loads with large eccentricities $e/h> 0.5$, e.g. for $f'c$ equals to 6840 psi, $P_{ACI} / P_{parab}$ is (96.7, 98.8 and
99.3%) corresponding to \( d_{cs} \) equals to \((0,1/2d_c \text{ and } d_c)\), respectively at \( e/h=1.0 \), while at \( e/h=0.25 \) it is found to be \((89.9, 89.7 \text{ and } 90.2\%)\). Also, it is noted that \( P_{ACI} / P_{parab} \) for a certain depth of cover spalling and \( e/h \) in the compression controlled zone is larger for the smaller value of \( f'_c \) (e.g., for \( e/h=0.25 \) and \( d_{cs} \) equals \( d_c \), \( P_{ACI} / P_{parab}=90.2, 91.7 \text{ and } 93.3\% \) for \( f'_c =6840, 5700 \text{ and } 4560\text{psi} \)) as shown in Fig.(6.8). Using approximated parameters with reduction factors for concrete stress-strain leads to obtaining less strength capacity other than using parabolic equation.

![Figure 6.8: Comparison of Column Strength with Full Cover Spalling Using ACI and Parabolic Equations](image)

For circular columns, it can be observed from Figure (5.23) that the determination of the column strength capacity by using the actual parabolic concrete stress-strain curve gives higher values than using simplified ACI stress block for all values of reinforcement ratios and for both the cross sections with and without loss. For \( \rho=1\% \), the percentage value of the strength capacity calculated by ACI equation to that calculated by parabolic equation \( P_{ACI}/P_{PARAB.} \) is equal to 94% for small \( e/h <0.4 \), while it increases to be 99% for large \( e/h \). For \( \rho=2\% \) and 3% the percentage value of \( P_{ACI}/P_{PARAB.} \) is equal to 92-93% for small \( e/h <0.4 \), while it increases to be 95-97% for large \( e/h \), while the maximum difference in
strength capacity value was found to be for $\rho=4\%$ as it is equal 93-94\% for all values of $e/h$ in both tension and compression controlled zones.

6.1.4 Steel Bars Yield Strength Effect

Figure (5.24) shows the effect of reinforcement yield strength $f_y$ on the strength capacity of the columns with and without cover spalling, it can be observed that increasing $f_y$ from 60 to 66 ksi and from 66 to 72 ksi doesn’t affect the strength of the non-deteriorated columns in balanced condition and increases the axial capacity by a very small amount up to (3\%) in the compression controlled zone, while with the increase of $f_y$ from 60 to 72ksi increases the axial strength by (12\%) at large values of eccentricities. For columns with cover spalling equals ($1/2d_c$), the increase of $f_y$ from 60 to 66 ksi increases the axial strength up to (8\%) at large values of eccentricities and up to (7\%) when $f_y$ increases from 66 to 72 ksi. The maximum strength gain was found to be in case of columns with all cover spalling that the axial capacity increases up to (15.5\%) in the tension controlled zone if $f_y$ increases from 60 to 72ksi as shown in Fig.(6.9). The reason of improving the performance of the deteriorated columns in case of tension controlled failure when using higher steel grade that the internal force contributed by concrete is small and the cross section strength depends mainly on the steel reinforcement.

![Figure 6.9: Effect of Using Different Steel Grade on the Capacity of Columns with Full Cover Spalling](image_url)
6.2 Experimental Work

To verify the developed theoretical interaction diagram, experimental tests were carried out on (18) column specimens by applying axial compressive load with different eccentricities. Case I was selected where the concrete cover spalling occurs at the compression side of the column in the experiments.

The following sections present the experimental results and discussion.

6.2.1 Failure Mode

All specimens in this study were short columns which fail due to the yielding of steel bars and crushing of concrete. The specimens were classified mainly into two groups: Group-A-of columns have length 30in and small eccentricities and Group-B-of columns have of length 36in and large eccentricities. In Group-A-, specimens with concentrated loaded columns, longitudinal cracks started at the first cracking load, the number of cracks increased, and the specimen began to bulge until the yielding of the steel bars, finally crushing of the concrete cover. The longitudinal cracks started at the compressive side and increased with the increase of the load and the breakout of the concrete occurred on the face with higher compressive stresses until after reaching the ultimate load when the concrete cover breakout occurred on all faces. That failure mode is primarily attributable to the brittle behavior of concrete material which is consistent with observations on different experimental work carried out by others (e.g., Bažant and Kwon 1994; Şener et al. 2004; Němeček and Bittnar 2004; Guo and Bažant 2006; Porras et al. 2014 and Jin et al.2017). The steel bars at the tension side didn’t reach the yielding stress since the column is basically considered under compression. All specimens loaded with small eccentricities experienced the same failure mode for that with no concrete cover spalling and with cover
spalling as shown in typical photographs presented in Figure (6.10-a, b and c) for sets #1, 2 and 3, respectively.

For Group-B- with large eccentricities, a typical failure mode was observed with transverse cracks appearing along the ties on the tension side of the column which became wider and the steel bars in the tension side yielded and caused the failure of the specimens. Also, as for Group-A-, a similar mode of failure was observed for all specimens loaded with large eccentricities regardless of whether the concrete cover spelled or not. Figure (6.11-a, b and c) show the failure modes of set #4, 5 and 6, respectively.
a) Set#1

(b) Set#2

(c) Set#3

Figure 6.10: Failure Mode of Specimens with Small Eccentricities
Figure 6.11: Failure Mode of Specimens with Large Eccentricities
6.2.2 Ultimate Load Carrying Capacity

Table 6.1 shows the experimental ultimate load values of the specimens for all sets which had different eccentricities and amount of concrete cover spalling. The eccentricity for all specimens was measured due to the original section without concrete cover spalling as shown in Figure (6.12).

