The stress-strain relationship of concrete confined with corroded transverse reinforcements

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2015

Zhang, G. (2015). The stress-strain relationship of concrete confined with corroded transverse reinforcements. Master's thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/65471>

<https://doi.org/10.32657/10356/65471>

NANYANG
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**THE STRESS-STRAIN RELATIONSHIP OF
CONCRETE CONFINED WITH CORRODED
TRANSVERSE REINFORCEMENTS**

ZHANG GUANGHUI
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
2015

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**SCHOOL OF CIVIL AND ENVIRONMENTAL
ENGINEERING**

**A thesis submitted to
Nanyang Technological University
in partial fulfilment of the requirements for the degree of
Master of Engineering
2015**

ACKNOWLEDGEMENT

The research reported in this master thesis was conducted at the School of Civil and Environmental Engineering of Nanyang Technological University in Singapore under the supervision of Associate Professor Li Bing.

I would love to express my heart-felt gratitude to my supervisor Associate Professor Li Bing for his guidance along the way of writing this thesis. He not only checks my research progress, but also gives me precious research materials and provides me with valuable suggestions when I am stuck with tenacious research problems. Furthermore his encouragement makes me continue my research with more positive attitude. And he urges me to live beyond the impossibilities and break the limits of my physical inabilities, for which my heart is filled with much joy and thanks.

I would also love to extend the appreciation to my Research friends, Ning Chaolie, Zhang Zhongwen, Vu Ngoc Son, etc., for their precious advice in research methods. They kindly shared research tips to help me save a lot of time when finishing this report. I like to give thanks to Research Fellow Associated Professor Yu Bo for helping me focus on the key points in the research and answering my questions in this research. Furthermore, I deeply appreciate all the staff in PE/CT Lab and CESEL. Experiments can't be done without their help and assistance. I also need to thank my mother for her constant love and unconditional support. Her words are the sources of my strength. I also would like to take this opportunity to give thanks to my girlfriend, who comes to my life like an angel. He always comforts me, and understands me in the pursuit of this degree.

Lastly, I want to acknowledge the School of Civil and Environmental Engineering, Nanyang Technological University for providing funds for this research and creating a beautiful environment for living and researching.

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SUMMARY

As service years increase, the corrosion of steel rebar stands out as a major problem for existing reinforced concrete (RC) structures in corrosion-inducing environment. The mechanism of steel corrosion in concrete is an electro chemical process, which is often accelerated by the ingress of aggressive chemicals, for example chloride ion. The accumulation of corrosion products on steel rebar is able to generate circular stress which could result in cover cracking. Corrosion of steel rebar will degrade the physical appearance and reduce its original cross section. Corrosion often appears to be non-uniform and localized. Corrosion damaged RC elements displayed smaller yield strength, ductility, energy dissipation capacity, etc. Corrosion level of stirrup tends to be higher than longitudinal rebar due to smaller diameter and less cover protection. Stirrup corrosion decreased confinement behaviour on concrete, thus exacerbating the degeneration of the deformation capacity and the ductility of the RC structures. Experimental studies on the concentric loading behaviour of reinforced concrete column with corroded stirrup and residual confined concrete strengths are presented. Based on maximum pit depth and nominal average pit depth, localization level is proposed to quantify the degree of pit corrosion. Experimental results prove that localization level is not strongly correlated with weight loss level. Columns with highly corroded stirrup have compromised compressive capacity and axial deformation capacity, which both are more related with weight loss level than localization. Analysis reveals that the mere reduction of cross section area can not sufficiently account for the effect of corrosion on the maximum lateral pressure of transverse reinforcement. Based on Mander confined concrete model, a new model is proposed by introducing correction factors to quantify the variations. Regression analysis has helped establish relationships between correction factors and weight loss level or localization level of stirrups.

Keywords: Stirrup Corrosion, Accelerated Corrosion, Chloride Attack, Pit Depth, Compressive Capacity, Deformation Capacity, Residual Confined Concrete Strength

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LIST OF SYMBOLS

t	Time
C	Chloride ion concentration
D_a	Apparent diffusion coefficient
C_s	Chloride concentration on the concrete surface
T_{ini}	Corrosion initiation time (in years),
α	Cover depth
k_e	Environmental function
k_t	Regression parameter
$D_{RCM,0}$	Chloride diffusion coefficient (m ² /S)
t_0	Reference point of time (years)
n	Aging exponent
C_{crit}	Critical chloride concentration
K_p	The rate of rust production
W_{crit}	The critical amount of corrosion products to induce cover crack
u_b	Bond strength
C_0	Corrosion level
f_{cu}	Concrete cubic strength
s	The slip between concrete and reinforcement bar
β	The bond strength reduction factor
T_{corr}	Time when corrosion initiates
d_{bi}	The diameter of uncorroded bar
T_f	The time when the bar is being corroded completely
w/c	The water cement ratio,
d_c	Cover depth
i_{corr}	Corrosion rate
κ	Corrosion penetration factor
r_{corr}	Corrosion rate by the uniform diameter loss of rebar
A_0	The uncorroded bar cross-section area

t	The elapse time in service years
T_f	The time when the bar cross section becomes zero
w_0	The weight of sound rebar
w_r	The remaining weight of the rebar after corrosion.
w_{loss}	The weight loss of corroded rebar
G_{f0}	The coefficient for determination of the fracture energy
d_{max}	The maximum aggregate size
f'_c	The cylinder compressive strength of concrete
ω	Weight loss level (%)
ω_t	Weight loss level of stirrups or transverse rebars (%)
ω_l	Weight loss level of longitudinal rebars (%)
δ	Localization level
A_m	Atomic mass of iron (56g/mol)
I	Corrosion current (A, 1A = 1C/s)
Z	Valency (Z is taken as 2, assuming most of rust product is due to $Fe(OH)_2$)
F	Faraday's constant (96485 C/mol)
m_r	The residual mass after being corroded and then being washed and cleaned.
m_0	The original mass of rebar before corrosion
p_{max}	The maximum pit depth
p_{ave}'	The nominal pit depth, $p_{ave}' = \omega d$
β	The strength reduction factor for cracked cover concrete
r_s	The specific weight of the steel
p_{cp}	The perimeter of the cross section
s	The stirrup spacing
C_c	The clear cover depth
f_{y0}	The yield strength of the uncorroded rebar
f_{yc}	The residual strength of corroded rebar
f_{ccr}'	Residual confined concrete strength with corroded transverse rebar

f'_{co}	The strength of unconfined concrete
f'_{cc}	The strength of uncorroded concrete
k_e	The confinement effective factor
f_l	The maximum lateral pressure of transverse reinforcement
ρ_s	The transverse rebar ratio in the square concrete core
f_{yh}	The yield strength of the transverse rebar
b_c	The side length of the square concrete core
w'_i	The i th clear distance between longitudinal bars
s'	The clear distance between the transverse rebar
ρ_{cc}	The longitudinal reinforcement ratio in the concrete core
f_{lc}	Maximum lateral pressure of corroded stirrup
η	The correction factor for maximum lateral confinement pressure
ζ	The correction factor for confined strain
ψ	The correction factor for curve slope

CHAPTER 1

INTRODUCTION

1.1 Introduction

The introduction chapter talks about research background and organization of this thesis. It elaborates the research scope by discussing background and the current research gap. Research scope is covered by narrowing down the research subject to corrosion damaged reinforced concrete column and the residual confined concrete strength. Finally, it specifies the organization of this report and reveals a detailed skeleton of the following chapters.

1.2 Background

A high percentage of the current existing RC structures constructed from 1950s to 1990s are approaching the designed service year. The aging problems have risen from these RC structures. Trillions of dollars is required to renew and update American aged infrastructure which were built mainly after World War II. In Europe, many RC structures have exceeded their designed lifespan and are likely to fall down. In Asia, aged RC buildings even had to be demolished before being completely abandoned. Aging is an urgent issue facing many RC structures around the globe.

Corrosion of steel bar is one of the major issues the aged RC structures have to confront with. Recent years have witnessed great economic loss incurred by steel bar corrosion. Last year one RC structure in Qingdao China was even demolished because the corrosion had severely deteriorated the columns and slabs. Corrosion issue is exacerbated by exposure to corrosion-inducing environment, including de-icing salt, saltwater along the coastline, carbon dioxide, sulfur dioxide, *et al.* The chloride from salt ($NaCl$) acts as an efficient catalyst in accelerating the corrosion process. In Southeast Asia, most major cities are located near the sea. Many RC structures in the coastal areas are under chloride attack. Further, humid and sultry

environment in Southeast Asia could further speed up the chemical reactions involved in the corrosion process.

The corrosion of the existing reinforced concrete structure has raised great concern over its safety and seismic performance among practise engineers, researchers and residents, etc., because steel bar is the most essential element in RC. Corrosion reduces the effective cross-section area of longitudinal and transverse reinforcement bars. Furthermore, the volume expansion of the corrosion product will give rise to cover cracking, as shown in Figure 1.1 and Figure 1.2, which results in the damage of RC element cross section and the degradation of bond strength. The mechanical property and the integrity of RC structure could be fundamentally damaged by steel bar corrosion.

Figure 1.1: Corrosion-induced Cover Cracking

Figure 1.2: Cover Cracking and Stirrup Fracture (Higgins and Farrow III 2006)

In recent years, research scholars, like Gong *et al* (2004), Ma *et al* (2012), Niu *et al* (2005), and Wang *et al* (2008) had conducted experimental research on the reinforced concrete column with corroded steel rebar. Dai *et al* (2005) and Liu *et al* (2010) had researched on the behaviour of corrosion damaged RC joint. Furthermore, Castel *et al* (2000), Ou *et al* (2014), Xia *et al* (2011) and so on had experimental investigated on how corrosion could affect the loading capacity and

seismic resistance of RC beam with corroded steel rebars. A detailed review of their lab testing is covered in following chapters.

In their experiments it has been found out that the transverse rebar could incur more severe damage than longitudinal rebar, and even break at the junction point with the longitudinal rebar. The diameter and concrete cover depth of transverse reinforcement are smaller than that of the longitudinal reinforcement so transverse reinforcement is more vulnerable to corrosion, especially those at the corner of the column. The main reason is that the smaller depth of concrete cover makes the transverse reinforcement more easily reached by chloride and other corrosion induction chemicals. And when the passivation layer over the surface of transverse reinforcement has already been damaged, the longitudinal reinforcement is still possibly under the protection of passivation layer. At this time, the junction between the transverse and longitudinal reinforcement will generate galvanic corrosion which happens between two different materials. Transverse reinforcement acts as the anode and the longitudinal reinforcement acts as the cathode, in which case the longitudinal rebar is protected to some degree and on the other hand the corrosion of transverse reinforcement is accelerated. Furthermore, the diameter of transverse reinforcement is smaller and thus more sensitive to the corrosion. Thus transverse reinforcement corrosion turns out to be often a more prevalent problem. The corrosion of stirrup will seriously degrade the confinement behaviour and thus reduce the compression and shear resistance of the column. However, up until now the specific research on how the corroded transverse reinforcement will affect the behaviour of RC structure component is limited (Ou and Chen 2014). Research efforts are required to reveal how the corroded of transverse reinforcement will change confined concrete strength and the consequent effect on the mechanical characteristic of RC component.

1.3 Organization of Report

Corrosion has been affecting the mechanical behaviour and compromising the performance of RC structure. The lack of research in this filed motivates me to determine my research scope as follows. This QE report is divided into five

chapters including this introductory chapter which covers research background and organization of report.

Chapter 2 focuses on the literature review on material degradation of corroded RC. Firstly the mechanism of corrosion is studied from the perspective of chemistry. It elaborates how the corrosion is initiated, progressed and finally cause concrete cover cracking. Bond deterioration is also explained in detail. Several bond-slip models had been reviewed. Chapter 2 also reviews on the researches on rebar strength reduction due to rebar corrosion. Chapter 3 reviews major experimental programs on corrosion damaged reinforced concrete component. The state-of-art experiment tests on corrosion damaged column, joint, beam are covered by describing the design of the specimens, accelerated corrosion experiment, loading test, and the conclusions of experiments.

Chapter 4 covers experimental study on residual confined concrete strength with corroded stirrup. It discusses the details of designing RC column specimens. Besides, it describes the procedures of applying current to stirrup to achieve corrosion meanwhile isolating longitudinal rebar. After having obtained corrosion-damaged corrosion, exert axial compressive loading on RC column. Then analyze the influence of corroded stirrup on the compression capacity and confined concrete strength. New model is proposed to estimate the stress-strain curve of confined concrete with corroded stirrup.

Chapter 5 talks about the major achievements from this study and suggestion for future research.

CHAPTER 2

LITERATURE REVIEW ON MATERIAL DETERORATION CAUSED REBAR CORROSION

2.1 General Information of Corrosion Mechanism

2.1.1 Chemical reaction of rebar corrosion

The corrosion of rebar is an electro-chemistry process by nature. It is initialized by neutralization of the alkaline in concrete by the acidity penetrating from the environment. Three major electric-chemistry reactions are involved in the corrosion process, shown as follows:

Equation 2.1 shows the major anodic reaction of dissolving iron metal into metal ion, the damage of steel rebar. Equation 2-2 illustrates the cathodic reaction of reducing oxygen into hydroxyl ions. Equation 2-3 demonstrates the formation of hydrated ferric oxide, generally known as rust. These three equations imply that the occurrence of corrosion relies on the presence of water and oxygen. The unavoidable pores in the concrete would let the water and oxygen penetrate in from the environment. The surface of original steel bar is closely adhered by the thin passivation layer, which is generally formed by condensed corrosion products (mainly FeO and Fe_2O_3). This passivation layer could prevent the oxygen from accessing the Fe element. The existence of passivation layer critically depends on the high PH value alkalinity. But the acidity from the environment infiltrates into the concrete through pores and touches the surface of the reinforcement, lowering the PH value of the surrounding of the reinforcement and thus neutralizing the

alkaline. As a result, acidity damages the passivation layer on reinforcement gradually.

After the rebar loses the protection of passivation layer, steel bar starts to lose iron element. This process is interfered by the corrosive chemical from the environment and within the concrete itself. The substances potentially causing and accelerating corrosion includes the chloride ion (Cl^-), carbon dioxide (CO_2), sulfur dioxide (SO_2), etc. The chloride comes from the de-icing salt, coastal vapour, etc. (Li *et al.* 2009). Chloride is highly active and capable to damage the passivation layer. Figure 2.1 vividly shows that chloride acts as catalyst in the corrosion process. Chloride is not contained in any of the corrosion products (Neville 1995).

Figure 2.1: Illustration of chloride attack on steel bar

As shown in Equation 2-4, chloride is able to form chloride complex ions with ferrous ion Fe^{2+} easily, which as a result destabilizes the equilibrium in Equation 2-1. The anodic reaction proceeds in the direction to cause more metal transformed into ferrous ions. As is expressed in Equation 2-5, the instable chloride complex ion $FeCl_4^{2-}$ could react with alkaline cement paste, thus releasing the chloride ions Cl^- :

Other substance includes CO_2 could cause the carbonization of the concrete, which will neutralize the concrete by creating carbonic acid. The carbonation is mainly that the carbon dioxide in the air diffuses into the concrete and neutralizes the alkalinity of concrete by the chemical process, shown in Equation 2-6:

Carbon Dioxide CO_2 could reduce the alkalinity of the concrete. Others chemicals include SO_2 and NO_2 could be combined with water in pore of concrete and create acidity as well. The decrease of alkalinity deteriorates the passivation film on the bar. Berkely & Pathmanaban had discovered that when a pH level drops down to 9.5 the corrosion of steel reinforcement would be initiated (Berkeley and Pathmanaban 1990).

2.1.2 Diffusion of chloride and corrosion initiation estimation

Chloride is very detrimental to the health of RC structure by dissolving more steel metal as has been discussed in the previous section. It could induce and accelerate the corrosion of steel bar. The corrosion process was subdivided into different phases, initial phase, pre-cracking phase and post-cracking phase. Initial phase begins with the diffusion of chloride, which could happen from the time when RC structure is constructed. Initial phase ends at the time when the passivation layer is destroyed locally. The duration of this phase mainly depends on the concrete cover, the transport rate of the chloride, water and oxygen and other environmental or structural factors. The primary transport of the chloride through the pores of the concrete cover is diffusion. Fick's second law of diffusion is applied to estimate the diffusion of the chloride into concrete (Ôemajtis 1998). It is a function of concrete cover depth and time, shown in Equation 2-6:

$$\frac{dC(x,t)}{dt} = D_a \frac{d^2C(x,t)}{dx^2} \quad (2-6)$$

where x is the distance from the surface of the concrete to the specified point in the concrete, t is the time, $C(x,t)$ is the chloride ion concentration, D_a is apparent diffusion coefficient (cm^2/s).

The solution to Equation (2.6) is as follows:

$$C(x,t) = C_s \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_a t}} \right) \right] \quad (2-7)$$

where C_s is the chloride concentration on the surface of concrete, erf is the error function.

Equation 2-8 establishes the relationship between time and chloride penetration status. It could be applied to estimate the time when chloride concentration value on the surface of rebar reaches the threshold value, which initiates the corrosion process. Various corrosion initiation estimation models had been proposed based on this relationship. Model proposed by Thoft-Christensen is as follows (Thoft-Christensen *et al.* 2008).

$$T_{ini} = \frac{d^2}{4D_c} \left[\operatorname{erf}^{-1} \left(1 - \frac{C_{crit}}{C_s} \right) \right]^{-2} \quad (2-8)$$

where T_{ini} is the corrosion initiation time (years), d is the cover depth (mm), D_c is the chloride diffusion coefficient (m^2/s), C_{crit} is the critical chloride concentration (wt % cement), C_s is the surface chloride concentration at the concrete surface (wt % cement), erf is the Gaussian error function.

CEB FIP Task Group 5.6 (béton and Concrete 2006) recommended probabilistic model:

$$T_{ini} = \left(\frac{\alpha^2}{4 \cdot k_e \cdot k_t \cdot D_{RCM,0} \cdot (t_0)^n} \left(\operatorname{erf}^{-1} \left(1 - \frac{C_{crit}}{C_s} \right) \right)^{-2} \right)^{\left(\frac{1}{1-n} \right)} \quad (2-9)$$

where T_{ini} is corrosion initiation time (years), α is cover depth (mm), k_e is environmental function, k_t is regression parameter, $D_{RCM,0}$ is chloride diffusion coefficient (m^2/s), t_0 is the reference point of time (years), n is aging exponent, C_{crit} is the critical chloride concentration (wt % cement), C_s is the surface chloride concentration (wt % cement), erf is the Gaussian error function.

$$erf(x) = \frac{2}{\sqrt{\pi}} \int_0^x e^{-p^2} dt \quad (2-10)$$

As is discussed above, the corrosion rate estimation is directly related to diffusion coefficient D_c , which describes the transport rate to chloride ions. And Equation 2-8 and 2-9 insinuates that corrosion initiation time T_{ini} decrease with diffusion coefficient D_c . Thoft-Christensen suggested that D_c was a function of water-cement ratio, temperature, the amount of silica fume (Thoft-Christensen *et al.* 2008). Increased w/c ratio means more fluid, which results in less chloride being bound by water. High temperature could cause more active thermal motion of particles, which reduces bound chloride as well. So diffusion coefficient D_c increases with w/c ratio and temperature. However, diffusion coefficient decreases with silica fume, because silica fume will inhibit the chloride from penetrating into concrete.

