ABSTRACT

CALDERON, VICTOR ALEJANDRO. Time Dependent Performance Based Design. (Under the direction of Dr. Mervyn Kowalsky.)

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Time Dependent Performance Based Design

by
Victor Alejandro Calderon

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APPROVED BY:

Dr. James Nau	Dr. Mohammad Pour-Ghaz
Dr. Rudolf Seracino	Dr. Thomas Birkland

Dr. Mervyn Kowalsky
Chair of Advisory Committee

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Chapter 1

INTRODUCTION

Bridges are designed based on discrete events with minimal consideration of interactions between hazards/loading, material aging (or more accurately condition) and bridge performance. The purpose of the research described is to study Time Dependent Performance Based Design that considers the effects of cumulative damage on the properties of the materials both as a function of time and current condition. Specific items of interest include corrosion, strain aging, low cycle fatigue and strength aging. In addition, since there is a high likelihood for a structure in a high seismic region to be subjected to more than one main shock throughout its life, it is deemed important to consider the effects of multiple earthquakes. As a consequence, the effects of repairs on the structural response are also of great importance. An analytical procedure is implemented such that it considers the effect of aging on structures, more specifically this study starts by evaluating an RC bridge Column. A series of condition dependent nonlinear time history analysis are performed assuming that a series of earthquakes occurs throughout the lifetime of the structure while at the same time changing the properties of the structure as time progresses. To achieve this a library of time dependent materials are developed. At the end of each series the main variables of study are the the limit state that was reached, the controlling mode of response (flexural or shear controlled), Equivalent Viscous Damping and the accumulated deformations. The series of earthquake proposed consists of (1) equally spaced main shocks only, (2) main shock-aftershocks series and (3) main shock-aftershock-repair series. At the end of the presentation recommendations on design of new structures and assessment of existing structures will be provided.

1.1 Motivation

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1.2 Scope and layout

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Chapter 2

LITERATURE REVIEW

Bridges are designed based on discrete events with minimal consideration of interactions between hazards/loading, material aging (or more accurately condition) and bridge performance. The purpose of the research described is to study Time Dependent Performance Based Design that considers the effects of cumulative damage on the properties of the materials.

In this chapter the available knowledge on the different topics that are available in the literature are synthesized. First a review on the different definitions of commutative damage is presented then the main idea for this research are established and the required components, then the different elements that form part of this study are presented and a general concept is established and presented in Chapter 3.

2.1 Cumulative Damage

There have been attempts by many researchers to stablish the best way to account for the accumulation of damage.

2.1.1 Damage Index

The effect of commulative damage in structures was first studied by by Park and Ang (1985) [28] in their study the authors proposed the Damage Index as shown in 2.1. The damage index was used as a measure to quantify damage in terms of the maximum experienced earthquake and the absorbed hysteretic energy.