![Eccentricity Measurements for the Specimens](image)

The axial load-moment capacity interaction diagram for the experimental strength values is shown in Figure (6.13) using the normalized values for the axial load, it can be seen that the column strength capacity decreases with the increase of the concrete cover spalling depth. For specimens in set#2, the axial load capacity decreased by (11.5%-19.5%) compared to specimen in set#1, while the axial load capacity for set#3 decreased by (22%-24.3%) compared to specimen in set#1. The reduction in axial load capacity for specimens in set#5 ranged from (19.4%-21.1%) compared to specimen in set#4, while the axial load capacity reduced by (23%-28.2%) for specimens in set#6 compared to specimen in set#4. It is also noted that the reduction in flexural strength caused by concrete cover spalling found to be in the same manner.
Table 6.1: The Ultimate Load Capacity for the Column Specimens

<table>
<thead>
<tr>
<th>Set No.</th>
<th>Specimens No.</th>
<th>Concrete Cover Spalling Depth, in</th>
<th>Eccentricity in</th>
<th>Ultimate Load Kips</th>
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<tbody>
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<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>240</td>
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<td></td>
<td>2</td>
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<td>0.875</td>
<td>185</td>
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<tr>
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<td>3</td>
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<td>7.0</td>
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</tbody>
</table>

It is observed that the reduction in load capacity due to concrete cover spalling in tension controlled region when the columns loaded with large eccentricities is more than that observed in compression controlled region, columns loaded with small eccentricities, that can be attributed to the decrease in the compressive force resulted from the concrete block and the steel bars in the compression side due to the location of the neutral axis at a certain value of \((e/h)\) while the tensile forces resulted from the steel bars remain with the same value which leads to decrease in the total axial load capacity, while for the small values of eccentricity the effect of the neutral axis location is smaller since the cross section is mainly is in compression.
6.2.3 Comparison between the Theoretical and Experimental Results

Table (6.2) shows the comparison between the values of the ultimate load capacity which observed from the experimental work and that obtained from the theoretical work using the properties of the concrete and steel materials and the properties of cross section for each specimen. It was found that the ratio between the experimental and the theoretical ultimate load capacity of the specimens ranges from (102%-109%) by using ACI simplified method and (89%-107%) for that obtained by parabolic equation, it is indicated that using ACI simplified method in the analysis is considered more acceptable than using the actual parabolic equation which overestimates the strength of some columns. Also, the prediction of strength capacity of columns with concrete cover spalling using the presented theoretical approach is applicable and the obtained strength capacity values are considered to be on the safer side since they are less than that determined from the experimental work.

Figure (6.14) shows the axial load-moment capacity interaction diagram for the theoretical and experimental results in terms of dimensionless factors $K_n$ and $R_n$ for different values of $(e/h)$. 

![Figure 6.13: Non-dimensional Interaction Diagram for the Experimental Specimens](image)
Table 6.2: Comparison Between Experimental and Theoretical Failure Load Results

<table>
<thead>
<tr>
<th>Set No.</th>
<th>Specimens No.</th>
<th>Theoretical Load, ACI Eq., ( P_{ACI} ), kips</th>
<th>Theoretical Parabolic Eq., ( P_{Parab.} ), kips</th>
<th>Experimental Ultimate Load, ( P_{Exp} ), kips</th>
<th>( P_{Exp} ) ( P_{ACI} )</th>
<th>( P_{Exp} ) ( P_{Parab.} )</th>
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<td>107</td>
<td>108</td>
<td>1.07</td>
<td>1.01</td>
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<td>1</td>
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<td>205</td>
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</tr>
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<td>1.0</td>
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<td>20.5</td>
<td>21</td>
<td>1.03</td>
<td>1.02</td>
</tr>
</tbody>
</table>
For the values of the steel bar strain at failure, Table 6.3 shows the average of the experimental values of strain in both compression and tension bars and the corresponding theoretical values calculated based on the assumption that the strain at the extreme compression fiber is equal to 0.003.

It can be observed that the experimental strain values are slightly more than the theoretical since the assumed ultimate strain at the extreme fiber is less than its actual value and hence the predicted column load carrying capacity is less than the experimental value.

Table 6.3: Comparison Between Experimental and Theoretical Steel Strains Results

<table>
<thead>
<tr>
<th>Set No</th>
<th>Spec. No.</th>
<th>Concrete Cover Spalling Depth, in</th>
<th>Eccentricity Ratio e/h</th>
<th>Ultimate Strain</th>
<th></th>
<th></th>
<th></th>
<th></th>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Theoretical</td>
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<td>Experimental</td>
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</tr>
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<td>0</td>
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<td>0.00292</td>
<td>0.00268</td>
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<tr>
<td></td>
<td>2</td>
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<td>0.125</td>
<td>0.00261</td>
<td>0.00052</td>
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<tr>
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<td>0.00048</td>
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<tr>
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<td>0.125</td>
<td>0.00261</td>
<td>0.00052</td>
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<td>2</td>
<td>0.75</td>
<td>0.785</td>
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<td>0.00390</td>
<td>0.03700</td>
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</tbody>
</table>
6.2.4. Load-Deflection Response

The lateral deflection at the column mid-height measured during the test corresponding to the applied eccentric load is shown in Figures (6.15) and (6.16) for the specimens with half cover spalling and for all cover spalling, respectively. It is observed that the load-deflection curves evolve in a similar way for all specimens. It is obvious that the columns with small eccentricities (0.875 and 1.75in) in (set#2 and #3) experienced small lateral deflection and large axial load capacity with brittle failure pattern, while the specimens in set#5 and #6 loaded with large eccentricities (3.5, 5.5 and 7in), the axial load capacity decreased and the lateral displacement becomes larger with the increase of the eccentricity and the failure mode transit from brittle to ductile. Therefore, the ductility capacity improves when the lateral displacement becomes larger and larger, as observed from the post-peak softening curves (Jin et al. 2017). This is definitely attributable to the
brittle-ductile transition behavior of the failure pattern or fracture mechanism (Porras et al. 2014).