Appropriate estimation of corrosion rate in the phase could provide essential guidance on the mechanical property of corroded concrete on time variance. Based on whether the corrosion is variable in the long term, the corrosion rate models are categorized into constant rate models and variable rate models. Corrosion rate developed by Vu Stewart is proven to be the most valid model to calculate the corrosion rate of RC structure under chloride attack (Vu and Stewart 2000, Vu, Stewart *et al.* 2005), so it substantially provides support for corrosion level estimation.

2.2 Literature Review on Material Degradation

2.2.1 Concrete deterioration caused by rebar corrosion

The physical deterioration of concrete caused by rebar corrosion is divided into two parts, before cover cracking (pre-cracking phase) and after cover cracking (post-cracking phase). The time of cover-cracking is deeply related to corrosion rate, cover depth, corrosion products type, concrete properties, etc.

Pre-cracking phase is the process of accumulation corrosion products to the extent of causing concrete cover cracking. Figure 2.2 demonstrates the relative volumes of each corrosion iron compound to iron metal (Nielsen and Geiker 2003).

Figure 2.1: Relative volume of each corrosion iron compound to iron metal

Corrosion products are a mix of different kinds of compounds. And the average volume of corrosion iron compound is five times the volume of the original metal iron (Bertolini *et al.* 2004). Thus the expansion of the corrosion products will generate circular stress in the concrete, which could eventually cause cover concrete to crack, as demonstrated in Figure 2.3.

Figure 2.3: Corrosion induced concrete cracking

Predicting the duration to reach the threshold circular stress holds value in estimating the degradation of material as a variant of time. Bhargava proposed analytical models to predict the time to cause cover cracking by considering the cracked concrete residual strength, the stiffness of reinforcement and corrosion products combine (Bhargava *et al.* 2005). In this model, the prediction is expressed as the solution of boundary value problem in which cover concrete is divided into cracked zone and uncracked zone and the governing stress equilibrium equations are expressed in radical displacement. The time to cover cracking estimation model developed by Y. Liu had focused on evaluating the amount of corrosion products needed to fulfil the interconnected void space around the reinforcing bar and to generate enough corrosion pressure to crack the cover concrete (Liu and Weyers 1998). Y. Liu assumed a constant corrosion rate and integrated the equation and found the time to cover cracking could be considered as follows:

$$T_{CR} = \frac{W_{crit}^2}{2K_p} \quad (2-16)$$

in which W_{crit} is the critical amount of corrosion products to induce cover cracking, K_p is the rate of rust production. The critical amount of corrosion products mainly include two parts; first part is the amount of products to fill the total interconnected pores around the rebar/concrete interface, the second part is the amount of products to produce critical tensile stress in the concrete.

Apart from empirical and analytical approach, some researchers used FEM software to simulate the chloride attack process and model the cover concrete

evolved as the corrosion products accumulated. Millard had considered all the procedures on the degradation of reinforced concrete in the coupling model. More specifically, the model simulated the coupling behaviour between hydration of concrete and oxygen transport, corrosion reaction, and mechanic damage of concrete. Pan & Lu (2011) and Pan & Wang (2011) had successfully developed FEM model to simulate the three key steps of corrosion evolution: corrosion-inducing chemical ingress, rebar corrosion, and concrete cover cracking. The analysis numerically modelled the chemical transport in the variably saturated concrete, predicted crack initiation time and evaluated the service life of RC in chloride-bearing environment. Researchers had harnessed the power of computer simulation to clarify the complex process of rebar corrosion.

2.2.2 Bond deterioration caused by rebar corrosion

The bond ensures the concrete and reinforcement will work coherently. When the bond strength is not enough to maintain the connectivity between concrete and reinforcement, the bond-slip failure could occur. The bond between the reinforcement and the concrete is composed of three different kinds of action, namely the chemical adhesion, the friction and the mechanical interlocking. Many researchers had found the different trend of bond strength changing in different phases of corrosion. In the primary phase, the expansion of the corrosion products enhances the confinement behaviour of the concrete to the rebar. The expansion of corrosion products exerts pressure on the interface between the surrounding concrete and bar, consequently increasing the friction. Moreover, the particle of corrosion products raises the friction coefficient, which is more obvious for smooth bars. The two factors confinement pressure and friction coefficient both contribute to the increase of bond strength in the primary phase. But as more corrosion products accumulate, the chemical cohesion will be deteriorated. It may even lead to the reduction of the friction coefficient. For deformed bar, the ribs could be eradicated, which deduces the mechanical interlocking. And the expansion effect of the corrosion products can lead to the cracking of the concrete cover, conversely resulting in the decrease of the confinement and friction force.

Thus the bond strength declines as the corrosion level exceeds the boundary value (generally it is 5%). The following graph better demonstrate the process of bond degradation.

Figure 2.4: Bond degradation mechanism of rebar corrosion

Many researchers had committed efforts to study the factors influencing the bond strength, including the cover to bar diameter ratio, stirrup ratio, and configuration of longitudinal bar. Fang *et al* (2004) had conducted experiments on how the transverse reinforcement and the configuration of the reinforcement bar will affect the bond on different levels. It has been found that the bond strength of deformed bars without transverse reinforcement declined considerably as the corrosion levels increase. However, corrosion levels do not affect the bond strength of deformed bars with transverse reinforcement significantly. Compared with uncorroded bars, the bond strength of corroded bar went through a post-peak plateau or residual bond strength. As for smooth bars, at the low to medium levels the bond strength surprisingly increased when the corrosion level increased. This is due to the increase of confinement pressure between concrete and bars when the corrosion levels increase. Another reason could be that the corrosion products between the bar and concrete enhance the friction coefficient, which eventually

increase the bond strength. Generally the bond strength of smooth bar without transverse reinforcement dropped faster than smooth bar with transverse reinforcement, as is the case with deformed bar. Therefore, how the bond strength will be affected by the corrosion level generally depends on the confinement effect of transverse reinforcement and surrounding concrete after being damaged by corrosion.

Chung *et al* had proposed bond strength model based on the LSM regression analysis of the experiment tests (Chung *et al.* 2008). It is as follows:

$$u_b = 16.87 \text{ for } C_0 \leq 2.0 \quad (2-17)$$

$$u_b = 24.7C_0^{-0.55} \text{ for } C_0 > 2.0 \quad (2-18)$$

In which u_b is the bond strength, and C_0 is the corrosion level (Chung *et al.* 2008). Further, the model is compared with models put forward by Lee *et al* (2002) and Cabrera *et al* (1996). The comparison showed that the new proposed model would provide a better prediction of the bond strength.

It has been found that the increase of bar diameter and the decrease of concrete strength will increase the risk of the corrosion-induced bond-slip failure. The combined effect of reinforcement corrosion and applied load (Li and Yang 2011) could affect the crack. Pan and Lu (2011) also developed an equation to predict the time of cover cracking. The corrosion time is directly related to the original bar diameter d , the percentage of mass loss of reinforcement ρ and the corrosion current density i_{corr} , as is expressed:

$$t = 234762 \frac{\rho d}{i_{corr}} \quad (2-19)$$

in which the units for t , ρ , d are hour, %, mm, $\mu\text{A}/\text{cm}^2$.

Numerical modelling simulation is another method to predict the bond strength. In the numerical approach, rust is assumed to behave like a granular material. Both experimental and numerical results indicated that deformed bar with stirrups the transverse reinforcement could counteract the bond loss caused by rebar corrosion on the corrosion level around 4% (Fang *et al.* 2004).

Frequently used bond-slip models for reinforced concrete in the literature are as following:

(1). Bond-slip Model proposed by Mirza and Houdle (Mirza & Houde 1979, Amleh & Mirza 1999)

Mirza and Houdle proposed a relationship which is as follows:

$$u_b = (5.3 \times 10^2 s - 2.52 \times 10^4 s^2 + 5.86 \times 10^5 s^3 - 5.47 \times 10^6 s^4) \sqrt{\frac{f_c}{40.7}} \quad (2-20)$$

Where u_b is the bond strength;

f_c is the concrete strength, unit in MPa;

s is the relative displacement between rebar and concrete.

(2). Bond-slip model in CEB-FIP 1990 (Beton 1993)

The transfer of tensile forces from reinforcement bars into concrete plays an essential role in the RC structure performance. The transfer depends on the bond strength between rebar and concrete. CEB-FIP 1990 provides guidance on the bond-slip value of confined concrete (Beton 1993), as shown in Figure (2.4) and Table (2.1). The bond strength depends on two major conditions: whether the concrete is confined or not; whether bond is good or bad. And the ascend branch of the model is defined by the equation:

$$\tau_b = \tau_{b,max} \left(\frac{s}{s_1} \right)^\alpha \text{ when } s \leq s_1 \quad (2-21)$$

where τ_b is the bond stress

$\tau_{b,max}$ is the maximum bond stress

s is the slip

s_1 is the slip at the maximum bond stress

α is the exponent index

Figure 2.5: Bond-Slip Relationship (béton 1993)

Table 2.1 Parameters for Defining Bond-Slip Value (CEB-FIP 1990)

	Unconfined Concrete		Confined Concrete	
	Good Bond Condition	All Other Bond Condition	Good Bond Condition	All Other Bond Condition
S_1 (mm)	0.6	0.6	1	0.6
S_2 (mm)	0.6	0.6	3	0.6
S_3 (mm)	1.0	1.0	Clear rib spacing	Clear rib spacing
α	0.4	0.4	0.4	0.4
$\tau_{\max}$ (MPa)	$2\sqrt{f'_c}$	$\sqrt{f'_c}$	$2.5\sqrt{f'_c}$	$1.25\sqrt{f'_c}$
τ_f (MPa)	$0.15\tau_{\max}$	$0.15\tau_{\max}$	$0.4\tau_{\max}$	$0.4\tau_{\max}$
S_4 (mm)	-	-	$0.5S_3$	$0.5S_3$
τ_y (MPa)	-	-	$0.5\tau_f$	$0.5\tau_f$

(3). Modified CEB-FIP 1990 bond-slip model for high strength concrete (Engstrom *et al.* 1998)

Engstrom *et al* (1998) conducted experiments tests indicating the yield of the bar would change the interlocking effect between the ribs on bar and the surrounding concrete because of the increase of axial strain and contraction of the bar after yielding. Engstrom *et al* proposed a modified bond-slip model if the steel goes in

the status of plasticity. The descending branch starts just after yielding of rebar, shown in Figure 2.6. Tests had confirmed the strength concrete would affect the bond values and parameters had been provided to indicate the bond-slip model of high strength concrete (béton 1993), shown in Table 2.2

Figure 2.6: Modified Bond-slip Relationship regarding the yield of rebar (Engstrom *et al.* 1998)

Table 2.2 Parameters for Defining Bond-Slip Value

Paramete rs	Bond Condition			
	Normal-strength Concrete		High-Strength Concrete`	
	Good	All Other Condition	Good	All Other Condition
S_1 (mm)	1	1	0.5	0.5
S_2 (mm)	3	3	1.5	1.5
S_3 (mm)	Clear rib spacing	Clear rib spacing	Clear rib spacing	Clear rib spacing
S_4 (mm)	3 rib spacings	3 rib spacings	3 rib spacings	3 rib spacings
α	0.4	0.4	0.3	0.3
$\tau_{\max}$ (MPa)	$0.45f_{cm}$	$0.225f_{cm}$	$0.45f_{cm}$	$0.225f_{cm}$
τ_f (MPa)	$0.40\tau_{\max}$	$0.40\tau_{\max}$	$0.40\tau_{\max}$	$0.40\tau_{\max}$
$S_{y,f}$ (mm)	$S_y + 2.5$	$S_y + 2.5$	$S_y + 2.5$	$S_y + 2.5$
S_5 (mm)	2 rib spacings	2 rib spacings	2 rib spacings	2 rib spacings
τ_{yf} (MPa)	$0.20\tau_{\max}$	$0.20\tau_{\max}$	$0.20\tau_{\max}$	$0.20\tau_{\max}$

Ou *et al* (2013) proposed a modification parameter based on the corrosion length of reinforcement bars to consider the reduction of bond strength resulted from corrosion. The modification factor Φ shows the change of L_{eu} , where L_{eu} is referred to equivalent unbounded length of the bar with the strain distribution due to slip elongation S_E from the joint. L_{eu} can be formed with equation for each steel strain value. The modifier factor to modify strain of the steel is shown in Equations:

$$L_{eu} = \frac{S_E}{\varepsilon_s} \quad (2-22)$$

$$\Phi = \frac{L_{eu}(\text{uncorroded bar})}{L_{eu}(\text{corroded bar})} \quad (2-22)$$

2.2.3 The reduction of rebar cross section area due to rebar corrosion

The deterioration due to the corrosion of steel rebar will change the bar area, so calculating average cross section area loss and weight loss ratio is the most straightforward way to measure the corrosion level. Choe *et al* (2008) developed time-varying bar diameter prediction model, based on the corrosion rate function proposed by Vu & Stewart (2000).

$$d_b(t, T_{ini}) = \begin{cases} d_{bi}, & t \leq T_{ini} \\ d_{bi} - \frac{1.0508(1 - w/c)^{-1.64}}{d_c} (t - T_{ini})^{0.71}, & T_{ini} < t \leq T_f \\ 0, & t > T_f \end{cases} \quad (2-26)$$

Where T_{ini} is the time when corrosion initiates, d_{bi} is the diameter of uncorroded bar, $d_b(t, T_{corr})$ is the diameter of the bar at time t , T_f is the time when the bar is being corroded completely and bar diameter reaches zero, w/c is the water cement ratio, d_c is the cover depth.

Vu & Stewart (2000) had put forward equations to prediction the performance of RC structure on the basis of time. The residual average bar diameter of steel bar could also be estimated by corrosion rate (Ghosh & Padgett 2010), as following:

$$D(t) = D_i - i_{corr} \cdot \kappa \cdot (t - T_{ini}) \quad (2-27)$$

where i_{corr} is the corrosion rate (mA/cm²); κ is the corrosion penetration (usually 11.6μm/year, assuming uniform corrosion penetration for generalized corrosion).

Corrosion rate could be evaluated as the uniform diameter loss of rebar, like

$$r_{corr} = i_{corr} \cdot \kappa, \text{ (unit: mm/year)}$$

Once the passivation film on the surface of reinforcement bar dissolves due to continued chloride attack (Ghosh & Padgett 2010), corrosion initiates and the time-variant corrosion level depending on the bar cross-section area loss can be expressed as the following:

$$\Delta c = 1 - \frac{A(t)}{A_0} = \begin{cases} 0 & \text{for } t \leq T_{ini} \\ 1 - \left(\frac{[D(t)]}{D_i} \right)^2 & \text{for } T_{ini} < t < T_f \\ 1 & \text{for } t \geq T_f \end{cases} \quad (2-28)$$

where Δc is the cross section area loss ratio, A_0 is the uncorroded bar cross-section area, t is the elapse time in service years, T_f is the time when the bar cross section becomes zero (completely corroded, or fractured), expressed as:

$$T_f = \frac{D_i}{r_{corr}} \quad (2-29)$$

Another method to estimate corrosion level is by calculating weight loss ratio (Ma *et al.* 2012). This method has been frequently used in experimental research because it is easier to measure residual weight than cross section area.

$$\Delta w = \frac{w_0 - w_r}{w_0} \quad (2-30)$$

where w_0 is the weight of sound bar;

w_r is the remaining weight after corrosion.

In literature, most of the research had been based on the assumption that corrosion rate is constant and corrosion level can be estimated by average cross section area loss and average weight loss. However, corrosion has the characteristic of non-uniformity by nature. Localized corrosion on rebar will cause stress and strain concentration, potentially creating defect zones. The most prominent kind of localized corrosion could develop to be pitting corrosion, which creates big pit depth and possibly break the steel rebar. (Wang *et al.* 2011). Higgins *et al* (2006) and Ou *et al* (2014) had measured the maximum pit depth and the minimum section area to calculate the corrosion level of steel bar. Partial length corrosion describes the case when the corrosion is concentrated along a certain length of rebar (Wang and Liang 2008). Lower part of exterior columns of frame or bridge columns are more prone to suffer from partial length corrosion because they are closer to the ground and exposed to more severe environment. But little research had been focused on using a similar parameter as weight loss level to estimate non-uniformities of corrosion.

2.2.4 Rebar material strength degradation due to rebar corrosion

This section will explain a state-of-art research on the stress degradation of corroded rebar. Corrosion changes the material property of steel rebar. This section specifies how the corrosion affects the constitutive relationship of rebar

2.2.4.1 Research by Allam & Maslehuddin (1990)

High temperature and humidity would make ideal environment for corrosion and wet dry cycles would form unstable rust product. Allam & Maslehuddin investigated the how arid atmospheric condition would make rebar being corroded and how corrosion would affect the mechanical strength of steel rebar (Maslehuddin *et al.* 1990, Allam *et al.* 1994). In the experiment three different types of hot-rolled cold-straightened rebar with diameters of 8, 10, 12 mm and three types of hot rolled rebar with diameters of 12, 16, 32 mm were laid bare in the arid air of the Arabian Gulf West Coast for sixteen months at maximum. After natural exposure, X-ray diffraction analysis was conducted to inquire into the morphology and microstructure of corrosion products on the surface of rebar (Allam *et al.* 1994). The research indicated that corrosion started as localized corrosion at discontinuous spot on the surface of rebar, which would gradually evolve to be uniform attack on the entire surface if long-term exposure to aggressive environment. Analysis showed that sulfur and chloride had high concentration in the corrosion product, which confirmed sulfur dioxide SO₂ and sea salt NaCl were major aggressive corrosion-inducing chemicals. Tensile test following ASTM A 370 Standard was used to get stress-strain curve, from which yield strength and ultimate strength were drawn. Bending test was used on corroded rebar sample and original rebar to study the ductility. Tensile and bending tests indicated yield strength, ultimate strength, elongation and ductility experienced little decrease or even slightly increase when corrosion level increases. Considering the relative slow corrosion rate in natural environment and short exposure period, this analysis conclusion was due to small corrosion level. Insignificant weight loss of the small corrosion level was unable to cause major mechanical change on rebar. This research also indicated natural exposure was

inefficient to generate large numbers of corroded steel rebar for experiment research. Hence, methods must be developed to achieve higher corrosion level.