$$D = \frac{\Delta_m}{\Delta_u} - \beta \frac{E_h}{F_y \Delta u} \tag{2.1}$$

$Delta_m$: Maximum deformation under earthquake

$Delta_u$: Ultimate deformation under monotonic loading

F_y: Calculated yield strength

E_h: Total hysteretic energy

β: Dimensionless constant

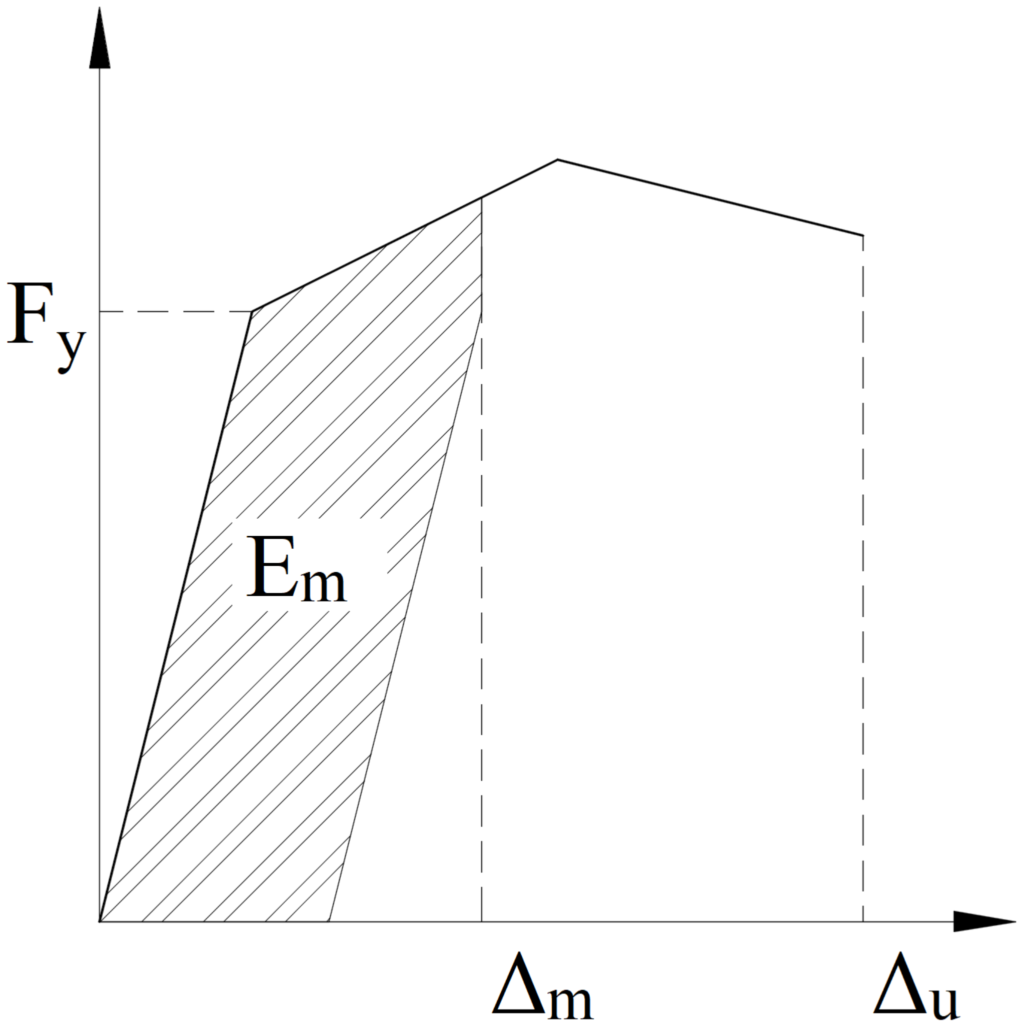

Figure 2.1 Parkn and Ang conceptual scheme

This equation was derived for concrete elements. The first term here is a simple, pseudostatic displacement measure. It takes no account of cumulative damage, which is accounted for solely by the energy term. A figure on the concept is shown in Fig. 2.1. The advantages of this model are its simplicity, and the flexibility on adpating the model to correlate with experimental data.

In its current form, this model has several limitations. Firstly the calibration of the β coefficient with observed damage, has shown to be very low ($\beta = 0.05 - 0.15$) rendering the second term relatively inconsequential compared to the contribution of the first term. In addition, the model was derived for reinforced concrete with poor shear detailing. The correlations observed in this model also showed the data to be sparse.

Depite its limitation, several studies have used or modified this model to study the effects of cumulative damage for different structures, of relevant importance are those performed by [13], who used a modified Park and Ang model, to model damagae at the local level for elements in a structural analysis program IDARC 3.0, in this software for the case of multiple degrees of freedom buildings they also added parameters to consider the damage at the inter-story level and the global model. Ghosh et al [8] developed a damage accumulation framework to develop probabilistic estimates of exceeding a damage index for multiple ground motions. Other regressions have been proposed by [11], Fajfar1992, Roufaiel but show no improvement in assessing the damage state of a structure. While these studies provide insight into some of the characteristics of damage accumulation they rely on the Park and Ang model and therefore carry the limitations of the model.

Krawinkler (1987) [12] proposed a method that would consider damage as a function of low cycle fatigue parameters, the form of this damage index for Steel Component, Weldements and local buckling have a general shape of the Miner model. This model relies on the accumulation of plastic deformations. While this model has proven to work well for the evaluation of individual elements it does not provide a way to generalize damage for other types of structures.

2.1.2 Probabilistic Approach

Increasing interest on the effect of cumulative damage has been experienced over recent years. These studies have focused on determining what is the increase in the risk of a structure due to the accumulation of factors due to many reasons such as multiple earthquakes, corrosion and life span of the structure. Two main approaches have been observed:

- Probabilistic Framework
- Fragility Curves

Proababilistic network

One of the most widely used probabilistic network is the Pacific Earthquake Engineering Research Center (PEER) Performance Based Design. PEER PBD can be expressed by the following equation:

$$\nu_{DM}(dm^{LS}) = \iint D_{DM|EDP}(dm|edp)|, dG_{EDP|IM}(edp|im)||\, d\nu_{IM}(im)| \tag{2.2}$$

Mackie et al [15] on the basis of the PEER PBD developed the Performance Based Damage Design (PBDD) and Perfomance Based Loss Design (PBLD) by defining the probabilistic demand, damage, and loss model parameters in terms of reinforced concrete column damage. The RC column damage was defined on terms of drift ratios defined for the limit states of concrete spalling, bar buckling and failure.

The authors show that for a given intensity measure (IM) and a confidence level of achieving a limit state, its is possible then to define the probability of exceeding that limit state.

While this methodology was able to define damage and incorporate it into the PEER PBD framework, the authors did not consider a better way to establish a limit state such as strains. Also recent research has shown that other intensity measures such as spectral displacement at effective fir mode period provide a better intensity measure.

Fragility Curves

Another common trend in this subject is the use of fragility curves to estimate the effect of damage in structures. Two main approaches were found one of them relied on the Park and Ang Model Damage Index to define damage. The second approach relates damage to drift.

Ghosh et al [8] formulated a damage acumulation framework. This study relied on the Park and Ang Damage index explained in the previous section. The study performed a series of nonlinear time history analyses for two cases:

- using a constant main shock hazard occurrence rate (3 main shocks in a 50 year period)
- Mainshock - Aftershock series using time-dependent aftershock hazard occurrence rate

Evaluation of the damage index exceedance probability in two cases was performed. The results from this study show regression equations that statistically predict the damage index as a function of earthquake intensity and damage history. This study revealed that for both main shock and aftershock scenarios there was a significant increase in the probability of damage index exceedance under repeated shock scenarios. It is important to notice that this study carries out the same disadvantages of the Park and Ang damage index.