Figure 6.15: Load vs. Mid Span Deflection for Columns with Half Cover Spalling

Figure 6.16: Load vs. Mid Span Deflection for Columns with All Cover Spalling
6.2.5 Ultimate Load Carrying Capacity of Corroded Specimens

Table 6.4 shows the experimental values of the strength capacity for the corroded specimens exposed to 28% corrosion amount. It is observed that with the increase of the eccentricity value, the strength capacity decreases. Upon a comparison of the ultimate load capacity of the corroded specimens with the theoretical ultimate load capacity of the uncorroded specimens, it is noted that the percentage of the reduction in capacity decreases with the increase of the eccentricity value. It can be attributed that the corrosion occurred on the compression bars that caused the concrete cover cracking in addition to the corrosion of the stirrups which lead to a reduction of confinement, so when the eccentricity increases the concrete effect on the column strength will decrease.

Table 6.4: Strength Capacity of the Corroded Specimens

<table>
<thead>
<tr>
<th>Specimens No.</th>
<th>Eccentricity in</th>
<th>Experimental Ultimate Load, Corroded, Kips</th>
<th>Theoretical Ultimate Load, Uncorroded, Kips</th>
<th>Reduction in Capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>185</td>
<td>273</td>
<td>32.3</td>
</tr>
<tr>
<td>2</td>
<td>0.875</td>
<td>165</td>
<td>202.8</td>
<td>18.7</td>
</tr>
<tr>
<td>3</td>
<td>1.75</td>
<td>125</td>
<td>143.4</td>
<td>12.8</td>
</tr>
</tbody>
</table>

Regarding the failure mode, it was observed that the specimens failed in a brittle manner as the applied load was with small eccentricity beside the corrosion of the bars at the compression side which causes a brittle failure also, Figure (6.17) shows the specimens at failure. The displacement measured at the mid-height of the column showed that with the increase of the eccentricity value the displacement corresponding the same load value increases and the ultimate displacement for larger eccentricity increased under less failure load as shown in Figure (6.18).
Figure 6.17: Failure Mode of Corroded Specimens

Figure 6.18: Load vs. Mid Span Deflection for Corroded Columns
6.2.5 Comparison with Previous Studies

To verify the predicted strength of the deteriorated columns in this study, a comparison was made with theoretical and experimental studies presented by other researchers. Tapan and Aboutaha, 2011 presented a theoretical study to evaluate the strength of columns exposed to corrosion for rectangular sections assuming six different corrosion deterioration cases with different corrosion amounts, cover to depth (C/D) ratios, and exposed bar lengths. The details of the cross-section and materials properties for the corroded bars used in the analysis are shown in Figure (6.19). The comparison was made for three cases according to the location of the corroded bars with a corrosion amount 10% in which the column reaches the fourth stage of deterioration which leads to full spalling of the concrete cover.

The different corrosion cases are Case I where the corrosion is at the extreme compression layer of bars; Case – II: Corrosion at the extreme tension layer of bars; and Case – III: Corrosion at the extreme left or right-side bars. For each case, the axial load-moment interaction curve was developed and shown with the compared curve in the same figure, Figures (6.20-6.22).
Figure 6.20: Comparison of the Column Interaction Diagram, Case I

Figure 6.21: Comparison of the Column Interaction Diagram, Case II

Figure 6.22: Comparison of the Column Interaction Diagram, Case III
It can be observed that the column interaction diagrams developed by the proposed analysis perfectly agree with that developed by Tapan and Aboutaha, 2011 for all cases taking into account that the effect of corrosion on the steel reinforcement properties was considered. However, the proposed model is considered identical to the previous model. To ensure the accuracy of the present model, the predicted values of the deteriorated columns should be verified with the available experimental results. Rodriguez et al 1996 carried out tests on 24 short columns exposed to accelerated corrosion; the specimens were divided into three series with the same rectangular cross-section (200x200mm), 7.87x7.87 in but different steel reinforcement area. The proposed theoretical analysis has been applied to the column in series 2 which cross-section is shown in Figure (6.23-a) with the material properties. Although these columns are axially loaded, nominal moments are present due to a combination of the non-uniformity of the corrosion, imperfections in the casting and testing regime and, at later stages spalling (Webster, 2000). The corrosion rate for this series ranges from 9 to 15%. However, the only corrosion consequence on the performance of the column included in the analysis is the concrete cover spalling. The column interaction diagram developed for series#2 using the present approach to evaluate the column strength capacity in addition to the strength values obtained from the test by Rodriguez et al. is shown in Figure (6.23-b).

The experimental axial load at failure and the corresponding eccentricities calculated by Rodriguez et al. from displacement readings taken on all four faces at mid-height are given in Table 6.5.
Figure 6.23: Comparison of Predicted P-M Interaction Diagram with (Rodriguez et al 1996) Experimental Results

Figure (6.23-b) reveals that the predicted strength values of the deteriorated column by proposed analytical model are in a good agreement with the test results in which their values are found to be outside the failure line for columns 23 and 25, while the other columns lies on the failure line, the comparison indicates that the evaluation of the column strength based on the present model and by considering only the concrete cover spalling induced corrosion gives reasonable results and doesn’t overestimate the column strength.
Table 6.5: Ultimate Axial Load and Corresponding Eccentricities for Columns Tested by Rodriguez et al.

<table>
<thead>
<tr>
<th>Column No.</th>
<th>P (kN)</th>
<th>$e_x$ (mm)</th>
<th>$e_y$ (mm)</th>
<th>$M_x$ (kN.m)</th>
<th>$M_y$ (kN.m)</th>
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<td>1040</td>
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<td>15.6</td>
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<td>1091</td>
<td>13.8</td>
<td>16.4</td>
<td>15.1</td>
<td>17.9</td>
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<tr>
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<td>1135</td>
<td>1.4</td>
<td>7.4</td>
<td>1.6</td>
<td>8.4</td>
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</table>

Another comparison has been made with the experimental data provided by Wang and Liang, 2008 for the strength capacity for eccentrically loaded columns exposed to accelerated corrosion. The section size is 200x200mm reinforced by 4 $\phi$18 steel bars. The details of the specimens are shown in Table (6.6). The average compressive strength of concrete is 40MPa, and the steel yield strength is 397.5MPa.