2.2.4.2 Research by Abdullah (2001)

Abdullah (2001) assessed the influence of corroded rebar on mechanical properties through accelerating corrosion process of rebars embedded in RC specimens. In the experiment a current with density of 2 mA/cm^2 was impressed on steel rebars with diameter of 6 mm and 12 mm. Only part of the specimens was immersed in 5% chloride solution so that rebars were still not soaked but left in air. That was to make sure the corrosion product were not dissolved in solution but still kept on the surface of rebars. The rebars were cleaned following the ASTM G1 standard for weight loss calculation. Instron Universal Testing machine with a capacity of 250 kN was used to test their force-extension relationship. In this research, two different kinds of stress were calculated by considering the average cross section area A_s and the original cross section area A_0 . It was put in this way:

$$A_s = (1 - \eta)A_0 \quad (2-31)$$

$$f = F/A_s \quad (2-32)$$

$$f' = F/A_0 \quad (2-33)$$

Average stress is represented by f , while nominal stress is represented by f' . Since original cross section area A_0 is bigger than average cross section area A_s , nominal stress f' is comparatively smaller than average stress f . It was discovered that when corrosion level reaches 11% the nominal tensile strength of 6-mm diameter rebar falls below the ASTM A 615 required 600 MPa, while 24% corrosion level for 12-mm diameter rebar.

2.2.4.3 Research by Palsson & Mirza (2002)

Palsson & Mirza (2002) studied the mechanical properties of rebar embedded in abandoned concrete bridge. The bridge was demolished because chloride attack on the steel rebar had made it unable to serve. A total number of 103 samples had been cut from the damaged structure. Tension tests were employed to study the

mechanical strength of the rebar samples. The results demonstrated that the yield strength and ultimate strength of all rebars had little variations, but the ductility dropped significantly as the corrosion increased. And the research considered the localization of corrosion along the rebar by the ratio of the smallest cross section to the largest over the gage length of 25 mm. The smaller ratio represented higher localization level. The analysis showed smaller ratio was linked with smaller ultimate strain, which attested that higher localization level was attributed to smaller ultimate strain. Quantitatively a ratio of 0.8 would cause a decrease of 50% in ultimate strain. This research holds great value because all steel rebar samples were directly fetched from real concrete structure which was dismantled after suffering from severe chloride penetration.

2.2.4.4 Research by Yuan *et al* (2000)

Yuan *et al* (2000) obtained the corroded rebars through confined concrete element, electric accelerating and machine manufacturing. Three bars with a diameter of 18 mm were collected from damaged reinforced concrete structure. And by impressing direct current on three rebars of diameter 12, 14, 16 mm and sinking rebars in NaCl solution, rebars of different corrosion level were acquired through accelerated corrosion level. And another batch three rebars of diameter 18 mm were manually cut off its corrosion section along the length in order to represent the removal of steel caused by corrosion. Afterwards all rebars were tested for load extension diagram. The test results indicated that when the corrosion increases, the nominal yield force and yield strength decreases. The elastic modulus did not incur apparent change due to corrosion. And when corrosion level exceeds 10%, the decrease rate of elongation will also increase. Moreover, the force extension diagram of higher corroded rebar had shorter plateau after yielding. And as corrosion increases, the rebar demonstrates less obvious necking shrink. And corrosion of less than 5% has negligible effect on mechanical changes on rebar. Finally, regress analysis had been conducted on the laboratory test results and finite element method modelling result. The following equations had been generated for nominal yield strength, nominal ultimate strength and nominal ultimate strain:

$$\sigma_{yc} = (0.962 - 0.848\Delta w) \sigma_y \quad (2-34)$$

$$\sigma_{uc} = (1.088 - 3.573\Delta w) \sigma_u \quad (2-35)$$

$$\varepsilon_{uc} = (1.088 - 3.573\Delta w) \varepsilon_u \quad (2-36)$$

In these equations, Δw is the average weight loss ratio $Q_c = 100\Delta$. If transforming into corrosion level in terms of average weight loss, they are as:

$$\sigma_{yc} = (0.962 - 0.00848Q_c) \sigma_y \quad (2-37)$$

$$\sigma_{uc} = (1.088 - 0.03573Q_c) \sigma_u \quad (2-38)$$

$$\varepsilon_{uc} = (1.088 - 0.03573Q_c) \varepsilon_u \quad (2-39)$$

This research used steel corroded rebar from three different methods, namely, collect from RC structure element, electric acceleration, machinery manufacturing. General conclusions had been drawn from tension experiments. However, constrain must be realized that the authors fail to differentiate among the data assembled from different methods. Statistic regression analysis may not recognize the differences emerged from using different methods. Especially data obtained through uniformly cut off cross section of rebar may be invalid considering corrosion is generally localized.

2.2.4.5 Research by Du *et al* (2005)

Du *et al* (2005) carried out experimental study on bare rebar and rebar embedded in concrete electrochemical accelerated corrosion or simulated corrosion. It included 78 bare rebars and 30 embedded rebars with the same length of 450 mm and with diameters ranging 8 mm, 16 mm, 32 mm. In the accelerated corrosion, currently density of 0.5, 1.0 or 2.0 mA/cm² was applied. Additionally, cross section of three rebar was reduced uniformly by machine methods so as to compare with accelerated corroded rebar and uncover the effect of natural local corrosion penetration. Two statistical equations were proposed to predict residual force and stress of rebar respectively from experimental result. They introduced force factor

α and strength factor β to reflect the degradation due to local corrosion attack, so the equations are like follows:

$$F = [1 - (0.01 + \alpha)Q_c]F_0 \quad (2-40)$$

$$f = [1 - \beta Q_c]f_0 \quad (2-41)$$

Comprehensive statistic regression analysis on experimental results yielded that force factor α is 0.0043 with correlation factor as 0.919 and yield strength factor β 0.0048 with correlation factor 0.508. And force factor could be expressed in the form of stress factor (Vu *et al.* 2005) as following:

$$\beta = \alpha \frac{A_{s0}}{A_s} \quad (2-42)$$

in which A_{s0} is the initial section area of uncorroded rebar and A_s is the average section area of corroded rebar. As it can be drawn from the equation, residual stress of corroded rebar is more severely reduced than its residual force. This piece of research also pointed out that residual force and stress of corroded rebar dropped faster than the corresponding averagely reduced rebar, which was attributed from local corrosion attack and stress concentration. Furthermore, it revealed that the force and extension relationship of corroded rebar could still maintain the shape without significant change. And it demonstrated that the influence of bar type and diameter on the residual force and stress both could be neglected with a 5% significance level. This piece of research creatively use mechanical methods to get uniformly reduced rebar and provided experimental proof that degradation of stress result from local corrosion. However, it fails to investigate the rebar with higher corrosion level than 16% and under dynamic loading. Hence, the validation of the regression parameters for the equation could be put to the question when applied to highly corroded rebar.

2.2.4.6 Research by Cairns *et al.* (2005)

Cairns *et al.* (2005) researched on the physical property of rebar through simulated corrosion, accelerated corrosion and numerical modelling. In the simulated pitting corrosion a portion of cross section of rebars was cut by mills with cylinder shank. It aims at intentionally creating mechanical defects on rebars in order to simulate corrosion in the mechanical way. The percentage of removed area to its original bar section area was 5, 10, 20, 30, 40 and 50 %. This research also included 25 rebars with diameter of 16 mm and 6 rebars with diameter of 20mm, which all were corroded by impressing electric current. Standard tension test was carried out on the corroded rebars obtained through simulated and accelerated corrosion. Moreover, Cairns conducted parameter study by using numerical method. Corrosion was modelled by assuming section loss which was normal distrusted along the whole bar length. This method was proved to be able to generate a large quantity of samples. The research investigates the ductility characteristic of corroded rebar. Three equations for yield strength, ultimate strength and elongation were derived from the results, like:

$$f_y = (1.0 - 0.012 \cdot Q_c) f_{y0} \quad (2-43)$$

$$f_u = (1.0 - 0.011 \cdot Q_c) f_{u0} \quad (2-44)$$

$$\varepsilon_u = (1.0 - 0.03 \cdot Q_c) \varepsilon_0 \quad (2-45)$$

In these equations, Q_c is corrosion level in percentage, f_{y0} , f_{u0} , ε_0 are yield strength, ultimate strength, elongation of uncorroded rebar, f_y , f_u , ε_u are yield strength, ultimate tensile strength, elongation of corroded rebar.

2.2.4.7 Research by Kashani & Crewe (2013)

Kashani & Crewe (2013) did a comprehensive research on how corrosion affects buckling mechanism, fracture mechanism, low-cycle high amplitude fatigue, energy dissipation capacity and their coupling effects. In view of the lack of research on cyclic behaviour of corrode rebar, Kashani *et al* (2013) designed

experiments to test its response under cyclic loading. Four reinforced concrete columns with dimension of $250 \times 250 \times 700$ mm were made. Eight B500 rebar with diameter of 12 mm was embedded in concrete with mean compressive strength of 30 MPa and with maximum aggregate size of 12 mm. Accelerated corrosion methodology was used on specimens to obtain corroded rebar of different corrosion levels. The experiment setup is shown in Figure 2.7.

Figure 2.7: Demonstration of Corrosion Experiment Setup (Kashani *et al.* 2013)

Monotonic and cyclic loading test was carried out on the corroded rebar using 250 KN universal testing machine. The test yielded results for stress-strain analysis. The equations are for slenderness ratio 8 and 10.

$$f'_{yc} = fy(1 - 0.016Q_c) \quad (2-49)$$

$$f_{yc} = fy(1 - 0.0065Q_c) \quad (2-50)$$

Where f'_{yc} is the nominal buckling stress of corroded rebar calculated through dividing the force by the original cross section area, f_{yc} is the average buckling stress calculated through dividing the force by the average reduced cross section.

Analysis results generate the following questions by regression analysis. Equations for slenderness ratio 15 and 20 are listed :

$$f_{yc}' = fy(1 - 0.020Q_c) \quad (2-51)$$

$$f_{yc} = fy(1 - 0.0124Q_c) \quad (2-52)$$

The experiment revealed that rebar easily fracture under tension after few cycles of loading. And it found out that cyclic response of rebar with slenderness ratio over 10 under high compression had paralleled characteristic with slender beam column element. The localized corrosion was the main reason for unsymmetrical buckling and reduced bulking capacity of corroded rebar under cyclic loading. It also put forward analytical model to simulate fatigue life, in the form of the followings:

$$\varepsilon_p = \theta_f(2N_f)^{-\alpha_c} \quad (2-53)$$

$$\alpha_c = \alpha(1 + 0.4Q_c) \quad (2-54)$$

ε_p is plastic strain at tension fracture, θ_f is material constant, whose value is not affected by corruptions and could be regarded the same as uncorroded rebar 0.219 for B500 rebar, α , α_c is another material strain of uncorroded and corroded rebar, Q_c is in percentage. The model is the first model to incorporate the effect of both inelastic buckling and low-cyclic high amplitude fatigue. However, this model may lose its accuracy gradually when the corrosion level exceeds 25%. Furthermore, this research did not compensate for the lack of study on the stress-strain model of corroded rebar under cyclic loading.

2.3 Summary

This chapter discusses the corrosion mechanism in RC structure, how corrosion affects concrete and rebar. There has been a comprehensive study on the corrosion mechanism research. Neutralization of alkaline and Chloride penetration cause and speed up steel bar corrosion reactions. The expansion of corrosion substance could results in concrete cracking. The interaction between concrete and steel rebar is also changed by corrosion products and cracking. Mechanism of bond degeneration is reviewed. Corrosion not only reduces the cross section area of rebar, but also affects the yield strength. Researchers had conducted experimental research on the stress-strain relationship of corroded steel rebar. Test samples were obtained from different methods, current acceleration corrosion method, and natural corrosion during service and machine manufacture method. The literature review has given a report on state of art research on stress-strain relationship of corroded rebar. The experiments indicated that the yield strength and ultimate strength of the corroded rebar deteriorated because of the localization of the corrosion. And the effect of corrosion on modulus was not significantly. The equations in Section 2.2.4 showed that the yield strength could function as a variable of corrosion level. However, defects in the research will have to be noted. Firstly, bare rebars sunk in the solution and rebar embedded in concrete have different localized corrosion. Rebar in concrete tends to have more pit corrosion than bare rebar. There exists a need to verify which has more similarity with real corrosion in reinforced concrete structure in service. Secondly, even though research founded ultimate strain decreased with the increase of ratio of minimum cross section to maximum cross section, a lack of study on how the ratio is related to yield strength and ultimate strength.

CHAPTER 3

LITERATURE REVIEW ON CORROSION DAMAGED REINFORCED CONCRETE ELEMENT

In order to conduct experimental investigation of mechanical properties of corroded RC structure, two principle methods could be used to obtain corroded RC component and frame for loading test. One is through natural corrosion; the other is through electro-accelerated corrosion. Natural corrosion generally takes a comparative longer time to expose RC structure to natural corrosion-inducing environment until the corrosion level reaches the designated level. Coastal environment is often selected because of its high chloride concentration. Another method is to carry out electro-accelerated corrosion, which was favoured by researchers, like Shi *et al* (2000), Ma *et al* (2012) and Gong *et al* (2004). The electrochemical nature of corrosion process makes it possible to apply external high voltage current to accelerate corrosion. It could help reach the corrosion level in months, which normally takes years. The Faraday law is used to predict the corrosion rate. This method has proven to be an effective and efficient way to corroded RC manually. Generally researchers tested these corroded RC component under cyclic static loading. The following will demonstrate experiments on electric-corroded RC column and joint and the general conclusions.

3.1 Experiment Research on Corroded RC Beam

3.1.1 Experiment research conducted by Castel *et al* (2000)

Castel *et al* (2000) conducted a long-term corrosion experimental study program on corroded RC beams which were 3000mm in length and with a rectangular cross section of 150 mm by 280 mm. The columns were reinforced with 6 mm transverse stirrup at a spacing of 220 mm and four 12 mm diameter longitudinal rebar. The program was started in the year of 1984 and continued for 13 years. In the experiment two beams were kept in man-made confined salty mist and two beams

were stored in lab conditions (Castel *et al.* 2000). All beams were subject to service loading in order to imitate the corrosion environment of RC structures in service. Experimental results indicated that when the compressive zone of RC structural component was not seriously corrosion damaged, the residual bearing capacity was only affected by the cross section area loss of tensile rebars. Comparative analysis between the two groups of beams revealed that corroded beam yielded larger deflection and correspondingly 35% of stiffness loss. Furthermore, the ductility of corroded beams exhibited significantly drop because of local corrosion of tensile rebars. Castel A. also studied the coupling effect of rebar cross section area reduction and bond degradation (Castel *et al* 2000). It was found that the two factors were not working alone and had exceedingly strong coupling effect on the global behaviour of corroded beam. The loss of bond can result in less tensile contribution from concrete and more tensile demand on steel rebar which could greatly contribute to the increase of tensile stress in steel rebar at the notch point, which can lead to post yield when other parts are still in elastic range.

3.1.2 Experiment research conducted by El Maaddawy *et al* (2005)

El Maaddawy *et al* (2005) studied the coupling effect of corrosion attack and service loading. Nine beams with a size of $152 \times 254 \times 3200$ mm were included in the experiment. Concrete mixture contains 2.25 percent of chloride ion by weight of cement to simulate chloride penetration. Eight of them are stored in misty confined environment containing sufficient oxygen and moisture, and further exerted with direct current to speed up corrosion reactions. Four of the corroded beams are imposed with 60 percent of their yield load during their corrosion exposure time, which ranges from 50 days to 310 days. In the experiment it has been found that flexural cracking caused by service load can increase the corrosion rate especially at the early stage, promote the generation of corrosion cracking, and raise the maximum corrosion crack width. Also the flexural capacity of beams is almost proportionally reduced by the corrosion level of steel rebar. It has been concluded that the effect of flexural cracking on improving the corrosion level becomes lessened when the corrosion exposure time increases. The

test results indicate that beams with great rebar weight loss exhibit appearance of pit corrosion, which seriously reduces the flexural capacity. However, comparison analysis concludes that the influence of pit corrosion on flexural capacity could be weakened by sustained loading on the beams.

3.1.3 Experiment research by Xia *et al* (2011)

Shear capacity of corroded reinforced concrete beam was investigated by Xia *et al* (2011) who had imposed constant current on stirrup and thus had achieved different corrosion levels in stirrup and longitudinal rebar. Eighteen beams with the dimension of $120 \times 230 \times 1200$ mm were manufactured, among which fifteen beams were corroded using accelerated wet-dry cycle corrosion method. In the cycle, ends of beams are wrapped with sponge which holds 5% sodium chloride solution and applied with current density of 100 uA/cm^2 for 4 days, after the beams are dried for 3 days. Corrosion-induced cracking was recorded. Four point monotonic loading tests were carried out on the all beams. Experiment results demonstrate that average corrosion-induced crack width increase as corrosion level goes up and maximum crack width increases faster than average crack width. Loading test indicates that corroded beam has reduced stiffness when being applied with 20-30% of its maximum load. Shear capacity could be reduced by the cross section area loss of steel rebar, the average crack width and maximum crack width of concrete cover, among which the average crack width has the biggest impact. In the loading test it has been observed that highly corroded beam fail by the fracture of stirrup. This experiment had realistically simulated the corrosion of beam, cover cracking was achieved, and the influence factors of corroded beam on shear capacity have been investigated.

3.1.4 Experiment research by Ou and Chen (2014)

Ou and Chen (2014) had studied the seismic performance of RC beam with corroded transverse reinforcement. Specimens are designed according to ACI 318

Code. Each specimen has one anchorage block end which is used for fixture, while the other end is free end used for exerting cyclic loading. The beam has a dimension of $300 \times 500 \times 1850$ mm. And it has 1.5 % of longitudinal reinforcement ratio and 1.8% of volumetric transverse reinforcement ratio. The transverse hoops at the plastic hinge region are linked with direct DC power supplies so as to confine corrosion at the most critical area. In the accelerated corrosion experiment, corrosion-induced cracking is constrained to occur in transverse direction since the longitudinal rebars are isolated from corrosion. Average weight loss of transverse hoop ranges from 0% to 35%. For the most highly corroded beam, pit corrosion is so severe that transverse hoop breaks at several points. Cyclic loading is exerted on the free end of beams with drift ratio from 0% to 6%. The envelope curves of the tests indicate that corrosion has insignificant effect on maximum force. But it also reveals that 6% corrosion level is a threshold value to cause detrimental decrease of seismic resistance. Deformation capacity and ductility could be reduced dramatically by increasing corrosion level over 6% due to the undermined confinement to concrete core. Corroded hoop also lose the ability to constrain the development of shear cracking. Corrosion-induced crack and loading-induced cracking could work together to cause severe concrete spalling, which leads to decreasing of shear resistance. Furthermore, analysis has concluded that shear resistance should be estimated by considering both the average corrosion level and the minimum cross section area of transverse rebar.