Ghosh et al [7] also studied the effects of corrosion in time dependent seismic fragility curves. This study characterizes corrosion in concrete columns as a continuous phenomena that occurs as a function of time once the time to corrosion has passed. Additionally the authors considered the effects of corrosion in Steel Bridge Bearings. The authors the ran a series of NLTHA analysis for different aging times of the structures. Based on those analysis time dependent fragility curves were presented. The results showed that as time increases, and as a consequence corrosion increases, the probability of exceeding a limit state increases. In this study limit states where defined on the basis of inter-story drifts which were obtained from experimental results and field observations [21]. It is important to mention that limit states were not defined on the basis of strains or other structural property rather from a survey performed in central southeastern United States departments of transportation on the premise of a range of experienced inter-story drifts and the time to repair them. Additionally assuming that corrosion is a continuous process has to be cautiously taken as valid since this is seldom the case in real structures.

While these studies provide a general view on how damage increases the likelihood of observing damage the methods used to arrive to those conclusions can be misleading since the definition of damage as either a Damage Index or Drift does not necessarily represent a quantifiable measure of damage it is our belief that strain based limit states will provide a better understanding and implications on the damage accumulation.

Chapter 3

Study Gap

3.1 Research Gap

As was stated previously, there is considerable discussion in the literature over the choice of viscous damping models for NLTHA of structures. The very fact that such a discussion is taking place, and gaining momentum, illustrates its broad impacts for performance-based design. As the engineering community continues to advance Performance Based Design, accurate estimates of deformation capacities and demands become essential. Such calculations were irrelevant in the force-based era, but are now vital.

Some researchers have agreed on the need to conduct a deep parametric study regarding the impact of various damping models on the nonlinear response of structures. Furthermore, the research that has been done mainly involves the consequences of the selection of damping models for the inelastic response of buildings where non-structural components contribute to damping. In the cases of bridges, where the systems approach are based on a bare-frame, limited studies have been conducted.

This research will develop a sensitivity analysis based upon NLTHA that considers numerous bridge typologies and damping models. It is expected that this research will help guide engineers on the consequences of the use/misuse of different damping models in non-linear analysis. In addition, it will provide a framework where practitioners might consider using Direct Displacement-Based Design (DDBD) as an alternative tool not only to design common highway bridges but also to verify procedures that are more complex.

3.2 General Objectives

Guide engineers on the consequences of the use/misuse of different damping models in the non-linear analysis of bridges.

3.3 Specific Objectives

Chapter 4

Methodology

A methodology that incorporates the different sources of cumulative damage in RC structures is proposed, the main elements that might induce damage in a structure are:

- Corrosion
- Strain aging
- Low-cycle fatigue
- Concrete Strength Aging

These effects generally affect the mechanical behavior of the materials, which are not considered when designing a structure. In the following paragraphs the different models available are studied and later incorporated into the analysis methodology.

4.1 Corrosion

One of the main phenomenon that affect the long term behavior of RC structures is corrosion of the reinforcing steel. Two types of corrosion are possible:

- Carbonation,
- Chloride attack

The main source of corrosion in most RC structures is Chloride Attack and it is the one that is assumed in the present study.

Corrosion of steel in concrete is an electrochemical process [19] this corrosion may be generated in two ways:

- Composition cells may be formed when to dissimilar metals are embedded in concrete or when significant variations exist in the surface characteristics of steel
- In the vicinity of steel concentration cells may be formed due to differences in the concentration of dissolved ions, such as alkalies and **chlorides**.

The corrosion process under chloride attack type of corrosion consists in first the protective film on the reinforcing steel surface is destroyed, a process known as **depassivation**, then initiation of corrosion happens, the electrical resistivity and the oxygen content control corrosion. Figure 4.1 schematically show this process.