The variables considered in the analysis using the proposed theoretical study are the depth of the concrete cover spalling and the loss in the area of the steel reinforcement. The reduced cross-sectional area is calculated as:

$$A'_s = A_s(1-\alpha)^2$$  \hspace{1cm} (6.1)

$$\alpha = 2P_r T / d$$  \hspace{1cm} (6.2)

Where, $A_s$ is the original cross-sectional area of the bar and $\alpha$ defined as the metal loss factor. The percentage weight loss $\rho$ can be shown to be equal to $(2\alpha)$ times 100.

In other words, the ratio of weight loss to the original weight of a bar equals $2\alpha$ or twice the metal loss factor (Azad et al. 2007). So, based on the given weight loss, the reduced
steel reinforcement area can be found. Figure (6.24) shows the comparison of the predicted strength values and the corresponding experimental values. Also, the column strength was predicted ignoring the effect of the reduction of steel area to investigate the importance of considering these variable in the analysis, and the values have been compared with the experimental results as shown in Figure (6.25).

Table 6.6: Details of Columns Tested by Wang and Liang, 2008

<table>
<thead>
<tr>
<th>Column No.</th>
<th>Cross section (mm)</th>
<th>Average depth of concrete cover (mm)</th>
<th>e (mm)</th>
<th>Average weight loss of steel %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Width</td>
<td>Height</td>
<td>Tensile</td>
<td>Compressive</td>
</tr>
<tr>
<td>1</td>
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<td>200</td>
<td>29</td>
<td>34</td>
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</table>

It is observed from both figures that the predicted strength values are smaller than that obtained from the experiments for the specimens with small eccentricities (e/h=0.25), while the predicted values are slightly more or equal to the corresponding experimental
values for the specimens with large eccentricities (e/h=0.75), that may be attributed to the neglecting of the effect of the corrosion on the reinforcement yield strength. However, the theoretical strength value of the reference specimen with no corrosion is also found to be more than the experiential. For the analysis made without considering the weight loss of steel bars, it is noted that the percentage value of the column strength taking into account the reduced area of the steel bars ranges from 97% to 100% in relation to that with no reduction in reinforcement area considered in the analysis.

Figure 6.24: Comparison of Predicted Strength Values with Wang and Liang, 2008 Experimental Results Considering Weight Loss of Steel Bars
Figure 6.25: Comparison of Predicted Strength Values with Wang and Liang, 2008
Experimental Results Ignoring Weight Loss of Steel Bars
CHAPTER VII

FINITE ELEMENT MODELLING OF REINFORCED CONCRETE SPECIMENS

7.1 Introduction

Finite element analysis has been widely used as a powerful tool for design engineers for helping them to reduce time and cost for the analysis and design of structures. Structural members can be modeled and simulated using different finite element software packages to study their behavior and performance under different loading conditions using linear and nonlinear static or dynamic stress analysis. Also, the available limited experimental results for certain tests can be proved and improved and then can be generalized to represent the different possible cases through the implementation of the finite element analysis software.

In this study, the tested columns were analyzed using ABAQUS software package through three-dimensional analysis under static loading to investigate the ultimate load capacity and the induced stresses and strains of concrete and steel.

When the structural elements are modeled properly with the application of the actual testing conditions, detailed visualization of the deformation of the structure, crack initiation, distribution of stresses and strains will be shown accurately.
7.2 Finite Element Modeling

Modeling the reinforced concrete structures in ABAQUS is complicated since the reinforced concrete is a composite material with two different constitutive in terms of their properties and behavior. There are four methods for incorporating the reinforcing bars into finite element model. In these methods, the nodes of the rebar elements are embedded and constrained within the nodes of the concrete host element. The concrete element is defined as solid or continuum elements. When dealing with solid host elements rebar elements may be defined as a beam, shell, membrane, surface, or solid elements (SIMULA, 2013).

There are three representations of the reinforcement in the FE model described as follows:

7.2.1 Discrete Model

The reinforcement in this model is modeled as a one-dimensional truss element connected to the nodes of the concrete mesh. In this model, the reinforcement displacement with respect to the surrounding concrete can be accounted. To consider the effect of bond, special elements must be placed at the interface between the concrete and steel because the concrete and steel are two totally independent parts (Li, 2007). The location of the reinforcement restricts the mesh patterns and hence increases the number of concrete elements and degree of freedom (Khalfallah and Ouchenane, 2007). In this model, concrete and reinforcing bar share common nodes at the interface as shown in Figure 7.1.
7.2.2 Smeared Model

In this model, the reinforcement is assumed to be distributed uniformly in the concrete element and the reinforced concrete material is considered as a homogeneous material. The constitutive relationship can be evaluated based on the composite theory by adding the properties of concrete and steel together (Chang, 1987). This model works well for distributed reinforcements, which is typically the case of slab reinforcements (Frérot, 2015).