3.2 Experiment Research on Corroded RC Column

3.2.1 Experiment research conducted by Shi *et al* (2000)

Shi *et al* (2000) carried out cyclic loading test on corroded column, and investigated how the axial loading and the corrosion level would affect the seismic performance (Shi and Niu 2000, Niu *et al.* 2005, Chen *et al.* 2005). Ten identical specimens were constructed. The cross section of column was 200 mm by 200 mm, and height was 1100 mm, resulting in an aspect ratio of 5.5. The boundary for the

column was prismatic with the dimension of 250 mm × 300 mm × 1100 mm. The columns were reinforced by six longitudinal bars with diameter of 14 mm and stirrup with diameter of 6 mm at the spacing of 80 mm. The boundary element, which represented the discontinuity, was reinforced by six longitudinal bars with diameter of 16 mm and stirrup which was the same as that of the column. The compressive strength of the concrete was 20.1 MPa. The yield strength of the longitudinal bar was 415.6 MPa and the stirrup bar would yield at the strength of 327.0 MPa. In order to best simulate the corrosion process in the real environment and carry out load test afterward, and prevent the bond failure at the discontinuities, the boundary element and the column top were covered with Paraffin. And the specimens were exposed to outdoor environment for two and a half years. And then they were immersed in NaCl Solution with the concentration of 5%. The current was charged for seven days, subsequently drawn out the liquid. Then the specimens were kept in outdoor natural environment without accelerated corrosion for seven days. Then repeated the procedure until the calculated corrosion levels had been reached. The actually corrosion level was calculated by weighing the bars which had been cleared of corrosion products after the loading test. The columns were exerted on reversed cyclic loading while a constant axial load was applied on the top. The axial load ratios included 0, 0.2 and 0.4 to research on how axial load affected the response. More detailing of the specimen is shown in Appendix A. The following conclusions were drawn from the test.

- 1) The extent of damage on the column depends on the duration of accelerated corrosion. The three major causes of corrosion damage include the deterioration of mechanical properties of rebar, the concrete cover cracking, and the degradation of the bond strength.
- 2) The increase of corrosion level would exacerbate the degradation of the yield strength, the stiffness, the ductility, the energy dissipation.
- 3) The corrosion of rebar could change the failure characteristic of the RC columns. Compared with the sound RC column, corroded column demonstrated more apparent signs of brittle failure.
- 4) The impact of corrosion on the strength, stiffness, ductility, energy dissipation of corroded RC columns was similar to that of sound columns.

3.2.2 Experiment research conducted by Gong *et al* (2004)

Gong *et al* (2004) had conducted experimental research on lateral cyclic behaviour of sixteen corroded columns subjected to constant eccentric compressive loading (Gong *et al.* 2004, Li *et al.* 2009). The experiment produced performance results of the columns, with regard to ultimate capacity, stiffness, ductility, and energy dissipation capacity. It had been found out that the accumulative energy dissipation capacity would decrease from 11.5 kN.m to 1.5 kN.m and ductility ratio decline from 6.29 to 3.96 as the reinforcement corrosion level increased from 8% to 19.8%. The hysteretic loops had an apparent form of fusiform and asymmetry. Moreover the transition of failure mode from ductile failure to brittle mode had been observed. The phenomenon that stirrup underwent more severe corrosion attack than longitudinal reinforcement conformed to the real life inspection results of the engineering structures. More details about the specimens were tabulated in Appendix A. Li and Gong had done future researches on using FRP and steel jacket to retrofitting the corrode specimens. The following general conclusions were derived from the analysis of test results:

- 1) The corrosion of reinforcement had contributed to the change of failure modes of columns under constant eccentric axial load and lateral cyclic loadings. The reduction ratio of ultimate strength, stiffness, ductility, and energy dissipation of corroded columns relies on the corrosion level of reinforcement.
- 2) The non-uniformity of the reinforcement corrosion had brought about the asymmetry of hysteretic loop. The severity of stirrup corrosion decreased confinement behavior and shear capacity, thus exacerbating the degeneration of the ductility of the specimens.

3.2.3 Experiment research conducted by Wang and Liang (2008)

Wang and Liang (2008) researched on the mechanical behaviour of reinforced concrete columns with partial length corrosion. Twelve columns were partially corroded in tensile or compression zone with certain length before being

eccentrically loaded. The specimen was shown in Appendix A. Through the test, two major discoveries had been made as follows.

- 1) Longitudinal and transverse reinforcement of RC columns with partial length corrosion would transform the performance under eccentric loading. The test showed the columns had reduced ultimate load and brittle failure behavior. Compared to columns with partial corrosion in the compressive zone, columns with corrosion in tensile zone had greater reduction ratio of load capacity when under large eccentric loading. And columns with tensile zone partially corroded experienced greater load capacity reduction under small eccentric loading.
- 2) Within the partial corrosion length, the crack resulting from loading had emerged with longitudinal corrosion cracks. The corrosion of stirrups deteriorated the confinement at the corroded zones. These greatly reduced the mechanical integrity of the column and created a weak zone within the whole length of the column, which significantly reduced the strength of the columns.

3.2.4 Experiment research conducted by Ma *et al* (2012)

Ma *et al* (2012) had investigated on how reinforcement corrosion would change seismic behaviour of circular columns. Thirteen RC circular columns were included in the experiment. The columns were electro-accelerated corroded on different levels ranging from 0% to 15.1%, after which the columns were subject to constant axial load and cyclic lateral displacement. Ma Y. had proposed different equations for loading capacity, stiffness, ductility, and energy dissipation of the columns based on the experiment results. The equations established linear relationship between the seismic behaviour of the columns with corrosion levels, as is shown below.

$$F_y = F_{y0}(1 - 0.885\rho)$$

$$F_u = F_{u0}(1 - 0.878\rho)$$

$$\bar{\eta} = \bar{\eta}_0(1 - 1.182\rho)$$

$$\mu = \mu_0(1 - 1.795\rho)$$

$$E_{sum} = E_{sum0}(1 - 4.789\rho)$$

Where F_y , F_u , $\bar{\eta}$, μ is the yield strength, the ultimate strength, the average normalized stiffness and ductility of columns with corrosion level ρ ; F_{y0} , F_{u0} , $\bar{\eta}_0$, μ_0 is the yield strength of uncorroded column. Furthermore, the following conclusions were drawn.

- 1) Highly corroded RC columns under high axial load experienced brittle failure in the test. Corrosion could change the failure behavior of columns under mild axial load.
- 2) Poor hysteretic response, reduced stiffness, degraded ductility and steep descending branch in envelop curve were observed in columns with axial load ratio as high as 0.4 and corrosion level from 10% to 20%.
- 3) Columns with corrosion loss ratio below 14% showed increased cumulative energy dissipation as the ultimate displacement increased. They had almost the same energy dissipation capability with the uncorroded columns.

3.3 Experiment Research on Corroded RC Joint

3.3.1 Experiment research conducted by Dai and Yuan (2005)

Dai and Yuan (2005) had carried out cyclic reverse loading tests on exterior beam-column connection. Two connection specimens were electric-accelerating corroded. Then the same loading history was exerted on all the specimens. Test results of the corroded joints were compared against the control uncorroded specimen. It had been found out that bond strength deterioration and concrete valid cross section reduction would seriously degrade the seismic capacity of exterior beam-column joints. The ratio is 1:3. Each specimen was composed of beam with cross section of 100mm $\times$ 200mm, and column with cross section of 150 mm $\times$ 150 mm. More details are shown in Appendix A. Other general conclusions were as follows.

- 1) Flexural crack would develop and connect with the corrosion crack as a consequence of the reversed loading action. The concrete was subdivided by these cracks, which resulted in serious cover spalling.
- 2) At the connection the spalling of concrete and slip of reinforcement bars

caused the rebar to pop out, demonstrating the characteristic of shear failure.

- 3) Corrosion of reinforcement would result in bar area decrease and concrete valid cross section reduction, as well as the bond strength degradation between concrete and longitudinal bars, which all contributed to the decline of stiffness and seismic strength. The deformation capacity of the corroded beam-column joints was seriously reduced.

3.3.2 Experiment research conducted by Liu *et al* (2010)

Liu *et al* (2010) had conducted research on the seismic behaviour on corroded beam joints. His experiment included five specimens which were designed according to common frame design specifications. Based on the actual corrosion rate, the corrosion levels for each beam joints specimen were 0%, 3.19%, 5.77%, 9.12% and 12.21% respectively. Cyclic loading were exerted on the specimens with the objectives to find out how the corrosion level affected the failure mode, hysteretic loop, and steel stress-strain curve. Based on the results, Liu *et al* had put forward formula to calculate the strength and ductility of beam joint with the consideration of corrosion damage. The test concluded that:

- 1) Before the test, the corrosion cracks were seen in the specimens. During the test, more rapid and wider crack grow were observed in the plastic hinge of joint beam as the corrosion level increased.
- 2) The area enveloped by the hysteretic loop was reduced by the increase the corrosion level. At the high corrosion level, the load-displacement curve was changed from fat robust shape to thin unstable S shape, which clearly demonstrated the decrease of energy dissipation capacity. The pinching effect was very apparent in highly corroded specimens.
- 3) The strength and stiffness of joint beam were reduced by the corrosion, for which there are three major reasons. Firstly, non-uniform corrosion had made pits on the bar, which could give rise to stress concentration. Secondly, corrosion had reduced the cross section area of rebars. Thirdly, the corrosion products expanded and caused the cracking of cover concrete, which results in the reduction of valid cross section of the RC.

3.4 Summary

The chapter has reviewed on major experimental studies on the corrosion damaged reinforced concrete column, joint and beam. Accelerated corrosion or natural corrosion methods had been employed to generate corrosion on steel rebar. Different kinds of loading test had been conducted on different types of RC elements. The test results have demonstrated that the reason for the mechanical degradation of corroded RC element lies in three kinds of material deteriorations, corrosion-induced concrete cracking, the reduction of rebar cross section and the bond strength at the interface between concrete and rebar.

Corrosion-damaged element has smaller stiffness and strength. Besides, corrosion could have a major negative impact on the energy dissipation and ductility of column under axial compression and lateral loading. It also discovered that corrosion of steel rebar have the potential to change the failure modes of column from ductile to brittle. In general corrosion is non-uniform and localized in morphology. The localization of corrosion leads to asymmetry of hysteretic loop and reduction of seismic resistance.

The strong coupling effect of rebar cross section area reduction and bond degradation has been studied by testing on corroded RC beams (Castel *et al.* 2000). The corrosion-induced bond loss can result in less tensile contribution from concrete and the increase of tensile stress in steel rebar. El Maaddawy, T. found sustained load will increase the corrosion at the early stage and pit corrosion is the main reason for the reduction of flexural capacity when the corrosion level is high (El Maaddawy *et al.* 2005). Xia *et al.* discovered that shear capacity could be reduced most effectively by the average crack width and highly corroded beam fail by the fracture of stirrup (Xia *et al.* 2011). Ou and Chen had concluded that corroded hoop also lose the ability to constrain the development of shear cracking (Ou and Chen 2014).

Partial length corrosion has detrimental effect on the performance of column under eccentric loading (Wang and Liang 2008). Compared to columns with partially corrosion damaged compressive zone, columns with corrosion in tensile zone had greater reduction ratio of load capacity when under large eccentric loading, while columns with tensile zone partially corroded experienced greater load capacity reduction under small eccentric loading. Test results imply that the cause is that corroded stirrups deteriorated the confinement behaviour at the corroded zones, which results in damage of the mechanical integrity and creation of weak zones within the whole length of the column.

Experiments have shown that corrosion-damaged joint could seriously reduce deformation capacity because of bond loss, valid concrete area loss, and rebar area loss (Dai and Yuan 2005). Flexural crack would develop and join with the corrosion crack as a result of the cyclic loading action. Furthermore it causes serious cover spalling at the joint. Results indicate cover spalling and rebar slip could cause brittle shear failure to corrosion-damaged joint. As the corrosion increases, the load-displacement curve of joint under cyclic loading was changed from fat robust shape to thin unstable S shape, which clearly demonstrated the decrease of energy dissipation capacity. The pinching effect was very apparent in highly corroded joints.

Literature review on corroded RC element experiments and the above discussion has insinuated that the corrosion of transverse rebar affects mechanical confinement on concrete (Castel *et al.* 2000, Dai and Yuan 2005, Wang and Liang 2008, Xia *et al.* 2011, Ou and Chen 2014). Transverse rebar has great significance on confined concrete strength and stress-strain relationship, which closely relates to the performance of RC element. Transverse rebar is the key element for providing shear capacity for RC element (Xia *et al.* 2011). Corroded transverse reinforcement has reduced cross section area, which results in decreased confinement force to concrete. Furthermore, pitting corrosion on transverse rebar can leads to yielding only at some localized point and thus decrease of deformation capacity and loss of ductility of concrete (Higgins and Farrow III 2006). But even

though there are many mature models to calculate the stress-strain relationship of confined concrete, none have taken into account the corrosion of steel rebar. There is a lack of experimental research on confinement behaviour of corroded transverse rebar. Experimental study also shows that corrosion is more than just rebar cross section reduction. The localization of corrosion could cause major changes in the performance. Therefore, this research will aim at investigating the residual strength and stress-strain relationship of corrosion damaged confined concrete.

CHAPTER 4

EXPERIMENTAL STUDY

ON CONFINED CONCRETE STRENGTH WITH CORRODED STIRRUPS

4.1 Introduction

Research experiments have found transverse reinforcement of RC structure to be more vulnerable to corrosion attack. The concrete cover depth and diameter of transverse reinforcement are smaller than that of the longitudinal reinforcement. The smaller depth of concrete cover makes the transverse steel bar more easily reached by chloride and other corrosive chemicals. Furthermore, the diameter of transverse rebar is smaller and thus more sensitive to the corrosion. In RC structures, transverse rebar corrosion turns out to be a more prevalent problem. Transverse rebar is essential to confine concrete, increasing strength and ductility of RC element. However, limited research had been conducted on how corroded transverse rebar would change the confinement behaviour on concrete. Even though intensive research efforts had been focused on the stress-strain relationship of confined concrete and several well-established models had been proposed, no experimental studies and no models have included the influence of corrosion. Therefore, there is a need to investigate how corroded transverse rebar affects the confined concrete experimentally. As such, this chapter covers experimental study on residual confined concrete strength with corroded rectilinear transverse reinforcement.

4.2 Details of Confined Concrete Specimens

A total of eight confined concrete specimens are constructed to investigate how corroded stirrup will affect the loading capacity of column, the confined concrete strength and stress-strain relationship. Details of specimen are illustrated in Figure 4.1. The parameters of each specimen are shown in Table 4.1. The specimen is in

prismatic shape with square cross section of 200×200 mm and height of 600 mm. The clear cover depth is 10 mm. Each specimen is longitudinally reinforced with four deformed rebars D10 with a diameter of 10 mm. The spacing of the transverse stirrup is 65 mm in the middle and decreased to 25 mm at the ends which extends 100 mm. And all the transverse stirrups are plain bar R6 with a diameter of 6 mm. The more condense reinforced ends are designed to ensure the failure will occur in the middle region. As is seen in Figure 4.3, transverse stirrup in the middle region is welded to two small steel bars, with the purpose to supply the direct current to stirrup for accelerated corrosion. It has to be noted that these two bars are not connected with the three stirrups at each end. This is to ensure that the ends are protected from accelerated corrosion and can be used for exerting load. Furthermore, the longitudinal rebar is wrapped with electric tape at the corners where it contacts transverse stirrups. Hence, the current running in stirrups can be contained within stirrups in the most possible way. Therefore, when the current run through the two special steel rebars to transverse hoops in the middle region, the longitudinal rebar D10 could be insulated from electronic accelerated corrosion.

The typical stress-strain curves of the uncorroded deformed rebar D10 and the uncorroded round plain rebar R6 are shown in Figure 4.2. The yield strength of steel rebar D10 is 521 MPa, while the yield strength of steel rebar R6 is 425 MPa. The concrete mix is designed with target characteristic strength of 30 MPa. The maximum aggregate size is 10 mm and the cement is ordinary Portland cement (OPC) in Singapore. The water is mixed with 5% of sodium chloride NaCl by the weight of cement so as to make chloride-contaminated concrete, just as is in experiments by Wang et al (2011), Li et al (2009), Ou & Chen (2014), etc. It has the same concentration as the solution for accelerated corrosion experiment which will be explained in the following section so as to minimize infiltration due to different concentration. The mix has a water to cement ratio of 0.58. In the mixture the amount of cement is 402 kg/m³, water is 229 kg/m³, the fine aggregate is 956 kg/m³, the coarse aggregate is 748 kg/m³, as is shown in Table 4.2. All specimens are casted horizontally and for 28 days for curing. It is kept within appropriate

range of temperature (25 °C ~ 40 °C) and covered with plastic sheeting to prevent loss of water. Afterwards, specimens will be merged in sodium chloride solution and be connected with DC power suppliers. Six standard cylinders with 100 mm in diameter and 200 mm in height are made with the same batch of concrete as that for casting specimens. They are kept until the last day when all the columns specimens are being tested. The mean value of the compressive strength f'_c of six cylinders is 29 MPa, which is acceptable even though it is 1 MPa lower than the target strength

Figure 4.1: Detailing of specimen

Figure 4.2: Stress-strain curve of steel rebar D10 and R6

Table 4.1: Details of confined concrete specimens

Specimen	Longitudinal reinforcement				Transverse reinforcement		
	Number and diameter of bars	Ratio ρ_l (%)	Yield strength of D10 (MPa)	Yield strength of R6 (MPa)	Diameter and spacing	Ratio ρ_s (%)	Yield strength (MPa)
AM0	4D10 2R6	1.037	521	425	R6@65	1	425
AM1	4D10 2R6	1.037	521	425	R6@65	1	425
AM2	4D10 2R6	1.037	521	425	R6@65	1	425
AM3	4D10 2R6	1.037	521	425	R6@65	1	425
AM4	4D10 2R6	1.037	521	425	R6@65	1	425
AM5	4D10 2R6	1.037	521	425	R6@65	1	425
AM6	4D10 2R6	1.037	521	425	R6@65	1	425
AM7	4D10 2R6	1.037	521	425	R6@65	1	425

Table 4.2: Concrete mix proportion

Cement	402	kg/m ³
Water	229	kg/m ³
Fine aggregate (sand)	956	kg/m ³
Coarse aggregate	748	kg/m ³

4.3 Accelerated Corrosion Program

The natural corrosion of steel is an electro chemical process, thus manually exerting direct current on the steel can accelerate the process rate great times faster than natural corrosion. Accelerated corrosion has been an efficient method of obtaining corrosion damaged reinforced concrete structure in recent years (Niu *et al.* 2005, Wang and Liang 2008, Ou *et al.* 2013, Ou and Chen 2014). In this experiment, seven specimens are soaked in fiber reinforced plastic tank which holds 5% NaCl solution, as depicted in Figure 4.4. Specimen AM0 is kept outside for controlled trial. External direct current (DC) power supply with a maximum voltage of 60 V and a maximum current intensity of 3 A is used to provide the current. The DC power supply possesses built-in ammeter and voltmeter to timely gauge the current intensity and voltage values, which are displayed on the monitors. As illustrated in the Figure 4.3 and Figure 4.4, the positive side of DC power supply was connected to steel stirrup, which acts as anode, while the negative side was linked with copper plate acting as cathode. The top ends of specimens are above the water level, so they can retain higher strength for compressive test afterwards. In the tank, two basic semi-cell reactions are happening, as follows:

Figure 4.3: Schematic plan of accelerated corrosion program

Figure 4.4: Accelerated corrosion program setup

In order to control the corrosion procedures for each specimen, the duration for applying current was predicted on the basis of Faraday's Second Law, which sheds

light on how to calculate the mass loss by the number of electrode. The weight loss level can be estimated by:

$$\omega = \frac{W_{\text{loss}}}{m_0} 100\% = \frac{A_m I t}{Z \cdot F \cdot m_0} 100\% \quad (4-1)$$

where ω is the weight loss level of the steel rebar (in percentage %),

A_m is Atomic mass of iron (56g/mol),

I is the corrosion current (A, 1A = 1C/s),

t is the time (in second),

Z is the valency (Z is taken as 2, assuming most of rust product is due to $\text{Fe}(\text{OH})_2$),

F is the faraday's constant (96485 C/mol),

m_0 is the original mass of steel rebar.