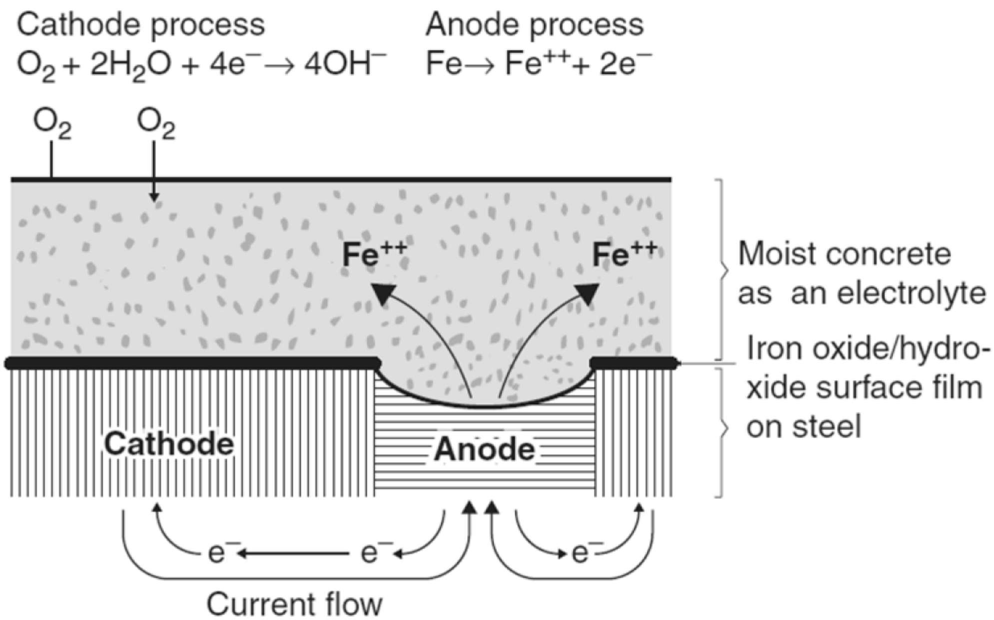

Figure 4.1 Corrosion Process in Reinforcing Steel Bar [19]

A literature review to characterize corrosion in reinforcing steel is presented to model corrosion can be modeled, the corrosion process is an extensive field of research and to accurately incorporate it into the analysis it is necessary to incorporate the following components:

1. Time to Initiation of Corrosion (Tcorr)
2. Corrosion growth in reinforcing steel

3. Mechanical Properties of Corroded Reinforcing Steel (fycorr, fucorr)

4. Cyclic Test on Corroded RC Columns

5. Flowchart of Corrosion Model Implemented

4.1.1 Time to Corrosion

Time to corrosion refers to the corrosion initiation at which the passivation of steel is destroyed and reinforcement starts corroding actively.

Christensen Model

Christensen [25] main goal was to generate a corrosion model that was general for all concrete elements, additionally the authors tried to generate a model that also included the appearance of cracks due to corrosion that would evetually grow and the spall the concrete. More specifically related to reinforcing steel corrosion they developed a model based on Fick's law of diffusion to model the rate of chloride penetration into concrete as a function of concrete cover and time.

$$\frac{\partial C(x,t)}{\partial t} = D_c \frac{\partial C(x,t)}{\partial x^2} \tag{4.1}$$

After solving equation 4.1 the following expression results:

$$T_{corr} = \frac{d^2}{4D_c} \left[erf^{-1} \left(\frac{C_{cr} - C_0}{C_1 - C_0} \right) \right] \tag{4.2}$$

d: Concrete cover
D_0: Diffusion coefficient
C_0: Equilibrium Chloride Concentration
C_{cr}: Critical chloride corrosion concentration

This model provides a means to calculate the Time for initiation of corrosion as a function of Concrete Cover and Diffusion concentration, the estimation of the Diffusion concentration depends on several factors such as environment, curing and water to cement ratio, however, it is not a reliable method to estimate the Time to Corrosion.

Gosh & Padgett Model

Gosh et al calculate time to corrosion based on Thoft-Christensen model, considering in-field corrosion related studies of existing bridge components in the United States exposed to deicing salts to obtain mean values of chlorides concentration and put them in a modified version of the Thoft-Christensen Model.

$$T_{corr} = \frac{x^2}{4D_c}\left[erf^{-1}\left(\frac{C_0 - C_cr}{C_0}\right)\right]^{-2} \tag{4.3}$$

D_c : Diffusion Coefficient a recommended value of $1.29\frac{cm^2}{year}$ is given in their study [7]
C_0 : Surface Chloride Concentration, recommended (0.10)
C_r : Critical Chloride Concentration, recommended (0.04)

While this model provides mean values for the time of initiation of corrosion, it is limited to environments that are controlled by **dicing salts only**.

Life 365

Is a software developed by a consortium of companies of the cementitious materials industries and academic institutions. This software relies on the studies summarized above, mainly using the Thoft-Christensen model, but as opposed to assuming dicing environments only, this software uses a database of chlorides concentration for different location in the USA and Canada, which gives more accurate results depending on the location and environment in which the structure is located..

While this is a more robust model to obtain the initiation of corrosion since it considers the location and environment of the structure and it also has the ability to include other durability issues, it is difficult to implement in a batch run mode since the program is in a closed format.

Liu & Weyers Model

This model tries to calculate the time to initiation of corrosion by calculating the amount of corroding products that are needed to fill the voids in the concrete cover that will eventually generate cracking in this area and therefore initiate the accelerated corrosion of the reinforcing steel this is characterized through the following set of equations:

$$W_{crit} = \rho_{rust}\left[\pi\left[\frac{Cf_t'}{E_{ef}}\left(\frac{a^2+b^2}{a^2-b^2}+\nu_c\right)+d_o\right]D+\frac{W_{st}}{\rho_{st}}\right] \tag{4.4}$$

$$T_{cr} = \frac{W_{crit}^2}{2k_p} \tag{4.5}$$

$$k_p = 0.098(\frac{1}{\alpha})\pi D i_{corr} \tag{4.6}$$

W_{crit}: Critical amount of corrosion needed to induce cracking.
W_{st}: Mass of corroded steel.
ρ_{rust}: Density of rust material.
ρ_{st}: Density of steel.
f_t': Tensile strength of the concrete.
E_{ef}: Effective elastic modulus of concrete $E_{ef} = \frac{E_c}{1+\phi_{crit}}$
ϕ_{crit} Creep coefficient of the concrete.
D: Diameter of bar.
d_o: Thickness of pore band around the steel/concrete interface.
ν_c: Poisson's ratio of concrete.
C: Cover depth
$a = \frac{D+2d_o}{2}$
$b = C + \frac{D+2d_o}{2}$

This model however is limited to corrosion on concrete slabs, since a series of experiments were developed to generate these equations, however, it is able predict the time to corrosion with great accuracy.