7.2.3 Embedded Model

In this model, the reinforcement can be modeled using different element types as embedded elements in the host elements. The displacement of reinforcement elements will be compatible with the displacement of surrounding concrete element. The advantage of this model is that the reinforcement can be represented regardless its location or distribution (Chang, 1987). A perfect bond between concrete and steel can be assumed in this model because the degrees of freedom of the reinforcement nodes are eliminated to be the same as the concrete nodes (Li, 2007). Figure 7.2 shows the embedded model representation.
7.3 Modeling of Reinforced Concrete Column Specimen in ABAQUS

Reinforced concrete column specimens used for the experimental work with cross-sectional dimensions 7 inches x 7 inches and 4#4 reinforcing bars were modeled and analyzed in ABAQUS (6.13). The eighteen specimens were modeled with the details as used in the experiments according to their length, concrete cover spalling and the loading conditions. The steel plate welded to the reinforcement and the steel brackets were also modeled. The model consists of three parts for specimens with small eccentricity and four parts for those with large eccentricity. The parts are concrete, reinforcing bars, steel plate and the steel brackets as shown in Figure 7.3-a and b. All parts were modeled as 3D-solid elements using 8 node brick elements C3D8 shown in Figure 7.4 with degrees of freedom in x, y and z directions at each node to predict the failure load, deformation and the stresses and strains in both concrete and steel.
Figure 7.3 ABAQUS Column Model with Full Concrete Cover Spalling

a) Columns with small eccentricity
b) Columns with large eccentricity
7.4 Material Properties

For the analysis of the specimens in this study, the material behavior other than elastic properties is needed to be defined. Three materials types for concrete, steel and rigid steel were created.

7.4.1 Concrete

Since concrete is a brittle material, its behavior in compression and in tension is different. Abaqus provides crack models to simulate the damage in the concrete elements using smeared crack concrete model, brittle cracking model or concrete damaged plasticity model. The concrete damaged plasticity model was selected in the present analysis assumes that the two main failure mechanisms are tensile cracking and compressive crushing of the concrete material. A typical stress-strain curve for concrete is shown in Figure 7.5.
The concrete compressive strength was considered to be 5000 psi, with the modulus of elasticity as 4031 ksi. The Poisson ratio is 0.2 and the five plastic damage parameters which include dilation angle ($\varphi$), flow potential eccentricity ($\varepsilon$), the ratio of initial equibiaxial compressive yield stress to the initial uniaxial compressive yield stress ($\sigma_{00}/\sigma_{0c}$), the ratio of second stress invariant on the tensile meridian to that on the compressive meridian ($K$) and the viscosity parameter ($\mu$) are recommended by Abaqus documentation for defining concrete material were set to 30°, 0.1, 1.16, 0.66 and 0.0 respectively.

The concrete in tension was modeled using a linear elastic approach until cracking is initiated at tensile strength. After crack initiation, the softening will occur and the post tension behavior for direct straining is modeled with tension stiffening. This behavior was characterized by stress-strain response curve as shown in Figure 7.6.
It is generally accepted that Hillerborg's (1976) fracture energy concept, is adequate for many practical purposes. In this approach, the concrete brittle behavior is characterized by a stress-displacement response rather than a stress-strain response. In this case, the fracture energy $G_f$ associated with the failure tensile stress $f_t$, are specified directly in the material property assumes a linear strength loss after cracking as shown in Fig.7.7. Typical values of $G_f$ range from 40 N/m (0.22 lb/in) for a concrete with a compressive strength of 20 MPa, 2850 lb/in$^2$ to 120 N/m (0.67 lb/in) for a high-strength concrete with a compressive strength of approximately 40 MPa, 5700 lb/in$^2$ (SIMULIA, 2013).
A linear stress-strain relationship up to 0.5 ultimate stress, $\sigma_{cu}$ is used in compression, the stress in the descending part was calculated using the numerical model by Hsu and Hsu (1994), Wahalathantri, et al. (2011) given by:

$$\sigma_c = \left[ \frac{\beta(\varepsilon_c/\varepsilon_o)}{\beta-1+(\varepsilon_c/\varepsilon_o)\beta} \right] \sigma_{cu} \text{ ksi} \quad (7.1)$$

Where:

$$\beta = \frac{1}{1-[\sigma_{cu}/(\varepsilon_o\varepsilon_o)]} \quad (7.2)$$

$$\varepsilon_o = 8.9 \times 10^{-5} \sigma_{cu} + 2.114 \times 10^{-3} \quad (7.3)$$

$$E_o = 1.2431 \times 10^2 \sigma_{cu} + 3.28312 \times 10^3 \text{ ksi} \quad (7.4)$$

$\varepsilon_o$, $E_o$ is the maximum strain and initial tangential modulus of elasticity, respectively.

Figure 7.8: Compressive Stress-Strain Relationship for ABAQUS, Wahalathantri, et al. (2011)

$\varepsilon_d$ strain is iteratively calculated using Eq.(7.1) when $\sigma_c = 0.8 \sigma_{cu}$.

Table 7.1: The Input Values for Compression Properties in ABAQUS.

<table>
<thead>
<tr>
<th>Stress, ksi</th>
<th>Inelastic Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 $\sigma_{cu}$</td>
<td>0</td>
</tr>
<tr>
<td>$\sigma_{cu}$</td>
<td>0.002171</td>
</tr>
<tr>
<td>0.8 $\sigma_{cu}$</td>
<td>0.00448</td>
</tr>
</tbody>
</table>
7.4.2 Steel Reinforcement

The model used to simulate the steel reinforcement was the classical metal-perfectly plastic model. The input for steel model includes elastic modulus, Poisson’s ratio and yield stress with values 29,000 ksi, 0.3 and 60 ksi respectively.

7.4.2 Steel Plates

For steel plates and brackets, the material was created as a rigid material of elastic properties.

7.5 Mesh and Convergence

All the modeled parts including concrete, reinforcement bars and steel plates were merged in the assembly module. Mesh convergence was performed to the combined part to give more accurate results with reasonable time for analysis, mesh size of 0.5 inches was used for analysis as shown in Figure 7.9.

(a) Columns with small eccentricity  
(b) Columns with large eccentricity

Figure 7.9: Mesh of ABAQUS Column Model with Full Concrete Cover Spalling
7.6 Boundary Conditions

Boundary condition represents structural support values of displacements and rotation variables at appropriate nodes. For the top end of the column the boundary condition represents the load with its specific eccentricity which was applied in the form of displacement same as in the experiments, so it is restrained in x and y-direction and the load is applied in the z-direction, while for the bottom end, the boundary condition represents the support which was restrained in x, y and z-direction. Figure 7.10 shows the column model with applied boundary conditions.