So by transform unit of the time into hour and the current intensity into ampere, the weight loss of the rebar is as:

$$W_{\text{loss}} = \frac{A_m I t}{Z \cdot F} = 1.045 I t \text{ (g)} \quad (4-2)$$

where t is in Hour,

I is in Ampere.

Even though the accelerated corrosion rate is determined by the current density (Malumbela *et al.* 2012), Equation 4-2 indicates that the final weight loss level is determined the product of current intensity and duration $I_{\text{corr}} T$. Current density is calculated by dividing current intensity by surface area of rebars. Suppose the corrosion rate will remain constant when current density keeps unchanged. Therefore, constant current intensity is to be applied between anode and cathode, controlled by DC power supply, so as to ensure constant density on transverse stirrup and the accuracy of weight loss estimation. The values are checked on a regular basis and any drift is corrected immediately. Table 4.3 demonstrates the estimated corrosion level of transverse stirrup given current density of

500 $\mu\text{A}/\text{cm}^2$. Corrosion level could reach 5% when stirrup rebar is electrified for 168 hours.

Table 4.3: Designed current density and corrosion duration

Specimen	Designed Current density on transverse stirrup, ($\mu\text{A}/\text{cm}^2$)	Designed Corrosion Days T, (day)	Target corrosion level of transverse hoop by average weight loss, ω_{est} (%)
AM0	0	0	0
AM1	500	7	7.45
AM2	500	14	14.90
AM3	500	21	22.35
AM4	500	28	29.80
AM5	500	35	37.25
AM6	500	42	44.70
AM7	500	49	52.15

4.4 Loading Test Setup and Instrumentation

When accelerated corrosion of each specimen completed, the specimen was extracted from tank and dried in the natural laboratory environment for at least a week before being tested under axial loading. This action aims at letting the water out, which would otherwise change the strength and produce inaccurate results. All confined concrete specimens are to be tested until failure under concentric axial compression. The Instron Loading Machine with a compression capacity of 5000 kN and tension capacity of 2500 kN is used to exert the axial compression load.

As shown in Figure 4.5 and 4.6, Linear Variable Differential Transducer (LVDT) is installed to test the axial deformation. The test region is 200 mm long and located in the middle of the specimen. Four LVDTs are fixated on two square aluminium hoops, which are screwed in column specimen. Two strain gauges are attached on the surface of the column to measure the strain of the concrete cover.

A 5 mm thick steel plate is placed on top of the column so as to ensure the compression load is uniformly distributed on a flat surface. The loading rate is 0.5mm/min. The axial deformation data is collected by digital data logger during the loading test.

Figure 4.5: The installment of LVDT

Figure 4.6: Loading test setup

4.5 Corrosion Level Assessment

Even though DC power supplies have the functionality of stabilizing, the current intensity and voltage still fluctuates. So current was monitored and fixed at an interval of 6 hours. The current duration was recorded for corrosion level estimation. Table 4.4 displays the actual current density and duration for each specimen according to recording and calculations.

Table 4.4: Actual current density and corrosion duration

Specimen	Actual current density ($\mu\text{A}/\text{cm}^2$)	Actual corrosion duration T, (hour)	Estimated corrosion level of transverse stirrup by average weight loss, ω_{est} (%)
AM0	0	0	0
AM1	290	185	4.76%
AM2	508	216	10.0%
AM3	523	382	16.3%
AM4	520	548	24.0%
AM5	516	790	35.2%
AM6	476	958	42.3%
AM7	436	1030	45.1%

Figure 4.7(a): The appearance of AM5 and AM6 just after taking out of tank

Figure 4.7(b): The appearance of AM7 just after taking out of tank

Figure 4.7(a) and Figure 4.7(b) shows Specimen AM5, AM6, AM7 right after the end of the accelerated corrosion duration and being taken out of the tank. The rust appear yellow mixed with green in colour, which proves that the rust is combination of FeO and Fe_2O_3 . Rust leaking out from concrete cracks are observed, as shown in Figure 4.7(a) and (b). Besides, AM7 has fractured stirrups and lost one longitudinal rebar. A large section of concrete at the corner has fallen off because of cover spalling caused by corrosion product.

After the corrosion experiment is finished, the concrete of each column is knocked off and all rebars are fetched. Figure 4.8 shows the stirrups are numbered as $AMx - T1$ to $AMx - T7$ from top to bottom in middle region, in which x denotes the specimen index number and the ranking of corrosion duration. Hence the stirrups in the 200 mm LVDT test region are $AMx-T6$, $AMx-T7$ and $AMx-T8$. Stirrups in 100 mm top ends are $AMx-TE-T1$, $AMx-TE-T2$, $AMx-TE-T3$, while stirrups in the 100 mm bottom end are named as $AMx-BE-T1$, $AMx-BE-T2$, $AMx-BE-T3$. Longitudinal rebars are cut for the only part within the middle region and the longitudinal deformed rebar is denoted as $AMx-Ly$, in which y denotes the numbering of longitudinal rebar. Figure 4.9(a)~(g) shows the appearance of the stirrups in the test region of corroded specimens. Note that in this thesis the stirrups mean the stirrup in the test region by default.

Figure 4.8: The illustration of the numbering of rebars from top to bottom

Figure 4.9(a): The appearance of stirrups AM1-T1~T7

Figure 4.9(b): The appearance of stirrups AM2-T1~T7

Figure 4.9(c): The appearance of stirrups AM3-T1~T7

Figure 4.9(d): The appearance of stirrups AM4-T1~T7

Figure 4.9(e): The appearance of stirrups AM5-T1~T7

Figure 4.9(f): The appearance of stirrups AM6-T1~T7

Figure 4.9(g): The appearance of stirrups AM7-T1~T7

Then the rust of the rebars were cleaned with steel brush, conforming to ASTM G1 (G1-03 2003). Afterwards immerse the rebars into 5% hydrochloride acid HCl for 20 minutes before being soaked in sodium hydroxide NaOH to neutralize the remaining acid on rebars. When rebars are dried, the remaining weight m_r is weighted. In the thesis, without any special annotations average weight loss level means the ratio of reduced weight to original weight, which is calculated by the following equation (Kashani *et al.* 2013).

$$\omega = \frac{W_0 - W_r}{W_0} 100\% \quad (4-3)$$

where W_0 is the original mass of rebar before corrosion

W_r is the residual mass after being corroded and then being washed and cleaned.

Table 4.5 displays the residual weight of each rebar in all specimens after test under axial loading and being cleaned of rust. Table 4.6(a)-(b) shows weight loss

and nominal uniform residual cross section of each rebar in Specimen AM3 and AM6. Weight loss of rebar in others specimens is tabulated in Appendix B. Note that for those incomplete stirrups, it is assumed that the missing part has same average weight loss as the remaining part. The corrosion level of each stirrup and each longitudinal rebar is not the same, which is attributed from the different chloride content and current density at different locations. Besides it could be seen that the top three stirrups have smaller corrosion level than other stirrups. The top three stirrups in the upper end were above the water level, so less chloride and water makes it unfavourable for steel corrosion reactions. Further, the three stirrups at top and bottom ends are detached from the longitudinal rebar R6 so they are not under direct current exertion. Consequently the top and bottom end both suffer from less corrosion attack, which makes them suitable for applying loading. Table 4.7 lists the average corrosion level in top end, bottom end, and middle region of all specimens. As is shown, the average weight loss level of the stirrups in the top and bottom end regions of Specimen AM3 is 3.94 % and 16.29 %, while the weight loss level of stirrups in middle region is 17.49%. Table 4.8 shows the average weight loss level of stirrup and longitudinal rebar of all specimens. Presented in Figure 4.10, the weight loss of longitudinal rebar is comparatively much lower than that of stirrup for column specimens, except AM6 and AM7. The reason may lie in the improper wrapping of insulation tape, which results in the leak of current from stirrup to longitudinal rebar. For AM1-AM5, the weight loss level is around 5%, which proves that the method used in this experiment to obtain corrosion-damaged stirrup is valid and can be further developed for future experimental research. Figure 4.11 shows the comparison between estimated weight loss and actual weight loss. When the corrosion time is less than 550 hours and the weight loss level is less than 25%, the estimation is accurate with accepted variation. However, when corrosion time extends longer, the corrosion rate tends to slow down and the estimation overestimates the real weight loss. Therefore, the assumption that corrosion rate is constant may not hold water when corrosion level is high.

Table 4.5: Residual Weight of Stirrups and Longitudinal Rebars in Specimens

Name	AM0	AM1	AM2	AM3	AM4	AM5	AM6	AM7
-	(g)	(g)	(g)	(g)	(g)	(g)	(g)	(g)
TE-T1	170.90	172.74	171.29	169.46	160.05	155.65	165.57	161.48
TE-T2	171.93	171.78	172.92	168.30	170.56	168.19	162.96	169.12
TE-T3	169.87	169.32	168.98	163.70	169.11	170.20	156.13	165.69
T1	164.40	164.85	155.10	147.29	141.94	130.21	117.06	124.01
T2	166.64	165.12	156.47	141.29	136.78	123.67	120.66	117.37
T3	165.20	164.37	156.44	142.91	140.38	135.68	126.33	114.38
T4	165.45	166.37	158.86	147.26	137.39	130.65	118.29	126.47
T5	168.61	165.51	158.31	142.80	136.35	134.57	119.61	115.82
T6	166.44	167.24	158.14	141.69	141.29	122.88	118.39	122.75
T7	165.64	165.42	155.58	141.68	147.29	127.60	122.65	118.29
BE-T1	167.29	170.18	162.34	152.85	163.70	158.53	126.93	111.23
BE-T2	167.70	169.57	163.10	136.59	168.30	150.73	126.25	122.12
BE-T3	166.72	169.98	157.54	147.52	169.46	158.48	138.42	133.82
L1	365.00	365.00	351.07	359.88	356.40	353.30	351.20	323.40
L2	366.00	363.00	344.04	353.52	352.00	358.40	310.80	353.10
L3	365.00	363.00	350.59	356.82	355.60	344.00	329.60	322.25
L4	365.50	365.00	351.56	353.36	342.40	331.78	351.60	321.80

Table 4.6(a): Weight loss level of stirrup and longitudinal rebars in AM3

Name	residual weight, w_r	Weight loss level, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM3-TE-T1	169.46	2.61	27.52
AM3-TE-T2	168.30	3.28	27.33
AM3-TE-T3	163.70	5.92	26.59
AM3-T1	147.29	15.35	23.92
AM3-T2	141.29	18.80	22.95
AM3-T3	142.91	17.87	23.21
AM3-T4	147.26	15.37	23.92
AM3-T5	142.80	17.93	23.19
AM3-T6	141.69	18.57	23.01
AM3-T7	141.68	18.57	23.01
AM3-BE-T1	152.85	12.15	24.83
AM3-BE-T2	136.59	21.50	22.18
AM3-BE-T3	147.52	15.22	23.96
AM3-L1	359.88	1.40	77.40
AM3-L2	353.52	3.15	76.03
AM3-L3	356.82	2.24	76.74
AM3-L4	353.36	3.19	75.99

Table 4.6(b): Weight loss level of stirrup and longitudinal rebars in AM6

Name	residual weight, w_r	Weight loss level, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM6-TE-T1	165.57	4.85	26.93
AM6-TE-T2	162.96	6.35	26.50
AM6-TE-T3	156.13	10.27	25.39
AM6-T1	117.06	32.72	19.04
AM6-T2	120.66	30.66	19.62
AM6-T3	126.33	27.40	20.55
AM6-T4	118.29	32.02	19.24
AM6-T5	119.61	31.26	19.45
AM6-T6	118.39	31.96	19.26
AM6-T7	122.65	29.51	19.95
AM6-BE-T1	126.93	27.05	20.64
AM6-BE-T2	126.25	27.44	20.53
AM6-BE-T3	138.42	20.45	22.51
AM6-D-L1	351.20	3.78	75.53
AM6-D-L2	310.80	14.85	66.84
AM6-D-L3	329.60	9.70	70.89
AM6-D-L4	351.60	3.67	75.62

Table 4.7: Average weight loss level of stirrups in different regions of Specimens

Name	ω_{top}	ω_{bot}	ω_{mid}	ω_{test}	Number of ruptured stirrups
AM0	0	0	0	0	0
AM1	1.78	3.89	4.85	4.93	0
AM2	1.69	7.47	9.78	9.27	0
AM3	3.94	16.29	17.49	17.06	2
AM4	4.27	3.94	19.42	20.67	3
AM5	5.36	10.40	25.68	23.2	5
AM6	7.15	24.98	30.79	30.23	5
AM7	4.92	29.66	31.11	31.67	7

Note:

ω_{top} denotes the average weight loss level of the three stirrups in the top end,

ω_{bot} denotes the average weight loss level of the three stirrups in the bottom end,

ω_{mid} denotes the average weight loss level of the seven stirrups in the middle region.

ω_{test} denotes the average weight loss level of the three stirrups in the LVDT test region

Table 4.8: Average weight loss level of rebars in the Specimens.

Name	Stirrup	Longitudinal rebar
-	ω_t	ω_l
Unit	(%)	(%)
AM0	0.00	0.00
AM1	4.93	2.60
AM2	9.27	4.30
AM3	17.06	2.49
AM4	20.67	3.67
AM5	23.20	4.97
AM6	30.23	8.83
AM7	31.67	9.55

Figure 4.10(a): Average weight loss of stirrup in the LVDT test region of specimens

Figure 4.10(b): Average weight loss of longitudinal rebars in the LVDT test region of specimens

Figure 4.11: Estimated weight loss versus real weight loss

Apart from estimating the corrosion status by weight loss, the localization of stirrup corrosion is investigated by measuring the pit depth on the surface of the stirrup. Each stirrup is divided into eight sections, as is illustrated in Figure 4.12. The most highly corroded point in each section of stirrup is identified firstly by carefully examination. Afterwards, the remaining diameter is measured by callipers and pit depth is determined by subtracting the original diameter by the remaining minimum distance between the two surfaces, as demonstrated in Figure 4.13. Then the overall maximum value and the average value of the eight pits are calculated. Note some stirrups have missing parts. The overall maximum pit depth of the incomplete stirrup equals the diameter of stirrup rebar, so as to consider the locality of the whole stirrup rebar.

Figure 4.12: The illustration of the divide of the region of the stirrup

Figure 4.13: Illustration of measuring pit depth

Table C.1-C.7 and Figure C.1-C.7 in Appendix C shows the pit depth of each stirrup in each specimen. Figure 4.14(a)-(d) illustrates the comparison of maximum pit depth, average pit depth, and nominal pit depth on each stirrup in the middle region of Specimen AM1, AM3 and AM5. The nominal pit depth is obtained by multiplying the diameter of stirrup by the weight loss level. It has been shown that average pit depth has little difference with the nominal difference. The widening gap between maximum pit depth curve and nominal pit depth curves demonstrates that the differences increases, as the weight loss level rises. The nominal average pit depth, which is calculated from the weight loss of each bar, makes an indication of the nominal residual cross section of steel rebar. Localized corrosion of Specimen AM4-AM7 is more severe than that of Specimen AM1-AM3.

Figure 4.14(a): Comparisons of pit depth on Stirrups in Specimen AM1

Figure 4.14 (b): Comparisons of pit depth on Stirrups in Specimen AM3

Figure 4.14(c): Comparisons of pit depth on Stirrups in Specimen AM5

Figure 4.14(d): Overall maximum pit depth of stirrups in middle region of each specimen

As the corrosion level increases, more ruptured rebars appear and the difference between maximum pit depth and average pit depth becomes larger, as is demonstrated in Figure 4.15. Many researchers had found the importance of catching the non-uniformity of cross section area and localized corrosion (Higgins and Farrow III 2006, Ou and Chen 2014). In fact, localization level can be measured by the deviation of maximum pit depth from average pit depth, expressed as:

$$\delta = \frac{p_{max} - p_{ave}'}{p_{ave}'} \quad (4-4)$$

where δ is the localization level,

p_{max} is the maximum pit depth,

p_{ave}' is the nominal pit depth, $p_{ave}' = \omega d$

Figure 4.16 and Table 4.9 display the localization level against stirrup weight loss level. It shows that stirrup with small weight loss has the possibility to possess a high localization level. The weight loss level of stirrups in AM3 is around 20%, but the localization level reaches up to 2, most as the same as column Specimen AM6 which is about 30% corroded. Since it is based on the maximum pit depth and the nominal average pit depth, this new parameter could sufficiently describe the extent of corrosion localization. It could be used as another parameter apart from weight loss level to measure the corrosion level.