4.1.2 Rate of corrosion

Vu et al. Model

To estimate the loss of steel cross section due to corrosion a time dependent corrosion rate model was developed by [26], this model implies that corrosion diminishes with time. As corrosion accumulates with time around the steel, it precludes the uncorroded steel to react with the environment. The model is shown in Eq. 4.7.

$$i_{corr} = \frac{37.5(1 - w/c)}{d_c} \tag{4.7}$$

w/c: Water Cement ratio d_c: Cover depth

In Fig. 4.2 the behavior of this model for different values of w/c ratios is shown. It can be seen that at larger values of cover depth the rate of corrosion decreases rapidly and as the water cement ratio increases the rate of corrosion decreases.

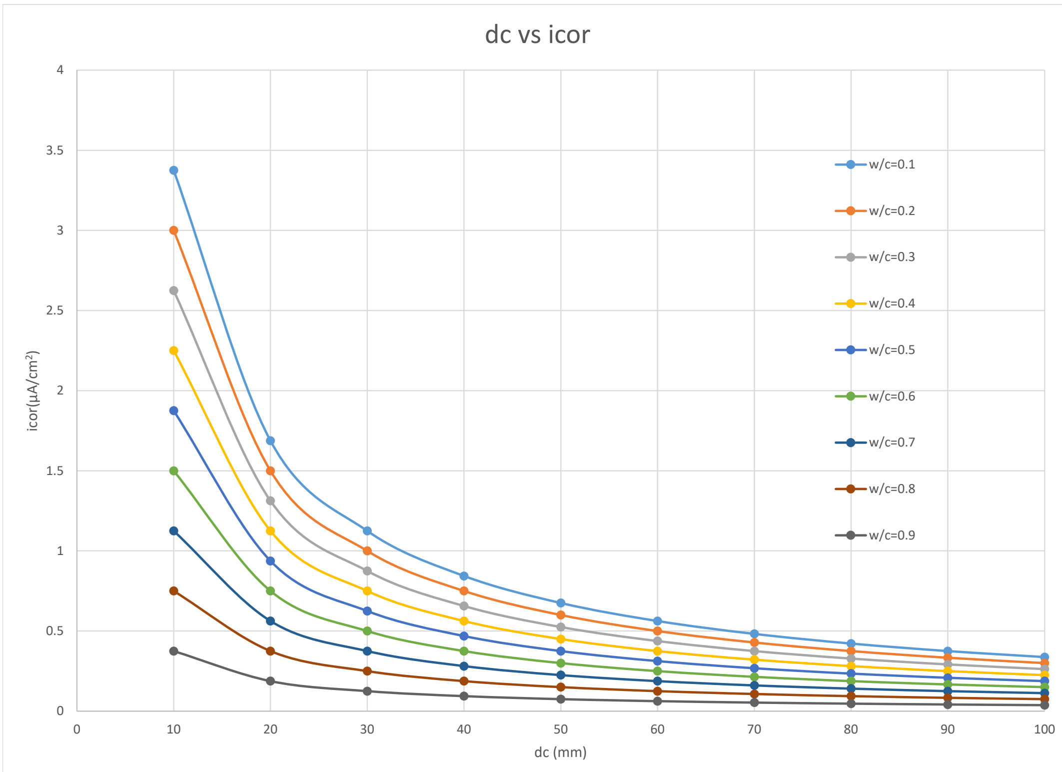

Figure 4.2 Concrete cover depth vs rate of corrosion

From the Vu et al model the diameter degradation is calculated according to Choe et al as:

$$d_{corr} = d_{bi} - \frac{1.0508(1-w/c)}{d_c}(t-t_{corr})^{0.71} \tag{4.8}$$

d_{bi}: Is the initial diameter of the bar

The diameter is plotted in Fig. 4.3.

These values would correspond to a level of corrosion that varies from 7% corrosion to 21% of corrosion for w/c ratios that ranges from 0.4 to 0.6 The level of corrosion is calculated as:

$$C = \frac{G_o - G}{g_o l_o} * 100 \tag{4.9}$$

Then the Corrosion level is plotted as a function of time in Fig. 4.4

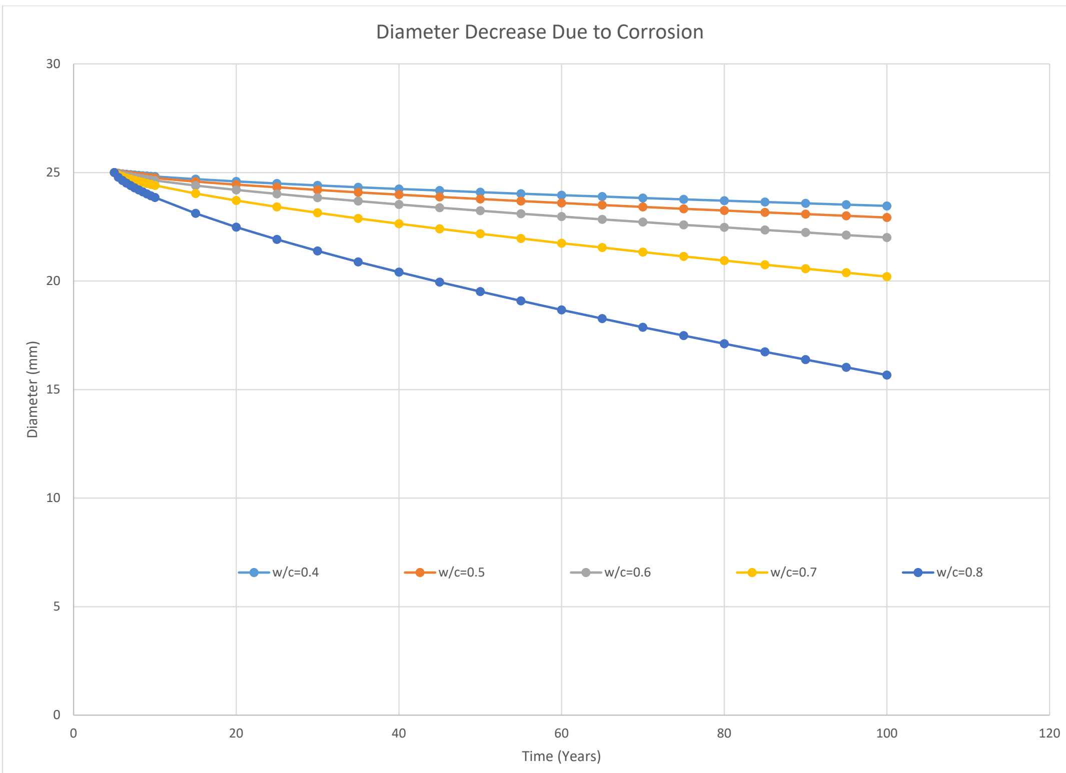

Figure 4.3 Diameter decrease due to corrosion

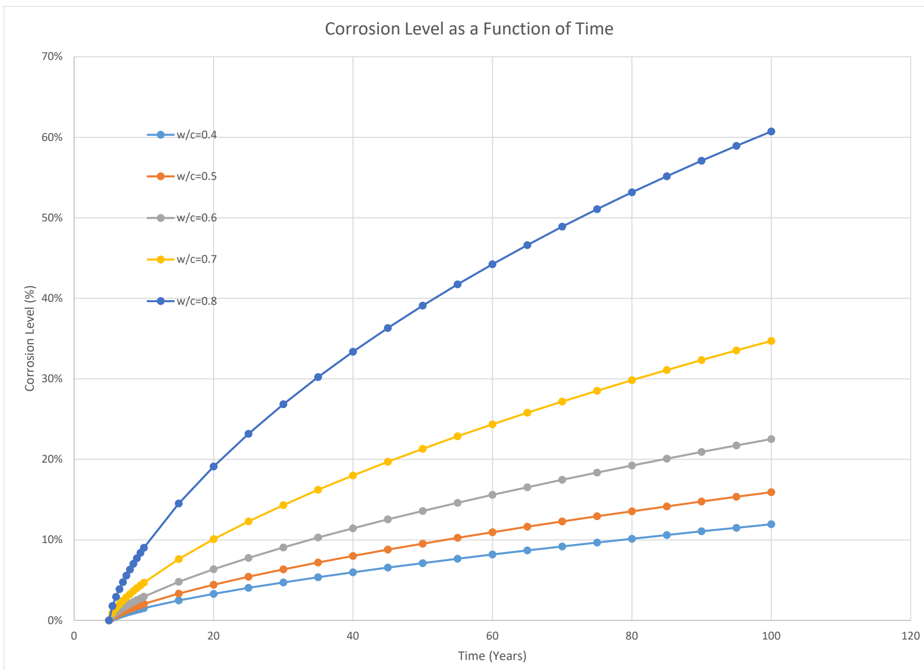

Figure 4.4 Corrosion Level vs Time (years)

4.1.3 Corrosion modified properties of reinforcing steel bars

In a study presented by Yuan et al [29] it was shown from experimental results that the mechanical properties of steel for different levels of corrosion could be modified for analysis as follows:

$$f_{y,C} = f_{yo}(1 - 0.021C) \tag{4.10}$$

$$f_{u,C} = f_{yo}(1.018 - 0.019C)$$

$$\delta_{s,C} = \delta_{so}(1 - 0.021C)$$

$$\varepsilon_{y,C} = \varepsilon_{yo}(1 - 0.021C)$$

Choe et al. Model Choe et al research is a seismic fragility estimates for RC columns subjected to corrosion, while the study is probabilistic in nature it defines the reduction in rebar cross section as:

$$d_b(t) = d_{bi} - 2\int_{T_{corr}}^{t} \lambda(t)dt \tag{4.11}$$

Considering the model proposed by Vu et al the bar diameter degradation can be expressed as:

$$d_b(t) = d_{bi} - \frac{1.508(1-\frac{w}{c})^{-}1.64}{d}(t - T_{corr})^{0}.71 \tag{4.12}$$

Where the diameter of the bar and the cover is in (mm). Pros: Easy way to calculate the reduction of bar diameter. Cons: The model carries out the assumptions made by Vu et al. concerning concentration of chlorides assumed and the diffusion assumed.