Figure 7.10: Boundary Conditions of Column Model with Full Concrete Cover Spalling
  a) Column with eccentricity=0
  b) Columns with eccentricity=5.5 in
7.7 Comparison of Experimentation with ABAQUS Column Model

Static analysis was performed to predict the ultimate failure load, stress and strains in the reinforcing bars, deflection and crack patterns. Figures 7.11 shows the comparison between the ultimate failure axial load obtained from numerical analysis in ABAQUS with that obtained from the experiments versus e/h for all sets. It can be observed that the numerical values for the failure axial load are in very good agreement with the experimental failure load values.

Also, a comparison was made for the displacement at column mid-height for the specimens with large eccentricities (e/h>0.4) that obtained from the tests and from ABAQUS for the sets with half and full concrete cover spalling shown in Figure 7.12-a and b. The load-displacement curves show that the maximum displacement values measured during the tests are compatible with those extracted from the ABAQUS model. Figures 7.13-a and b show the visualization of the deflected shape for the ABAQUS model with full concrete cover spalling subjected to small and large eccentricity.

Finally, Figures 7.14-a and b show a visualization of the strain in the direction of the applied load for the model which represents the crack pattern in the columns. it can be observed that the cracks are transverse for the model loaded with large eccentricity which means that the mode of failure is ductile, while the cracks are longitudinal for the model loaded with a small eccentricity which indicates that the mode failure is brittle.

The other cases modeled in ABAQUS with their details are shown in the appendix A.
Figure 7.11: The Failure Axial Load Obtained From Experiments and ABAQUS
Figure 7.12: Load-Deflection Curve Obtained from Experiments and ABAQUS
   a) Columns with Half Concrete Cover Spalling
   b) Columns with Full Concrete Cover Spalling

Figure 7.13: Deflected Shape of Column Model with Full Concrete Cover Spalling
   a) Column with eccentricity=0
   b) Columns with eccentricity=7 in
Figure 7.14: Crack Pattern for Column Model with Full Concrete Cover Spalling
a) Column with eccentricity=0
b) Columns with eccentricity=7 in
CHAPTER VIII
DATA ANALYSIS

8.1 Introduction

This chapter presents the predictive analysis for the available data from the theoretical approach to generalize a formula to estimate the failure load for columns with concrete cover spalling. STATISTICA software was used for this purpose which is a flexible analysis software that helps to create innovative models and enables to predict functions as described in the following sections.

8.2 Nonlinear Function Estimation

There are different statistics models available in STATISTICA used to estimate a function for the given data. However, there is no function that can fit directly the available data, so the functions were predicted by nonlinear estimation with user-specified regression that a certain function was given and check its validity based on Gauss Newton method which solves the parameters until the iteration converges (TIBCO STATISTICA, 2015). This model was used to estimate a formula for the failure load for deteriorated columns with both rectangular and circular sections included all the assumed six cases explained in chapter III.
8.3 Prediction of the Eccentric Load at Failure

All the data related to the size of the cross section, material properties, concrete cover spalling depth and loading condition were exported to STATISTICA spreadsheet for analysis and function estimation. It was observed through several trials that there is no general formula can be predicted to express the failure load for all values of applied eccentricities, hence the available data were classified based on the applied eccentricity into two categories, one for the columns section characterized as compression controlled and the other for the transit and tension controlled.

8.3.1 Failure Eccentric Load in Compression Controlled Zone

The failure load of the column sections characterized as compression controlled when the eccentricity ratio \(e/h\) is less than 0.4 was estimated as:

\[
P_f = P_o \left[ a_1 e^{a_2 \left( \frac{e}{h} \right)} - a_3 \left( 1 - \frac{d_{cs}}{d_c} \right) + a_4 \frac{A_g}{A_{st}} \right]
\]  

(8.1)

The parameter values are as given in Table 8.1.

Where:

- \(P_f\) is the ultimate failure load of the column, lb
- \(P_o\) is the nominal axial load capacity of the original cross section at \(e=0\), kips
- \(e/h\) is the eccentricity ratio noting that \(h\) is the depth of the non-deteriorated cross-section.
- \(d_{cs}, d_c\) is the depth of the concrete cover spalling and the concrete cover, respectively.
- \(A_g, A_{st}\) is the gross area of the cross-section and the area of reinforcement, respectively.
Table 8.1: Summary of Analysis for e/h< 0.4

<table>
<thead>
<tr>
<th>Location</th>
<th>a₁</th>
<th>a₂</th>
<th>a₃</th>
<th>a₄</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Rectangular Sections</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression Side</td>
<td>954</td>
<td>-2.47</td>
<td>-55.3</td>
<td>-0.27</td>
<td>0.999</td>
</tr>
<tr>
<td>Tension Side</td>
<td>917</td>
<td>-2.61</td>
<td>-51.1</td>
<td>-1.05</td>
<td>0.996</td>
</tr>
<tr>
<td>Left Side</td>
<td>930</td>
<td>-2.39</td>
<td>-70.0</td>
<td>-0.31</td>
<td>0.999</td>
</tr>
<tr>
<td>All Sides</td>
<td>775</td>
<td>-2.58</td>
<td>-207.3</td>
<td>-0.77</td>
<td>0.996</td>
</tr>
<tr>
<td>Compression and Left Side</td>
<td>884</td>
<td>-2.51</td>
<td>-118.4</td>
<td>-0.47</td>
<td>0.999</td>
</tr>
<tr>
<td>Tension Side and Left Side</td>
<td>890</td>
<td>-2.44</td>
<td>-109.7</td>
<td>-0.51</td>
<td>0.998</td>
</tr>
<tr>
<td><strong>Circular Sections</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>All Sides</td>
<td>779</td>
<td>-3.26</td>
<td>-202.3</td>
<td>-0.56</td>
<td>0.997</td>
</tr>
</tbody>
</table>

Figures 8.1 through 8.4 illustrate the observed versus predicted values and the normal probability plot of residuals for Case I of the rectangular cross-section where the concrete cover spalling takes place at the compression side and for the circular cross-section. The plots for the other cases are shown in the Appendix B.
Figure 8.2: Normal Probability of Residuals for Case I

Figure 8.3: Predicted vs Observed Values for Circular Sections
The plots show that the relationship between the observed and the predicted values in addition to the normal probability of residuals is approximately linear which indicates that the quality of the regression is very good, and the failure load formula is well expressed. It can be observed from Table 8.1 that the coefficient of correlation $R$ is close to 1, also $R^2$ for all cases is more than 0.97 which means that 97% of the variables are related.