Figure 4.15 Differences between maximum pit depth and nominal pit depth
and weight loss level

Table 4.9: Average localization level of
stirrups in each specimen

Specimen	localization level
unit	-
AM0	0
AM1	1.679
AM2	1.126
AM3	2.255
AM4	2.335
AM5	3.802
AM6	1.707
AM7	2.234

Figure 4.16: Localization level versus weight loss level of stirrups in specimens

4.6 Observed Behavior and Compression Test Results

The behavior of eight columns under axial compression loading was observed and photographed using digital camera. Failure of RC columns were characterized by cover concrete cracking, concrete spalling, stirrup rebar yield, stirrup rebar fracture, longitudinal rebar yield, and longitudinal rebar buckling. Figure 4.17-20 shows the appearance of Specimen AM0, AM3, AM6 and AM7 at four different stages of loading, namely before loading, the maximum compression load F_{max} , ultimate failure at load $0.3F_{max}$, and after taking off the LVDT installments,

Specimen AM0 has 1% transverse reinforcement ratio and 1.04% longitudinal reinforcement ratio and no rebars are being corroded. Figure 4.17(a) shows its appearance before testing. No rust was observed on the surface, while other specimens all had rust in yellow, due to the seeping of corrosion products. Figure 4.17(b) illustrates that when the maximum compressive load was just reached, one major crack on the left edge had been generated from the bottom vertically extending upwards, one inclined crack on the right edge and cover spalling had started to develop. Figure 4.17(c) depicts when the column was in the failure stage,

large cracks had been observed and concrete crushing had been more severe than at the maximum load stage. Figure 4.17(d) shows the specimen when LVDT instrumentation and slack cover concrete had been removed. Cracks were observed in the concrete core by careful examination. After knocking off the concrete, investigating on steel age had discovered the buckling of longitudinal rebars. Specimen AM0 demonstrates the characteristic of typical reinforced concrete column.

The appearance of Specimen AM3 in different stages is shown in Figure 4.18(a)-(d). Before being tested under the axial loading, AM3 had been sunk in sodium chloride solution and exerted with direct current density of $500 \mu A/cm^2$ for 382 hours. Corrosion level of stirrups in middle region was 17.49% while top and bottom ends were 3.94 % and 16.29 %. The localization level of stirrup in AM3 had reached up to 2.26, and fracture due to corrosion had been found on stirrups, as is shown in Figure 4.18. Furthermore deformed longitudinal rebars and plain longitudinal rebars had 2.49 % and 11.03% corrosion level. Figure 4.18(a) illustrates a yellow strip where a lot of corrosion substance had leaked out from two small holes and one vertical crack, which can be found by careful investigation. The crack is believed to be caused by corrosion products. Figure 4.18(b) shows that two major vertical cracks had developed about 10 mm away from the edge of the surface. The left crack had run through the rusty strip. Compression loading had further developed the corrosion-induced cracking, which can be confirmed by Figure 4.18(c) and 4.18(d). Two major cracks had extended from top to bottom. Figure 4.18(d) shows that a large chunk of cover concrete had split off the core and three stirrups had corrosion-induced fracture. Specimen AM6, as is shown in Figure 4.19(a)-(d), is another example to demonstrate the behavior of columns with highly corroded stirrup. As same as Specimen AM3, large vertical cracks at the edge appeared when the maximum load was reached. Large parts of cover spalling were apparent at the surface where stirrups rupture, as is displayed in Figure 4.19(c) and 4.19(d). High-level corrosion of stirrup has accumulated large quantity of iron substance between concrete cover and concrete core, resulting in that crack develops from one stirrup bar outwards, even connect with crack of

other stirrup rebar and eventually cause cover spalling, as is shown in Figure 2.3 of Chapter 2. The corrosion-induced cracking is aggravated by axial loading.

Specimen AM7, illustrated in Figure 4.20(a)-(d), had 31.11% corroded stirrups in the test region. Obvious corrosion-induced cover spalling was also observed at one edge, and the middle part of one longitudinal rebar with length of about 400 mm was missing, as is shown in Figure 4.20(a). Corrosion-induced cover cracking had made the stirrup rebar under so severe corrosion attack that all the stirrups had fractured. The longitudinal rebar was believed to suffer from high level weight loss at the junction joint with stirrup, which led to the fall off of the longitudinal rebar. The remaining three deformed longitudinal rebars and two plain rebars had an average corrosion level of 8.83% and 15.04%. Top end had severe cover spalling and had lost a whole stirrup, which can be found out in Figure 4.20(c). Figure 4.20(b) shows that the specimen had less cracks and unobvious cover spalling than other specimens when the maximum compressive load was reached. At the end of the testing, a crack was found running in the middle at the top end, show in Figure 4.20(d). The missing of one longitudinal rebar in the 400 mm middle region had made the square column weaker at one side and subject to bending when the compressive loading was concentric. Furthermore, highly corroded stirrup had caused the column to be insufficiently reinforced. So cover cracking and yielding of longitudinal rebar were not fully developed, and column Specimen AM7 displayed brittle failure in the loading program.

Figure 4.17: Specimen AM0, (a) Before loading test, (b) At the maximum load F_{max} , (c) At load $0.3F_{max}$, (d) After taking off LVDT and loose concrete

Figure 4.18: Specimen AM3, (a) Before loading test, (b) At the maximum load F_{max} , (c) At load $0.3F_{max}$, (d) After taking off LVDT and loose concrete

Figure 4.19: Specimen AM6, (a) Before loading test, (b) At the maximum load F_{max} , (c) At load $0.3F_{max}$, (d) After taking off LVDT and loose concrete

Figure 4.20: Specimen AM7, (a) Before loading test, (b) At the maximum load F_{max} , (c) At load $0.3F_{max}$, (d) After taking off LVDT and loose concrete

Figure 4.21: Appearance of specimens with corroded stirrup after testing

Figure 4.21(a)-(f) illustrates the appearance of all specimens after the loading test was completed. Careful examination found that the buckling of longitudinal rebars occurred in all the specimens. Cover cracking in different degrees was observed respectively. Apart from Specimen AM7, other specimens experienced major cover spalling in the middle region, where the spacing of stirrup is 65 mm.

The deformation of the LVDT test region is measured by LVDT and recorded by data logger. Figure 4.22 shows load and axial deformation curves of specimens. Table 4.9 presents maximum compression load $F_{\max}$, and the displacement at $0.85F_{\max}$, $0.5F_{\max}$, $0.3F_{\max}$. Maximum compress load represents the residual compression capacity of the corrosion damaged columns. As demonstrated in

Figure 4.23(a) and 4.23(b), the residual compressive capacity reduces with the increase of the weight loss level of both stirrup and longitudinal rebar. Multilinear regression analysis on test data could establish a relationship between compressive capacity and weight loss level of stirrup and longitudinal, as shown in Figure 4.23(c). It is expressed as:

$$F_{\max r} = F_{\max} - 7.5\omega_t - 6.9\omega_l - 0.4\omega_t\omega_l \quad (4-5)$$

where $F_{\max}$ is the original compressive load capacity

$F_{\max r}$ is the residual compressive load capacity of column with corroded stirrup

ω_t is the weight loss level of stirrup (%)

ω_l is the weight loss level of longitudinal rebar (%)

In this experiment, weight loss level of longitudinal rebars in AM1-4 is around 5% and in AM5-6 is about 9%, much lower than that of stirrups. And both have the same trend as corrosion time increases. The localization level is a parameter which could take into account pit depth on rebars, so it could indicate the residual cross section of highly corroded rebars. Figure 4.23 presents the relationship between residual compressive capacity and the weight loss level. Figure 4.24 displays the relationship between residual compressive capacity and the localization level. The comparison of Figure 4.23 and Figure 4.24 indicates that the changing trend of compressive capacity of columns is more closely related to the weight loss level than localization level as the corrosion deteriorates. Figure 4.25 displays the relationship between stirrup weight loss level and axial deformation characteristics. All specimens reach the maximum compressive load at the same axial deformation. However, the slope of descending branch of curves is growing sharper as the weight loss level and localization level arises. Besides, deformation at 50% and 30% of the maximum compressive force has a clear decreasing trend as the stirrup weight loss increases, which affirms the deformation capacity of the columns is reduced by the corrosion.

Figure 4.22: The axial deformation and load curves

Table 4.10: Compression Capacity of Specimens

Specimen	Maximum load	Displacement at $F_{\max}$	Displacement at $0.85F_{\max}$	Displacement at $0.5F_{\max}$	Displacement at $0.3F_{\max}$
	$F_{\max}$	D_1	$D_{0.85}$	$D_{0.5}$	$D_{0.3}$
unit	KN	mm	mm	mm	mm
AM0	1276	0.630	1.800	2.990	8.010
AM1	1241	0.846	1.245	3.255	9.423
AM2	1159	0.590	1.498	3.993	6.420
AM3	1147	0.691	1.418	2.851	5.323
AM4	1083	0.694	1.501	4.204	5.593
AM5	956	0.995	1.930	3.093	4.766
AM6	992	0.460	0.895	1.365	6.476
AM7	766	0.373	0.783	1.478	1.228

Figure 4.23(a): Relationship between maximum compressive load and stirrup weight loss level

Figure 4.23(b): Relationship between axial deformation and longitudinal rebar weight loss level

Figure 4.23(c): Relationship between axial deformation and stirrup and longitudinal rebar weight loss level

Figure 4.24: Relationship between maximum compressive load and stirrup localization level

Figure 4.25: Relationship between axial deformation and stirrup weight loss level

4.7 Stress-Strain Curve of Confined Concrete with Corroded Stirrup

The residual confined concrete strength with corroded stirrup is obtained through analysing the experimental results of confined concrete column under concentric compressive loading. The contribution of the longitudinal rebars and cover concrete is deducted from the total applied load (Mander *et al.* 1988). The cover concrete of the specimen has suffered from corrosion-induced cover-cracking in different degrees. The strength of the cracked concrete is calculated based on softening theory (Hsu 1992, Ou & Chen 2014) by introducing a reduction factor β , which is expressed as Equation 4.6 and 4.7. This factor β is based on the assumption that the volume of corrosion products clustered in the cracks makes up 25% of steel rebar volume loss (Ou and Chen 2014).

$$\beta = \frac{0.9}{\sqrt{1 + 600\varepsilon_r}} \quad (4-6)$$

$$\varepsilon_r = 0.25 \frac{\omega/r_s}{p_{cp}sC_c} \quad (4-7)$$

where β is the reduction factor for cracked cover concrete

ω is the weight loss level,

r_s is the specific weight of the steel

p_{cp} is the perimeter of the cross section

s is the stirrup spacing

C_c is the clear cover depth

The force in longitudinal bar is computed by the uniform residual cross section and the stress. The deformation when the buckling of longitudinal rebars happened is used to calculate the buckling strain. The stress in that rebar will be zero after the strain exceeds that strain in the calculation in order to consider the reduction due to buckling. And at other strain, the stress in corrode steel bar is determined by the stress-strain curve from the tension test, and the yield strength of the rebar

is calculated based on the relationship by Du et al (2005), as is shown in Equation 4.8:

$$f_{yc} = (1 - 0.48\omega)f_{y0} \quad (4-8)$$

where ω is the weight loss level (%)

f_{y0} is the yield strength of the original uncorroded rebar

f_{yc} is the yield strength of corroded rebar

Figure 4.26 depicts the stress-strain curves of confined concrete with corroded transverse rebar. Specimen AM1 has moderate weight loss level of 4.93%, so the confined strength changes slightly compared with that of AM0, which have no corrosion. Specimen AM2, AM3 and AM4 had weight loss level of 9.3%, 17.1% and 19.4% and localization level of 1.126, 2.255 and 2.335. As it can be seen, specimen AM2, AM3 and AM4 have experienced more obvious reduction in confined strength. The weight loss and the pit depth in these specimens are so prominent as to significantly affect its enhancement effect on the core concrete. The stirrups become less effective in holding the expansion of the concrete core when it's under compression. However, specimen AM5 and AM6 both have highly corroded stirrup over 35% and corrosion-induced fracture in the LVDT test region. The ratio of residual strength to $0.85f_{ccr}'$ has dropped to 1.09, showing axial behaviour with unconfined concrete. Specimen AM7 has one missing longitudinal rebar, so the axial deformation is not uniform on four sides and brittle failure mode is observed from the test observation and the stress-strain curve. The column failed under compressive and bending load. The non-uniformity has contributed to the confined concrete strength of 20 MPa, which is lower than axial concrete strength $0.85f_{ccr}'$. Therefore, column AM7 will not be considered as a valid specimen for the axial loading test with the purpose to derive confined concrete strength with corroded stirrup. The effect of corroded stirrup on confined concrete strength can be divided into two aspects, one is the reduced confined strength and the other is the slope of the descending branch of the curve, as is illustrated in Figure 4.27 and Figure 4.28. As weight loss level and localization increases, the confined concrete strength declines and strain at $0.5f_{cc}$ and $0.3f_{cc}$

also decreases greatly, which indicates the slope of the descending branch is shaper. Based on linear regression analysis of the data points, the relationship between the residual confined concrete strength with stirrup weight loss level ω can be expressed as:

$$f_{ccr}' = f_{cc}' - 0.278\omega \quad (4-9)$$

where f_{cc}' is original confined concrete compressive strength

f_{ccr}' is the residual confined concrete strength with corroded stirrup

ω is the weight loss level (%)

Figure 4.26: Stress-strain curves of confined concrete with corroded stirrup

Table 4.11: Stress-Strain of Confined Concrete

	Confined strength	Confined strain	Enhancement ratio	Strain at $0.85f_{cc}$	Strain at $0.5f_{cc}$	Strain at $0.3f_{cc}$
	f_{cc}	ϵ_{cc}	$f_{cc}/0.85f'_c$	$\epsilon_{0.85}$	$\epsilon_{0.5}$	$\epsilon_{0.3}$
Specimen	MPa	mStrain	-	-	-	-
AM0	35.168	0.328	1.43	0.01017	0.01750	0.03630
AM1	34.129	0.272	1.38	0.00562	0.01826	0.03500
AM2	31.728	0.338	1.29	0.00745	0.01320	0.03795
AM3	31.632	0.345	1.28	0.00778	0.01362	0.02135
AM4	30.892	0.347	1.25	0.00767	0.01299	0.02074
AM5	26.986	0.454	1.09	0.01011	0.01526	0.01834
AM6	26.747	0.328	1.09	0.00826	0.01355	0.02304
AM7	20.066	0.187	0.81	0.00338	0.00515	0.00619

Figure 4.27: Relationship between confined concrete strength and stirrup weight loss level

Figure 4.28: Relationship between confined concrete strain and stirrup weight loss level

Figure 4.29: Relationship between confined concrete strength and stirrup localization level

Extensive experimental studies on confined normal strength concrete and high strength concrete have yielded well-established analytical models for confined concrete strength and stress-strain curves. But these models maybe inadequate for confined concrete with corroded stirrup by reducing the stirrup cross section and yield strength. There exists great necessity to assess the most dominant confined concrete models against the experimental results.

(1) Mander Model (Mander et al. 1988)

Mander and Priestley had put forward stress-strain equation by introducing the effective confining stress based on the configuration of longitudinal rebar and transverse rebars (Mander et al. 1988). The equation for calculating the maximum strength f'_{cc} of square confined concrete as follows:

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + 7.94 \alpha_s \frac{f_l}{f'_{co}} - 2 \alpha_s \frac{f_l}{f'_{co}}} \right) \quad (4-10)$$

$$f_l = \frac{1}{2} k_e \rho_s f_{yh} \quad (4-11)$$

where f'_{co} is the strength of unconfined concrete

k_e is the confinement effective factor

f_l is the maximum lateral pressure of transverse reinforcement

ρ_s is the transverse rebar ratio in the square concrete core

f_{yh} is the yield strength of the transverse rebar

α_s equals $(21.2 - 0.35 f'_{co}) \frac{f'_l}{f'_{co}}$ for normal strength concrete

The confinement effective factor k_e is the ratio of the effective confined concrete to the total concrete core, like:

$$k_e = \frac{A_e}{A_c(1 - \rho_{cc})} \quad (4-12)$$

For square concrete section with square hoops, the confinement effective factor is as following (Mander et al. 1988):

$$k_e = \frac{\left(b_c^2 - \sum_{i=1}^n \frac{(w'_i)^2}{6} \right) \left(1 - \frac{s'}{2b_c} \right)^2}{b_c^2(1 - \rho_{cc})} \quad (4-13)$$

where b_c is the side length of the square concrete core

w'_i is the i th clear distance between longitudinal bars

s' is the clear distance between the transverse rebar

ρ_{cc} is the longitudinal reinforcement ratio in the concrete core

(2) Iyengar Model (Iyengar *et al* 1970)

Iyengar, Desayi and Reddy tested confined concrete specimens and proposed confinement index (Iyengar *et al.* 1970), which is defined as:

$$C_c = (\rho_s - \bar{\rho}_s) \frac{f_{yh}}{f'_{co}} \quad (4-14)$$

where ρ_s is the volumetric transverse reinforcement ratio of the square confined concrete, $\bar{\rho}_s$ is the volumetric transverse reinforcement ratio of the equivalent least confined lateral dimension. The maximum strength and the strain of the square confined concrete core with stirrup can be rewritten in the following equations (Iyengar *et al.* 1970, Dodd 1992):

$$f'_{cc} = f'_{co}(1 + 0.4C_c) = f'_{co} \left(1 + 0.4 \left(1 - \frac{s}{d_s} \right) \frac{\rho_s f_{yh}}{f'_{co}} \right) \quad (4-15)$$

$$\epsilon_{cc} = \epsilon_{co}(1 + 3.5C_c) = \epsilon_{co} \left(1 + 3.5 \left(1 - \frac{s}{d_s} \right) \frac{\rho_s f_{yh}}{f'_{co}} \right) \quad (4-16)$$

(3) Muguruma Model (Muguruma & Watanabe, 1990)

Muguruma and Watanabe proposed the equation of maximum confined concrete by introducing a confining coefficient (Muguruma and Watanabe 1990, Dodd 1992), which is calculated by:

$$C_c = \rho_s \frac{\sqrt{f_{yh}}}{f'_{co}} \left(1 - \frac{s}{2b_c} \right) = \frac{2}{\sqrt{f_{yh}}} \frac{f_l}{f'_{co}} \left(1 - \frac{s}{2b_c} \right) \quad (4-17)$$

The maximum confined concrete strength is defined as:

$$f'_{cc} = f'_{co}(1 + 47C_c) = f'_{co} \left(1 + \frac{94}{\sqrt{f_{yh}}} \frac{f_l}{f'_{co}} \left(1 - \frac{s}{2b_c} \right) \right) \quad (4-18)$$

The strain at the maximum stress is:

$$\varepsilon_{cc} = \varepsilon_{cc}(1 + 457C_c) = \varepsilon_{cc} \left(1 + \frac{914}{\sqrt{f_{yh}}} \frac{f_l}{f'_{co}} \left(1 - \frac{s}{2b_c} \right) \right) \quad (4-19)$$

(4) Li Model (Li *et al* 2001)

Li *et al* had (2001) conducted quasi-static axial loading test on normal and high strength concrete columns with transverse rebars with normal yield and high yield strength. A stress-strain model has been proposed:

$$f_c = \begin{cases} E_c \varepsilon_c + (f'_{co} - E_c \varepsilon_{co}) \frac{\varepsilon_c^2}{\varepsilon_{co}^2} & \text{when } 0 \leq \varepsilon_c \leq \varepsilon_{co} \\ f'_{cc} - (f'_{cc} - f'_{co}) \frac{(\varepsilon_c - \varepsilon_{co})^2}{(\varepsilon_{cc} - \varepsilon_{co})^2} & \text{when } \varepsilon_{co} \leq \varepsilon_c \leq \varepsilon_{cc} \\ f'_{cc} - \beta f'_{cc} \frac{\varepsilon_c - \varepsilon_{cc}}{\varepsilon_{cc}} \geq 0.4f'_{cc} & \text{when } \varepsilon_{cc} \leq \varepsilon_c \leq \varepsilon_{cu} \end{cases} \quad (4-20)$$

where ε_{cu} is the compressive strain of confined concrete when the first fracture of transverse hoop occurs.