With this information, the corrosion level is calculated as:

$$CL = \frac{d_i - d(t)}{d_i} \tag{4.13}$$

4.1.4 Physical test on corroded RC Structures

Recent studies [14], [18] and [27] have been developed to assess the force-displacement relationships in cantilever RC Columns. These columns were subjected to Quasi-Static Loading Protocol, the concrete columns were subjected to accelerated corrosion to obtain different Corrosion Levels (CL), the range of CL for these studies correspond to $CL = 0\%20\%$. In these studies the accelerated corrosion was performed via an electrochemical process directly applied to the reinforcing steel as shown in Fig. 4.5.

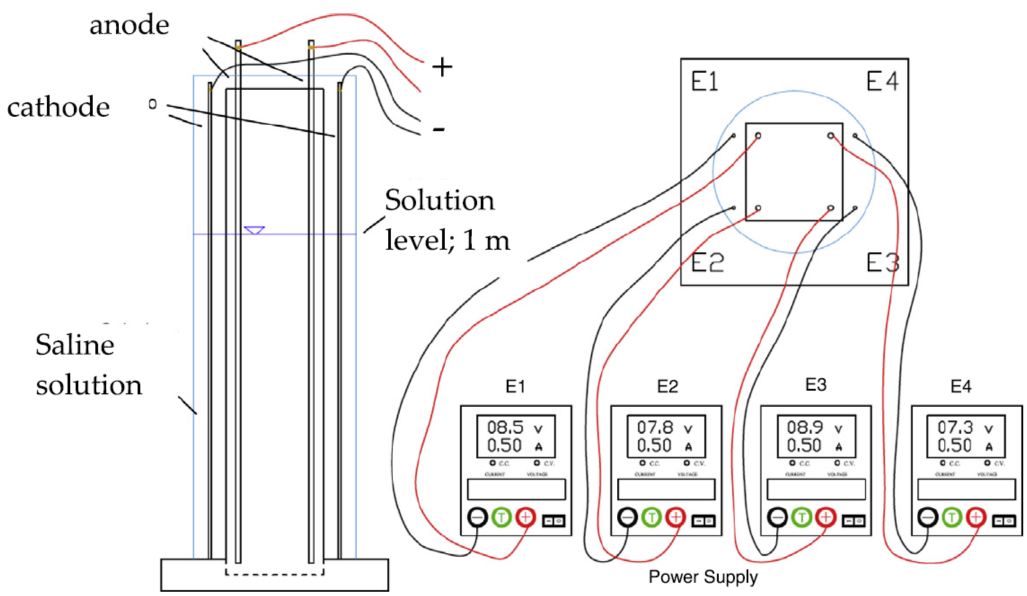

Figure 4.5 Corrosion Process for RC Column [18]

The resulting force displacement of these experiments is shown in Fig. 4.6 it can be seen that there is a reduction not only on the strength of the system but also on the displacement capacity.

As stated in the previous section the mechanical properties of steel are affected by corrosion, in the previous studies [18] the authors performed tension tests on corroded reinforcing steel. In these tests a reduction in the mechanical properties of steel was observed as well as a reduction in the rupture strain ε_{srup}, see Fig. 4.7.

While these studies provided an insight into how corroded RC Columns behave under cyclic loading, they did not considered the generation of the protective film due to the alkaline environment of the concrete, this film can modify mechanical properties of corroded steel. Additionally the accelerated corrosion process used a 3% $NaCl$ concentration solution while the chloride attack in concrete usually has a 1.0% - 1.5% concentration of the same Chloride. Therefore the results obtained from these studies might not accurately represent the actual conditions