8.3.2 Failure Eccentric Load in Transition and Tension Zones

The failure load of the column sections characterized as other than compression controlled when the eccentricity ratio $(e/h)$ is equal or larger than 0.4 was estimated as:

$$ P_f = P_o \left[ a_1 e^{a_2 \left( \frac{e}{h} \right)} - a_3 \left( 1 - \frac{d_{cs}}{d_c} \frac{e}{h} \left( \frac{A_{st}}{A_g} \right)^{a_5} \right) + a_4 \frac{A_g}{A_{st}} \right] \quad (8.2) $$

The parameters estimates are shown in Table 8.2.
Table 8.2: Summary of Analysis for $e/h \geq 0.4$

<table>
<thead>
<tr>
<th>Concrete cover spalling location</th>
<th>a1</th>
<th>a2</th>
<th>a3</th>
<th>a4</th>
<th>a5</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression Side</td>
<td>800</td>
<td>-2.68</td>
<td>-167</td>
<td>-1.40</td>
<td>0.51</td>
<td>0.997</td>
</tr>
<tr>
<td>Tension Side**</td>
<td>828</td>
<td>-2.44</td>
<td>-15</td>
<td>0.50</td>
<td>-51.6</td>
<td>0.997</td>
</tr>
<tr>
<td>Left Side</td>
<td>766</td>
<td>-2.45</td>
<td>-148</td>
<td>-1.27</td>
<td>0.62</td>
<td>0.997</td>
</tr>
<tr>
<td>All Sides</td>
<td>599</td>
<td>-2.70</td>
<td>-197</td>
<td>-1.40</td>
<td>0.3</td>
<td>0.989</td>
</tr>
<tr>
<td>Compression and Left Side</td>
<td>689</td>
<td>-2.51</td>
<td>-170</td>
<td>-1.37</td>
<td>0.38</td>
<td>0.995</td>
</tr>
<tr>
<td>Tension Side and Left Side</td>
<td>724</td>
<td>-2.52</td>
<td>-164</td>
<td>-1.35</td>
<td>0.49</td>
<td>0.996</td>
</tr>
<tr>
<td>Circular Sections</td>
<td>639</td>
<td>-2.98</td>
<td>-150</td>
<td>-0.91</td>
<td>0.24</td>
<td>0.989</td>
</tr>
</tbody>
</table>

**For this region there is an exception for the equation which was estimated as:

$$P_f = P_o \left[ a_1 e^{a_2 \left( \frac{e}{h} \right)} - a_3 \left( 1 - \frac{d_{cs}}{d_c} \right) + a_4 \frac{A_g (1-A_{st})}{A_{st}} \right] \quad (8.3)$$

The units of $P_f$ and $P_o$ are in lb and kip, respectively.

Figures 8.5 through 8.8 illustrate the observed versus predicted values and the normal probability plots of residuals for Case I of the rectangular cross-section and for the circular cross-section for $e/h \geq 0.4$. The plots for the other cases are shown in the Appendix B.

Similarly, as in the previous case, it is noted the plots show a linear relationship between the observed and the predicted values and for the normal probability of residuals which declares that the failure load formula is well expressed. It can be observed from Table 8.2 that the coefficient of correlation $R$ is close to 1, also $R^2$ for all cases is more than 0.97 which means that 97% of the variables are related.
Figure 8.5: Predicted vs Observed Values for Case I

Figure 8.6: Normal Probability of Residuals for Case I
Figure 8.7: Predicted vs Observed Values for Circular Sections

Figure 8.8: Normal Probability of Residuals for Circular Sections
CHAPTER IX

CONCLUSIONS

Based on the theoretical and experimental work developed in this research, the following conclusions can be drawn:

1. The developed interaction diagrams for the deteriorated columns demonstrated that the axial load and flexural strength capacity of the columns decrease with the increase in the depth of concrete cover spalling by various proportions due to the location of the cover loss and the reinforcement ratio.

2. For rectangular column sections, it was found that among the three cases out of six for the deterioration assumed in the theoretical study for one side of column cover loss, the most critical case with the largest reduction in strength is Case I where the cover loss is at the compression side especially for (e/h greater than 0.2), therefore, it had been selected as a case study for the experimental work.

3. As the applied eccentricity increases, the strength of the deteriorated columns decreases for all cases and for any amount of concrete cover spalling.

4. The increase in the amount of steel reinforcement area enhances the strength of the deteriorated columns by various proportions depending on the location of the cover loss and loading conditions. For circular columns, it was found that the increase of $\rho$ to 3% and 4% increase the residual strength for large values of eccentricities, while using less amount of reinforcement increases the residual strength for small eccentricities.
5. The predicted values of the strength capacity by using the actual parabolic stress-strain curve for concrete in the analysis are larger than that obtained by using the simplified method of ACI in the compression controlled zone, while it has no effect on the strength in the tension controlled zone especially for e/h greater than 0.7. However, using the parabolic stress-strain relationship can overestimate the column strength with very small eccentricities e/h less than 0.2 for both rectangular and circular sections.

6. Using steel bars of yield strength 72 ksi significantly increases the residual strength in deteriorated columns in tension controlled zone compared to the yield strength 60 ksi, but it has less influence on the columns loaded with small eccentricities.