Based on regress analysis on test results and Mander's model, the maximum confined concrete strength is proposed to be:

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + 7.94 \alpha_s \frac{f'_l}{f'_{co}}} - 2 \alpha_s \frac{f'_l}{f'_{co}} \right) \quad (4-21)$$

where $\alpha_s = (21.2 - 0.35f'_{co}) \frac{f'_l}{f'_{co}}$ for normal strength concrete

$f'_l = k_e f_l$ for square confined concrete

The strain at the maximum concrete strength and the ultimate strain for square confinement and normal strength concrete and rebar are defined as:

$$\varepsilon_{cc} = \varepsilon_{co} \left(1 + 11.3 \left(\frac{f'_l}{f'_{co}} \right)^{0.7} \right) \quad (4-22)$$

$$\varepsilon_{cu} = \varepsilon_{co} \left(2 + (122.5 - 0.92f'_{co}) \sqrt{\frac{f'_l}{f'_{co}}} \right) \quad (4-23)$$

Li (1994) also proposed a parameter β of 0.2 to control the slope of the descending branch of the stress-strain curve of normal strength concrete and rebar.

(5) Fafitis Model (Fafitis and Shah 1985)

Fafitis and Shah suggested the maximum confined concrete model and the corresponding strain of square confined concrete (Fafitis and Shah 1985), as follows:

$$f'_{cc} = f'_{co} + \left(1.15 + \frac{21}{f'_{co}}\right) f_l \quad (4-24)$$

$$\varepsilon_{cc} = 1.49 \times 10^{-5} f'_{co} + 0.0296 \frac{f_l}{f'_{co}} + 0.00295 \quad (4-25)$$

where the confinement pressure of square core is defined as:

$$f_l = \frac{A_{sh} f_{yh}}{d_e s} \quad (4-26)$$

where d_e is the equivalent diameter of the square section and equals the side length of the square section, A_{sh} is the total cross section of the stirrups.

Figure 4.30(a): Comparison between experimental and predicted stress-strain curves of column specimen AM0

Figure 4.30(b): Comparison between experimental stress-strain curves of corroded confined concrete and analytical curve by Mander Model

(6) Proposed Model

Figure 4.30(a) shows the comparison between the analytical stress-strain curves predicted by different models and experimental result of uncorroded column specimen AM0. All models could predict the ascending branch with good accuracy. In the analytical curve by Iyengar model, stress descends faster than the experimental curve, while in analytical curve by Fafatis model stress descends slower. Mander model and Muguruma model demonstrate the most satisfactory agreement on the confine stress, confined strain, and descending branch with the experimental curve. Figure 4.30(b) displays stress-strain curves of corroded column specimens, which demonstrates that neither models can yield good result for confined concrete with corroded stirrups. Corrosion-damaged confined concrete has degraded confined strength and increased curve slope in the descending branch.

Mander model is derived from theoretical analysis of confinement behavior and has been verified against many experimental programs, so it could provide guidance for corrosion-damaged confinement model. In Mander model, confined concrete strength depends on

the effective lateral compression provided by transverse reinforcement (Mander *et al.* 1988). Corroded transverse rebar has reduced cross section area and decreased yield strength (Du *et al.* 2005), which can be determined by the linear relationship with the weight loss level. Besides, a correction factor should be introduced to quantify the reduction of confinement effective coefficient caused by localized corrosion and bond degradation (Chung *et al.* 2008), etc. Therefore, the proposed equation of the maximum lateral pressure can be expressed as:

$$f_{lc} = \frac{1}{2} \eta k_e \rho_s f_{yh} (1 - \omega) (1 - 0.48\omega) \quad (4-27)$$

where η is the confinement correction factor of confined concrete with corroded stirrup

f_{lc} is effective lateral pressure of corroded stirrup, ω is the weight loss level of corroded stirrup

The residual confined strength f'_{ccr} has the same form with original equation (Mander *et al.* 1988), represented as :

$$f'_{ccr} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + 7.94 \alpha_s \frac{f_{lc}}{f'_{co}}} - 2 \alpha_s \frac{f_{lc}}{f'_{co}} \right) \quad (4-28)$$

Based on Mander model (Mander 1983, Mander *et al.* 1988), the strain at the peak stress of the corroded confined concrete is calculated by the following equation by introducing correction factor ζ :

$$\varepsilon_{ccr} = \varepsilon_{co} \left(1 + 5\zeta \left(\frac{f'_{ccr}}{f'_{co}} - 1 \right) \right) \quad (4-29)$$

where ε_{co} is the strain of unconfined concrete

ε_{ccr} is the strain of confined concrete with corroded stirrup

ζ is the correction factor for confined strain

Mander (Mander *et al.* 1988) had also proposed the stress-strain relationship model of the confined concrete. The control parameter of the curve is r . The stress-strain curve of corrosion-damaged confined concrete has steeper descending branch. Hence, correction factor ψ is proposed as follows:

$$f_c = f'_{cc} \frac{x r_c}{r_c - 1 + x r_c} \quad (4-30)$$

$$r_c = \psi \frac{E_c}{E_c - \frac{f'_{ccr}}{\epsilon_{cc}}} \quad (4-31)$$

where x equals $\frac{\epsilon_c}{\epsilon_{ccr}}$

ψ is the correction factor for curve slope

Table 4.12 displays correction factors η , ζ , ψ for effective lateral pressure, confined strain and curve slope, which are obtained from test data of the confined concrete strength of all specimens. Presented in Figure 4.31(a)-(b), correction factor η for effective lateral pressure is more related to weight loss level rather than localization level. The linear relationship between correction factor and weight loss level could be represented as:

$$\eta = 1.0057 - 0.0159\omega \quad (4-32)$$

where η is the correction factor for the effective lateral pressure

ω is the weight loss level of corroded transverse rebar (%)

As shown in Figure 4.31(c)-(d), correction factor ζ for confined strain is not linearly related with weight loss level and localization level. Polynomial fitting analysis discovers that factor ζ could be expressed as the second order polynomial equation of localization level:

$$\zeta = -0.613\delta^2 + 4.033\delta - 2.55 \quad (4-33)$$

where δ is the localization level based on pit depth

ζ is the correction factor for confine strain

Illustrated in Figure 4.31(e)-(f), relationship between curve slope correction factor ψ and weight loss level or localization level can't be expressed by linear fitting line with acceptable accuracy. The correction factor is better estimated by polynomial equation, expressed as:

$$\psi = -0.223\delta^2 + 1.307\delta - 0.204 \quad (4-34)$$

where δ is the localization level based on pit depth

ψ is the correction factor for curve slope

Table 4.12: Correction Factors

Specimen	Weight loss level	Localization level	Lateral Pressure Correction factor	Confined Strain Correction Factor	Curve Slope Correction Factor
	ω	δ	η	ζ	ψ
AM0	0	1	1	1	1
AM1	4.93	1.68	0.965	0.98	1.03
AM2	9.27	1.13	0.855	1.33	1.25
AM3	17.06	2.26	0.734	3.25	2.05
AM4	19.42	2.34	0.625	4.05	1.93
AM5	25.68	3.80	0.614	3.86	1.85
AM6	30.79	1.71	0.542	3.55	1.55

Figure 4.31(a): Correction Factor η versus stirrup weight loss level

Figure 4.31(b): Correction factor η versus stirrup localization level

Figure 4.31(c): Correction factor ζ versus stirrup weight loss level

Figure 4.31(d): Correction factor ζ versus stirrup localization level

Figure 4.31(e): Correction factor ψ versus stirrup weight loss level

Figure 4.31(f): Correction factor ψ versus stirrup localization level

As discussed above, original Mander model is unable to predict the axial stress-strain behavior of confined concrete with corroded stirrup. Hence correction factors have been put forward to quantify the variation caused by the corrosion of stirrup. These factors are corrected with the corrosion level stirrup, either weight loss level or localization level. However, the comparison between Figure 4.31(a)-(b), Figure 4.31(c)-(d), and Figure 4.31(e)-(f), indicates that the correction factors are more closely related to the weight loss level of stirrup instead of localization level. The proposed model is a modified Mander model with corrections factors for lateral pressure, confined strain and curve slope. Equation (4-28) demonstrates that the confined concrete strength is closely related to lateral pressure. So the correction factor for lateral pressure is the dominant one to determine the strength of confined concrete with corroded stirrup and thereof the stress-strain curve.

Illustrated in Figure 4.32(a)-(g), stress-strain curves by the modified model has a strong agreement with the experimental curves. Shown in Figure 4.32(a), the analytical curve by proposed model displays smaller initial stiffness. However, the ascending branch of experimental curve is well predicted by Mander model and Muguruma model, while the descending branch agrees more with analytical curve by proposed model. Displayed in Figure 4.32(b), 4.32(d) and 4.32(f), experimental curves of specimen AM2, AM4, and

AM6 are well matched with prediction curves in aspect of confined strength, confined strain and curve slope. Figure 4.32(c) and 4.32(e) presents that analytical curves by proposed model have smaller initial stiffness and west-predicted descending branch. Overall the proposed model could estimate the stress-strain relationship of confine concrete with corroded stirrup with good accuracy. However, more experimental studies shall be carried out to uncover how the effect of stirrup corrosion on residual confined strength varies due to different reinforcement ratio, different cylinder concrete strength, and different stirrup shape. More test data are needed to verify the proposed model in terms of maximum lateral pressure, confined strain and curve slope. And there is a lack of model to estimate the ultimate strain of corroded confined concrete.

Figure 4.32(a): Comparison between experimental curve and analytical curve by proposed model of Specimen AM1

Figure 4.32(b): Comparison between experimental curve and analytical curve by proposed model of Specimen AM2

Figure 4.32(c): Comparison between experimental curve and analytical curve by proposed model of Specimen AM3

Figure 4.32(d): Comparison between experimental curve and analytical curve by proposed model of Specimen AM4

Figure 4.32(e): Comparison between experimental curve and analytical curve by proposed model of Specimen AM5

Figure 4.32(f): Comparison between experimental curve and analytical curve by proposed model of Specimen AM6

Figure 4.32(g): Comparison between experimental curve and analytical curve by proposed model of Specimen AM7

4.8 Conclusion

This chapter describes an experimental study on the structural performance of columns with highly corroded stirrup under concentric compressive loading. The weight loss level of stirrup and longitudinal rebars was quantified. Pit depth of stirrups of specimen was measured for the proposed parameter localization level to quantify the extent of pit corrosion. The compressive capacity and residual confined strength of concrete with corroded square stirrup was analysed. Based on this experimental study, the following conclusions can be drawn:

1. The weight loss level of longitudinal rebar is much smaller than that of stirrups for most of the specimens. This experiment had successfully isolated the longitudinal rebars from the electronic current running in stirrups. Wrapping the longitudinal rebars at the junction joints with insulation tape and applying direct current to stirrup directly through another metal bar is an effective method of obtaining reinforced concrete specimen with corroded stirrup.

2. Based on pit depth on rebar, localization level is proposed to measure the degree of localised corrosion. Localised corrosion often appears in the way of smaller diameter at certain points of the rebar, creating pits of different depth. As the weight loss level increase, the localization level tends to increase, but one are not necessarily determined by the other. Localization level can be very high even though the weight loss level is comparatively low.

3. Concentric compressive loading tests on reinforced concrete column with corroded stirrup demonstrate that the effective compressive force drops with the increase of corrosion level of stirrup. Highly corroded columns could have greatly compromised compressive capacity and axial deformation capacity. Besides, the descending branch of compressive load and axial deformation tend to have increased slope.

4. Confined concrete strength is obtained by subtracting the contribution of cover concrete and longitudinal rebar from the total compressive force. The effect of transverse rebar corrosion on confined concrete strength is examined through analysing the strength and the stirrup weight loss. As the weight loss level increases, confined concrete strength decreases. A linear relationship is established between confined concrete strength and weight loss level. Mander

Model could not estimate the stress-strain curve of confined concrete with corroded stirrup.

5. Based on Mander Model, correction factors η , ζ , ψ for effective lateral pressure, confined strain, and curve slope parameter are proposed to better quantify the effect of corrosion, among which correction factor η is the most dominant one. Analysis on the factors reveals that η is linearly related with stirrup weight loss level, and ζ , ψ is better estimated by polynomial equation of localization level. The proposed model could estimate the stress-strain curve of the corroded confined concrete specimens with a good agreement with experimental curve. However, a lack of data on different transverse reinforcement ratio, different concrete strength, and different transverse rebar configuration must be noted. Future research program shall be done to verify the proposed model with more data points and greater variety.

CHAPTER 5

SUMMARY AND RECOMMENDATIONS FOR FUTURE RESEARCH

Corrosion has been one of the urgent issues that RC structures face in recent years. The corrosion of transverse reinforcement has significant effect on confinement behaviour. This thesis has been focused on investigating the stress-strain relationship of confined concrete. The achievements of the thesis are as follows:

Firstly, this thesis has covered detailed literature review on the material degradation caused by corrosion of steel rebar in reinforced concrete structures. The mechanism of steel corrosion in concrete is an electro chemical process, which is often accelerated by the ingress of aggressive chemicals. Chloride attack is the most dominant one to cause structure deficiency. Submerging reinforced concrete in Sodium Chloride solution and applying the direct current on steel bar makes a more efficient method to obtain corrosion-damaged concrete than natural corrosion. The accumulation of corrosion products on steel rebar is able to generate circular stress which could cause cover cracking. Cover cracking will reduce the cross section of the concrete component. Corrosion of steel rebar will degrade the physical appearance and reduce its original cross section. Corrosion often appears to be non-uniform and localized. Chloride attack could cause pit corrosion which is one of the prominent localized corrosion. Law of Diffusion provides a valid theory to predict the chloride concentration in concrete and thus offer theory to estimate the corrosion initiation time and corrosion level. Corrosion will also change the interaction between steel bar and concrete. Experimental research has found that bond strength could be seriously affected. Predicted equations have been put forward to estimate the bond strength degradation based on the corrosion level of rebar. This thesis also reviews experimental studies on the strength of corroded steel rebar. It has found out that corrosion could compromise the yield strength and ductility, especially for the highly corroded steel rebar. Based on regression analysis on the results, linear relationship had been

developed between the strength of steel rebars and average weight loss of steel rebar.

Secondly, this thesis has reviewed on experimental research on corrosion-damaged reinforced concrete structures. Many scholars have drawn valuable conclusions on how corrosion affects mechanical behaviour, strength and seismic capacity. Corrosion damaged RC column under vertical and horizontal loading displayed smaller yield strength, ductility, energy dissipation capacity. Besides, the corrosion of rebar could change the failure characteristic of the RC columns. Corroded column demonstrated more apparent signs of brittle failure. And the localization of the reinforcement corrosion had brought about the asymmetry of hysteretic loop. Stirrup corrosion decreased confinement on concrete and shear capacity, thus exacerbating the degeneration of the ductility of the specimens. Experimental study on reinforced concrete joint with corroded rebar discovered that crack grows faster and wider as the corrosion level rises, and even connect to the corrosion-induced crack and exacerbate cover spalling under cyclic load. As bond strength could be reduced by the increase of corrosion level, which results in the damage of the stiffness and deformation capacity. Hysteretic loop of corroded joint demonstrates the decrease of energy dissipation capacity and the increase of pinching effect. Review on testing of reinforced concrete beam with corroded rebars found that bond strength reduction can result in less tensile contribution from concrete and more tensile demand on steel rebar which can cause yield at the cracked point. Furthermore, shear capacity of beam could be reduced by the cross section area loss of steel rebar, the average crack width and maximum crack width of concrete cover.

Finally, this thesis covers the concentric loading test on eight reinforced concrete columns with corroded stirrup. Based on maximum pit depth and weight loss level, localization level is proposed to quantify the degree of pit corrosion. Wrapping the longitudinal rebars with electronic insulation tape at the junction joint could effectively isolate longitudinal rebar from electronic corrosion attack. As weight loss of rebar increases, pit depths appear bigger and the localization level tends to

rise consequently. However, results attest that localization level could be quite high even though the weight loss is still low. Besides, the descending branch of compressive load and axial deformation curve has steeper slope with the rise of weight loss level. And highly corroded columns could have greatly compromised the compressive capacity. Finally, confined strength of concrete core is obtained by deducting the contribution of cover concrete and longitudinal rebar. Experimental results prove that the increase of weight loss level could diminish confined concrete strength. The uniform reduction of cross section area can not sufficiently account for the effect of corrosion, so correction factors are introduced to better estimate confined concrete strength. Based on Mander Model (Mander *et al.* 1988), new model is proposed to predict the stress-strain curve of corroded confined concrete. Based on Mander Model, correction factors η , ζ , ψ for effective lateral pressure, confined strain, and curve slope parameter are proposed to better quantify the effect of corrosion. Analysis reveals that correction factor for effective lateral pressure f_{lc} is linearly related with stirrup weight loss level, while correction factor for confined strain and curve slope ζ , ψ is better estimated by polynomial equation of localization level. The proposed model could estimate the stress-strain curve of corroded confined concrete with good agreement with experimental curve.