7. The experimental strength results obtained for the specimens are in very good agreement with the predicted theoretical results and all the experimental values are larger than the corresponding theoretical values which verify the applicability of the proposed model.

8. It was observed that the specimens subjected to axial load with small eccentricities experience brittle failure pattern and small values of lateral displacement, while the ductility capacity improved for the specimens subjected to axial load with large eccentricities and those columns fail in a ductile pattern, but the strength capacity is reduced: the larger the loading eccentricity, the greater the lateral displacement.

9. A comparison has been made between the predicted values using the present model and that calculated or obtained from the previous theoretical and experimental work by others and excellent agreement was found which proves the accuracy of the present model.
10. For the failure load of the corroded specimens, it was found that with the corrosion amount of 28%, the reduction in strength is 32.3, 18.7 and 12.8% of the original strength for e/h = 0.0, 0.125 and 0.25, respectively.

11. A numerical analysis was made by using ABAQUS model for the specimens tested in the laboratory to verify the theoretical and experimental results and it was found that the failure load and the column mid-height displacement values obtained from ABAQUS are matching with the experimental values, the strains at reinforcement and concrete at failure were found to be more than those calculated from the theoretical analysis which is based on the assumption of the concrete strain at extreme compression fiber is 0.003.

12. The predicted equation based on the data analysis using nonlinear analysis in STATISTICA represents the failure load of the deteriorated columns very well for all cases that were studied with a coefficient of determination more than 97. One equation was developed for the prediction of the failure load with e/h less than 0.4 and another one for e/h equal or greater than 0.4.
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APPENDIX A.

ABAQUS RESULTS
Figure A.1: Boundary Condition for Column with Half Cover Spalling, $e/h=0.5$

Figure A.2: Boundary Condition for Column without Cover Spalling, $e/h=1.0$
Figure A.3: Strain in Z-direction for Column without Cover Spalling, e/h=0.0

Figure A.4: Strain in Z-direction for Column without Cover Spalling, e/h=0.125
Figure A.5: Strain in Z-direction for Column without Cover Spalling, e/h=0.25

Figure A.6: Strain in Z-direction for Column without Cover Spalling, e/h=0.5
Figure A.7: Strain in Z-direction for Column without Cover Spalling, e/h=0.785

Figure A.8: Strain in Z-direction for Column without Cover Spalling, e/h=1.0
Figure A.9: Deflected Shape for Column without Cover Spalling, e/h=0.0

Figure A.10: Deflected Shape for Column without Cover Spalling, e/h=1.0
Figure A.11: Strain in Z-direction for Column with Half Cover Spalling, e/h=0.0

Figure A.12: Strain in Z-direction for Column with Half Cover Spalling, e/h=0.25
Figure A.13: Strain in Z-direction for Column with Half Cover Spalling, e/h=0.785

Figure A.14: Strain in Z-direction for Column with Half Cover Spalling, e/h=0.785
Figure A.15: Deflected Shape for Column with Half Cover Spalling, e/h=0

Figure A.16: Deflected Shape for Column with Half Cover Spalling, e/h=1.0
Figure A.17: Strain in Z-direction for Column with Full Cover Spalling, $e/h=0.125$

Figure A.18: Strain in Z-direction for Column with Full Cover Spalling, $e/h=0.25$
Figure A.19: Strain in Z-direction for Column with Full Cover Spalling, e/h=0.5

Figure A.20: Strain in Z-direction for Column with Full Cover Spalling, e/h=0.785
Figure A.21: Deflected Shape for Column with Full Cover Spalling, $e/h=0.125$

Figure A.22: Deflected Shape for Column with Full Cover Spalling, $e/h=0.25$
Figure A.23: Deflected Shape for Column with Full Cover Spalling, e/h=0.5

Figure A.24: Deflected Shape for Column with Full Cover Spalling, e/h=0.785
Figure A.25: Stresses in Reinforcement and Concrete for Column without Cover Spalling, $e/h=0.125$

Figure A.26: Stresses in Reinforcement and Concrete for Column without Cover Spalling, $e/h=1.0$
Figure A.27: Stresses in Reinforcement and Concrete for Column with Half Cover Spalling, $e/h=0$

Figure A.28: Stresses in Reinforcement and Concrete for Column with Half Cover Spalling, $e/h=0.785$
Figure A.29: Stresses in Reinforcement and Concrete for Column with Full Cover Spalling, e/h=0.125

Figure A.30: Stresses in Reinforcement and Concrete for Column with Full Cover Spalling, e/h=1.0
APPENDIX B

STATISTICA RESULTS
B.1 STATISTICA Plots for Column Section Characterized as Compression Controlled
e/h < 0.4

Figure B.1: Predicted vs Observed Values for Case II

Figure B.2: Normal Probability of Residuals for Case II
Figure B.3: Predicted vs Observed Values for Case III

Figure B.4: Normal Probability of Residuals for Case III
Figure B.5: Predicted vs Observed Values for Case IV

Figure B.6: Normal Probability of Residuals for Case IV
Figure B.7: Predicted vs Observed Values for Case V

Figure B.8: Normal Probability of Residuals for Case V
Figure B.9: Predicted vs Observed Values for Case VI

Figure B.10: Normal Probability of Residuals for Case VI
B.2 STATISTICA Plots for Column Section Characterized as Compression Controlled $e/h \geq 0.4$

Figure B.11: Predicted vs Observed Values for Case II

Figure B.12: Normal Probability of Residuals for Case II
Figure B.13: Predicted vs Observed Values for Case III

Figure B.14: Normal Probability of Residuals for Case III
Figure B.15: Predicted vs Observed Values for Case IV

Figure B.16: Normal Probability of Residuals for Case IV
Figure B.17: Predicted vs Observed Values for Case V

Figure B.18: Normal Probability of Residuals for Case V
Figure B.19: Predicted vs Observed Values for Case VI

Figure B.20: Normal Probability of Residuals for Case VI