Apart from the conclusions and achievements described above, the limitation of the experiment tests must be noted. Recommendations for future research are given as follows:

1. Even though the weight loss level of stirrup ranges from 5% to 31%, the spacing is 65 mm in the test region for all the specimens, which makes the transverse reinforcement ratio of uncorroded column specimen only 1%. It is a medium value. Great significance lies in finding out the confined concrete strength of more condense reinforced column with corroded transverse reinforcement.
2. The cross section of the reinforced concrete column is square in shape. Experiment study needs to unveil the effect of corrosion on circular column confined with spiral steel bar and square column confined with octagonal hoops.

3. High strength concrete and high strength deformed steel has been used populously in reinforced concrete structure. The effect of corrosion on confined concrete strength can vary in extent.

4. Correction factor has been proposed to quantify the additional influence of stirrup corrosion apart from cross section loss. However, extensive research needs to be carried out how the correction factors vary with different transverse reinforcement ratio, shape of transverse hoop, and the cylinder concrete strength. Relationships shall be established so the model of estimating confined concrete strength could be verified.

5. Existing models of stress-strain curve of confined concrete has not account for the influence of corroded rebar. As has been found out in Chapter VI, the descending branch of corroded confined concrete exhibit shaper slope, which indicates enhancement effect wanes and the stress decreases faster. Research needs to found out the analytical model to predict the stress-strain curve with corroded transverses rebar.

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APPENDIX A

Details of RC Specimens with Corroded Rebar in Literature

Figure A.1 Configuration of electric-accelerated corrosion experiment
(Niu *et al.* 2005)

Figure A.2: Dimension and reinforcement detailing of Niu, Shi and Wang's specimens
(Shi and Niu 2000, Niu *et al.* 2005)

Figure A.3: Dimension and reinforcement detailing of Gong's specimens
(Gong, Zhong et al. 2004)

Figure A.4: Dimension and reinforcement detailing of Wang X. H. 's specimens (Wang and Liang 2008)

Figure A.5: Dimension and reinforcement detailing of Ma's specimens (Ma, Che et al. 2012)

Figure A.6: Dimension and reinforcement detailing of Dai's specimens (Dai and Yuan 2005)

Table A.1: General material properties of experimental specimens before corrosion in literature

	Section type	Dimension	Clear cover depth	Longitudinal reinforce ratio	Transverse reinforce ratio	Cubic Concrete compressive strength	Yield strength of longitudinal bar	Yield strength of stirrup
Niu et al (2005) & Shi et al (2000)	Rectangular	200x200x1100	20	6d14 = 1.355%	d6@80 = 0.354%	31.1 MPa	415.6 MPa	327 MPa
Gong et al (2004)	Rectangular	200X200x1500	22	6d14 = 1.32%	d6.5@90 = 0.369%	27.9 MPa (C25)	335 MPa (HRB335)	235 MPa (HPB235)
Wang & Liang (2008)	Rectangular	200x200x700 (1100)	30	4d18 = 2.54%	d8@100 = 0.502%	30 MPa	397.5 MPa (f_u : 602.5 MPa)	402.5 MPa (f_u : 475 MPa)
Ma et al (2012)	Circular	D260x1000	20	6d16 = 2.27%	d8@100 = 0.386%	32.4 MPa	373.2MPa (f_u : 572.3 MPa)	402.5 MPa (f_u : 510.7 MPa)
Dai & Yuan (2005)	Joint	Shown in Figure A.6	20	Shown in Figure A.5	Shown in Figure A.5	2.8 MPa	352 MPa (d=12)	268 MPa
							372 MPa (d=14)	

APPENDIX B

Weight Loss of Stirrups and Longitudinal Rebars of Each Specimen

Table B.1: Weight loss level of stirrups and longitudinal rebars
in AM1

Name	residual weight, w_r	Weight loss, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM1-TE-T1	172.744	0.72	28.10
AM1-TE-T2	171.784	1.27	27.94
AM1-TE-T3	169.315	2.69	27.54
AM1-T1	164.846	5.26	26.81
AM1-T2	165.120	5.10	26.86
AM1-T3	164.367	5.54	26.73
AM1-T4	166.370	4.39	27.06
AM1-T5	165.512	4.88	26.92
AM1-T6	167.243	3.88	27.20
AM1-T7	165.421	4.93	26.90
AM1-BE-T1	170.180	2.20	27.68
AM1-BE-T2	169.574	2.54	27.58
AM1-BE-T3	169.981	2.31	27.65
AM1-L1	365	0.00	78.50
AM1-L2	363	0.55	78.07
AM1-L3	363	0.55	78.07
AM1-L4	365	0.00	78.50

Table B.2: Weight loss level of stirrups and longitudinal rebars in AM2

Name	residual weight, w_r	Weight loss, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM2-TE-T1	171.29	1.56	27.86
AM2-TE-T2	172.92	0.62	28.12
AM2-TE-T3	168.98	2.88	27.48
AM2-T1	155.10	10.86	25.23
AM2-T2	156.47	10.07	25.45
AM2-T3	156.44	10.09	25.44
AM2-T4	158.86	8.70	25.84
AM2-T5	158.31	9.02	25.75
AM2-T6	158.14	9.12	25.72
AM2-T7	155.58	10.59	25.30
AM2-BE-T1	162.34	6.70	26.40
AM2-BE-T2	163.10	6.26	26.53
AM2-BE-T3	157.54	9.46	25.62
AM2-L1	351.07	3.82	75.50
AM2-L2	344.04	5.74	73.99
AM2-L3	350.59	3.95	75.40
AM2-L4	351.56	3.68	75.61

Table B.3: Weight loss level of stirrups and longitudinal rebars
in AM3

Name	residual weight, w_r	Weight loss, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM3-TE-T1	169.46	2.61	27.52
AM3-TE-T2	168.30	3.28	27.33
AM3-TE-T3	163.70	5.92	26.59
AM3-T1	147.29	15.35	23.92
AM3-T2	141.29	18.80	22.95
AM3-T3	142.91	17.87	23.21
AM3-T4	147.26	15.37	23.92
AM3-T5	142.80	17.93	23.19
AM3-T6	141.69	18.57	23.01
AM3-T7	141.68	18.57	23.01
AM3-BE-T1	152.85	12.15	24.83
AM3-BE-T2	136.59	21.50	22.18
AM3-BE-T3	147.52	15.22	23.96
AM3-L1	359.88	1.40	77.40
AM3-L2	353.52	3.15	76.03
AM3-L3	356.82	2.24	76.74
AM3-L4	353.36	3.19	75.99

Table B.4: Weight loss level of stirrups and longitudinal rebars
in AM4

Name	residual weight, w_r	Weight loss, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM4-TE-T1	160.05	8.02	26.03
AM4-TE-T2	170.56	1.98	27.74
AM4-TE-T3	169.11	2.81	27.50
AM4-T1	141.94	18.42	23.09
AM4-T2	136.78	21.39	22.25
AM4-T3	140.38	19.32	22.83
AM4-T4	137.39	21.04	22.35
AM4-T5	136.35	21.64	22.18
AM4-T6	141.29	18.80	22.98
AM4-T7	147.29	15.35	23.96
AM4-BE-T1	163.70	5.92	26.62
AM4-BE-T2	168.30	3.28	27.37
AM4-BE-T3	169.46	2.61	27.56
AM4-L1	356.40	2.36	76.65
AM4-L2	352.00	3.56	75.70
AM4-L3	355.60	2.58	76.48
AM4-L4	342.40	6.19	73.64

Table B.5: Weight loss level of stirrups and longitudinal rebars in AM5

Name	residual weight, w_r	Weight loss, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM5-TE-T1	155.65	10.55	25.31
AM5-TE-T2	168.19	3.34	27.36
AM5-TE-T3	170.20	2.18	27.68
AM5-T1	130.21	25.17	21.18
AM5-T2	123.67	28.92	20.11
AM5-T3	135.68	22.03	22.07
AM5-T4	130.65	24.91	21.25
AM5-T5	134.57	22.66	21.89
AM5-T6	122.88	29.38	19.99
AM5-T7	127.60	26.66	20.75
AM5-BE-T1	158.53	8.89	25.78
AM5-BE-T2	150.73	13.38	24.51
AM5-BE-T3	158.48	8.92	25.77
AM5-L1	353.30	3.21	75.98
AM5-L2	358.40	1.81	77.08
AM5-L3	344.00	5.75	73.98
AM5-L4	331.78	9.10	71.36

Table B.6: Weight loss level of stirrups and longitudinal rebars in AM6

Name	residual weight, w_r	Weight loss, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM6-TE-T1	165.57	4.85	26.93
AM6-TE-T2	162.96	6.35	26.50
AM6-TE-T3	156.13	10.27	25.39
AM6-T1	117.06	32.72	19.04
AM6-T2	120.66	30.66	19.62
AM6-T3	126.33	27.40	20.55
AM6-T4	118.29	32.02	19.24
AM6-T5	119.61	31.26	19.45
AM6-T6	118.39	31.96	19.26
AM6-T7	122.65	29.51	19.95
AM6-BE-T1	126.93	27.05	20.64
AM6-BE-T2	126.25	27.44	20.53
AM6-BE-T3	138.42	20.45	22.51
AM6-D-L1	351.20	3.78	75.53
AM6-D-L2	310.80	14.85	66.84
AM6-D-L3	329.60	9.70	70.89
AM6-D-L4	351.60	3.67	75.62

Table B.7: Weight loss level of stirrups and longitudinal rebars in AM7

Name	residual weight, w_r	Weight loss, ω	Nominal uniform residual cross section, A_r
Unit	g	%	mm ²
AM7-TE-T1	161.48	7.19	26.26
AM7-TE-T2	169.12	2.81	27.51
AM7-TE-T3	165.69	4.77	26.95
AM7-T1	124.01	28.73	20.17
AM7-T2	117.37	32.55	19.09
AM7-T3	114.38	34.26	18.60
AM7-T4	126.47	27.32	20.57
AM7-T5	115.82	33.43	18.84
AM7-T6	122.75	29.45	19.96
AM7-T7	118.29	32.02	19.24
AM7-BE-T1	111.23	36.08	18.09
AM7-BE-T2	122.12	29.82	19.86
AM7-BE-T3	133.82	23.09	21.77
AM7-D-L1	323.40	11.40	69.55
AM7-D-L2	353.10	3.26	75.94
AM7-D-L3	322.25	11.71	69.31
-	-	-	-

APPENDIX C

Pit Depth on Corroded Stirrup

Table C.1: Pit depth on stirrups in Specimen AM1

Name	Maximum pit depth at each region of the stirrup								Overall maximum pit depth	Average Pit depth
	p ₁	p ₂	p ₃	p ₄	p ₅	p ₆	p ₇	p ₈	p _{max}	p _{ave}
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
AM1S3-T1	0.69	0.32	0.15	0.46	0.21	0.23	0.27	0.1	0.69	0.30
AM1S3-T2	0.73	0.06	0.31	0.46	0.98	0.06	0.22	0.03	0.98	0.36
AM1S3-T3	0.89	0.64	0.25	0.85	0.43	0.57	0.1	0.43	0.89	0.52
AM1S3-T4	0.11	0.15	0.43	0.1	0.19	0.17	0.26	0.22	0.43	0.20
AM1S3-T5	0.33	0.44	0.2	1.02	0.22	0.07	0.63	0.16	1.02	0.38
AM1S3-T6	0.2	0.4	0.09	0.45	0.09	1	0.41	0.09	1	0.34
AM1S3-T7	0.14	0.36	0.1	0.38	0.04	0.22	0.25	0.29	0.38	0.22

Note: p₁ denotes the maximum pit depth in region number 1,
p_{max} denotes the maximum pit depth on the each rebar
p' is the nominal uniform pit depth.

Table C.2: Pit depth on stirrups in Specimen AM2

Name	Maximum pit depth at each region of the stirrup								Overall maximum pit depth	Average Pit depth
	p ₁	p ₂	p ₃	p ₄	p ₅	p ₆	p ₇	p ₈	p _{max}	p _{ave}
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
AM2-T1	0.83	1.08	0.41	1.00	0.15	0.20	0.06	1.27	1.27	0.6250
AM2-T2	0.50	1.16	0.78	0.93	0.05	1.13	0.15	0.18	1.16	0.6100
AM2-T3	0.85	1.18	0.12	1.01	0.56	0.49	0.48	0.37	1.18	0.6325
AM2-T4	1.29	0.28	0.44	1.09	0.40	0.20	0.14	0.27	1.29	0.5138
AM2-T5	0.85	0.75	0.90	0.44	0.18	0.29	0.13	1.08	1.08	0.5775
AM2-T6	1.49	0.68	0.45	0.80	0.00	0.29	0.10	1.46	1.49	0.6588
AM2-T7	0.60	1.19	0.25	0.57	0.31	0.29	0.94	0.42	1.19	0.5713

Table C.3: Pit depth on stirrups in Specimen AM3

Name	Maximum pit depth at each region of the stirrup								Overall maximum pit depth	Average Pit depth
	p ₁	p ₂	p ₃	p ₄	p ₅	p ₆	p ₇	p ₈	p _{max}	p _{ave}
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
AM3-T1	0.95	1.46	0.77	0.33	0.29	0.98	0.35	0.85	1.46	0.92
AM3-T2	0.8	0.41	1.44	0.4	0.33	0.96	0.5	2.51	2.51	1.13
AM3-T3	4.24	0.66	1.29	1.64	1.88	0.9	0.51	0.61	4.24	1.07
AM3-T4	0.95	0.8	1.54	1.7	0.45	0.37	1.33	0.36	1.7	0.92
AM3-T5	1.71	1.13	1.09	2.59	0.55	0.58	0.98	0.03	2.59	1.08
AM3-T6	1.56	1.37	1.02	1.58	1.16	0.72	1.5	1.38	1.58	1.11
AM3-T7	0.55	0.2	1.15	1.55	1.05	0.55	1.85	1.25	1.85	1.11

Table C.4: Pit depth on stirrups in Specimen AM4

Name	Maximum pit depth at each region of the stirrup								Overall maximum pit depth	Average Pit depth
	p ₁	p ₂	p ₃	p ₄	p ₅	p ₆	p ₇	p ₈	p _{max}	p _{ave}
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
AM4-T1	1.26	1.00	0.13	0.85	0.81	1.24	0.47	1.68	1.68	0.93
AM4-T2	0.49	2.34	0.78	1.58	1.81	0.75	1.31	2.20	2.34	1.41
AM4-T3	3.38	0.82	0.55	1.13	1.04	1.12	0.30	0.29	3.38	1.08
AM4-T4	2.28	1.10	1.94	0.67	0.86	0.60	0.82	0.58	2.28	1.11
AM4-T5	2.88	1.60	0.28	3.95	1.40	0.96	0.98	0.63	3.95	1.59
AM4-T6	3.58	1.93	1.19	0.89	1.41	1.24	0.41	0.87	3.58	1.44
AM4-T7	1.71	1.70	0.44	1.42	2.10	0.88	0.70	2.08	2.1	1.38

Table C.5: Pit depth on stirrups in Specimen AM5

Name	Maximum pit depth at each region of the stirrup								Overall maximum pit depth	Average Pit depth
	p ₁	p ₂	p ₃	p ₄	p ₅	p ₆	p ₇	p ₈	p _{max}	p _{ave}
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
AM5-T1	1.55	2.13	0.81	2.90	1.30	1.46	0.95	1.97	2.9	1.63
AM5-T2	3.83	1.47	2.31	3.64	0.55	0.97	0.50	*	3.83	1.90
AM5-T3	2.06	*	0.85	2.25	4.55	*	1.79	0.69	4.55	2.03
AM5-T4	1.06	*	1.33	1.75	0.89	*	1.29	2.28	2.28	1.43
AM5-T5	*	3.08	0.43	1.11	2.06	1.54	1.32	2.64	3.08	1.74
AM5-T6	2.68	3.43	4.50	2.63	2.49	1.42	0.84	0.95	4.5	2.37
AM5-T7	1.31	2.19	3.88	*	0.47	2.00	1.19	*	3.88	1.84

Table C.6: Pit depth on stirrups in Specimen AM6

Name	Maximum pit depth at each region of the stirrup								Overall maximum pit depth	Average Pit depth
	p ₁	p ₂	p ₃	p ₄	p ₅	p ₆	p ₇	p ₈	p _{max}	p _{ave}
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
AM6-T1	2.18	2.44	1.88	2.81	2.20	1.49	1.24	2.67	2.81	2.11
AM6-T2	*	*	3.05	*	*	*	2.09	*	3.05	2.57
AM6-T3	2.29	2.23	1.52	1.09	1.18	0.92	0.92	2.34	2.34	1.56
AM6-T4	2.28	3.65	2.49	3.24	1.63	*	2.16	0.37	3.65	2.26
AM6-T5	3.42	2.32	2.62	5.05	1.34	1.54	1.30	1.15	5.05	2.34
AM6-T6	2.82	1.36	1.33	3.97	3.07	1.60	*	3.10	3.97	2.46
AM6-T7	2.01	0.83	2.75	0.76	1.22	2.50	2.52	1.62	2.75	1.78

Table C.7: Pit depth on stirrups in Specimen AM7

Name	Maximum pit depth at each region of the stirrup								Overall maximum pit depth	Average Pit depth
	p ₁	p ₂	p ₃	p ₄	p ₅	p ₆	p ₇	p ₈	p _{max}	p _{ave}
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
AM7-T1	2.56	2.63	2.55	*	0.95	1.19	*	*	2.63	1.98
AM7-T2	2.45	1.26	1.06	2.09	1.43	1.03	1.30	3.08	3.08	1.71
AM7-T3	4.14	3.95	4.00	2.21	2.04	*	0.68	0.33	4.14	2.48
AM7-T4	3.94	3.33	1.75	3.95	3.01	0.86	0.96	0.23	3.95	2.25
AM7-T5	2.78	3.27	3.26	2.29	2.00	1.08	2.70	0.64	3.27	2.25
AM7-T6	2.91	0.92	1.57	0.98	1.87	1.11	1.23	1.55	2.91	1.52
AM7-T7	2.70	2.12	1.59	*	0.95	1.04	*	*	2.7	1.68

Figure C.1: Comparisons of pit depth on Stirrups in Specimen AM1

Figure C.2 : Comparisons of pit depth on Stirrups in Specimen AM2

Figure C.3: Comparisons of pit depth on Stirrups in Specimen AM3

Figure C.4 : Comparisons of pit depth on Stirrups in Specimen AM4

Figure C.5: Comparisons of pit depth on Stirrups in Specimen AM5

Figure C.6: Comparisons of pit depth on Stirrups in Specimen AM6

Figure C.7: Comparisons of pit depth on Stirrups in Specimen AM7

Figure C.8: Overall maximum pit depth of stirrups in middle region of each specimen