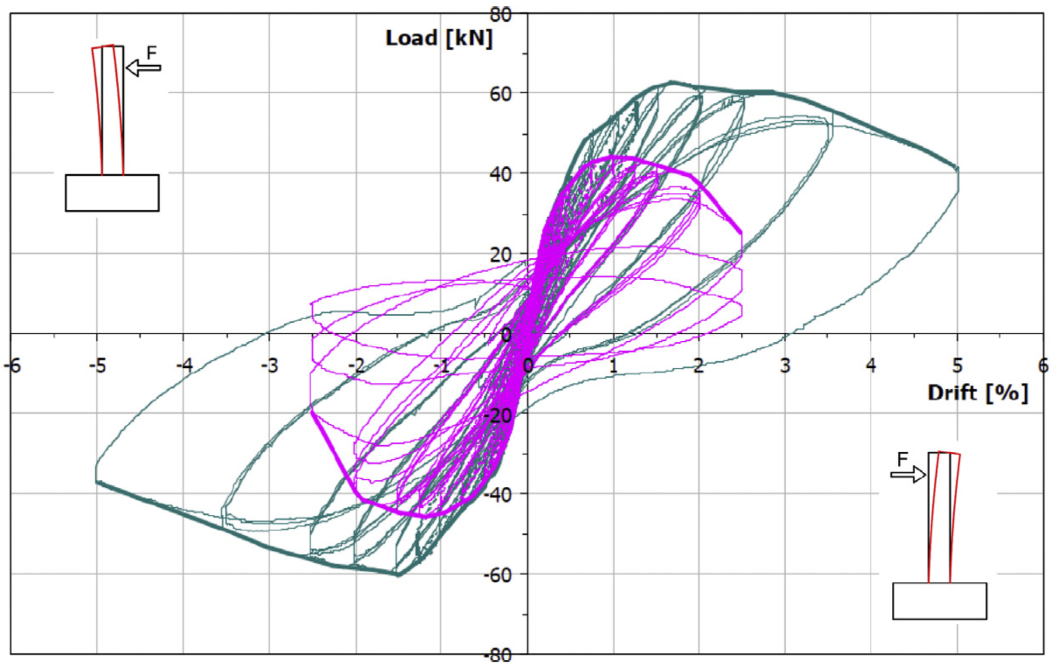

Figure 4.6 Corrosion Process for RC Column [18]

of corroded RC columns, thus an experimental campaign is proposed that would shed a light into the properties of corroded reinforcing steel inside concrete and is discussed in the following subsection.

4.1.5 Proposed Experimental campaign

As explained in the previous section the steel inside concrete generates a protective film and after chloride attack reaches the surface of the steel, this protective film starts to be eliminated. This same process will be simulated through the following steps.

1. Passivation of reinforcing steel
2. Accelerated corrosion of Reinforcing Steel
3. Tension Tests
4. Buckled Bar Tension (BBT) Test

Passivation of reinforcing steel

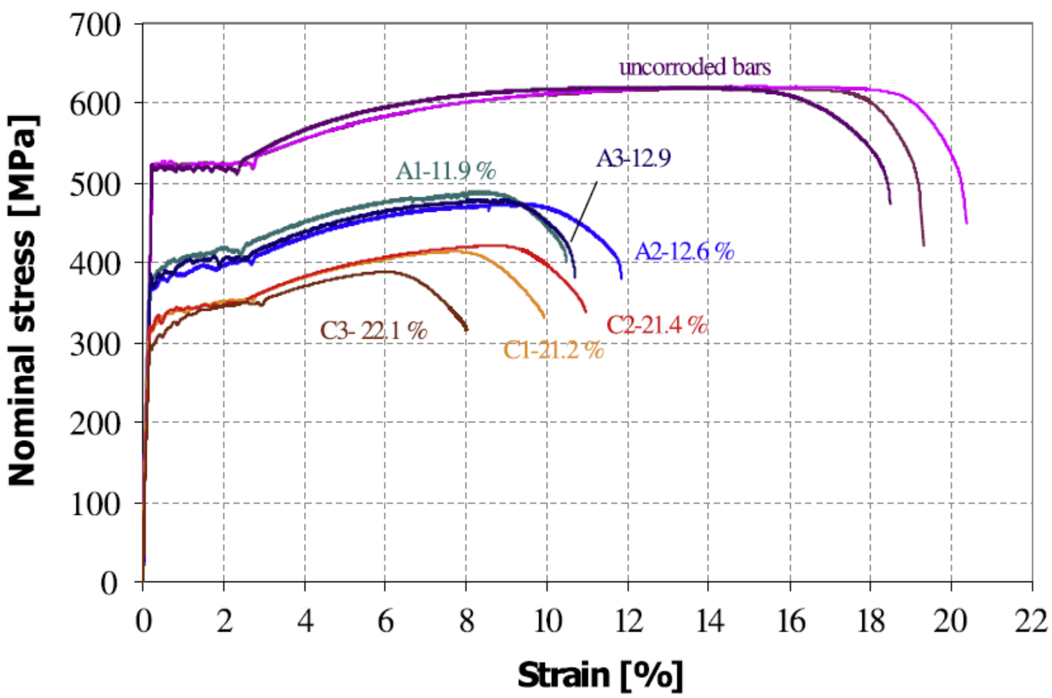

Figure 4.7 Corroded Rebars Stress-Strain Curves [18]

Methods to generate the passive film on reinforcing steel are available in the literature [6]. According to this study it is possible to generate the passivation process in the same way as it occurs to reinforcing steel inside the concrete. A porous solution will be generated with the following concentrations:

- Saturated Calcium Hydroxide $Ca(OH)_2$
- Sodium Hydroxide $Na(OH)$ 4.00 g/l
- Potassium Hydroxide (OH) 11.22 g/l
- Calcium Sulfate Dihydrate $Ca(SO)_4 + 2H_2O$ 13.77 g/l

The rebars will be placed in a container with the pore solution for a minimum of 8 days. Annodic Polarization Tests will be measured on the rebars to determine the passive current density. A figure of this process is shown in Fig. 4.8. Additionally The ends of the rebars will be protected to prevent corrosion in these zones of the specimens, the protection at the ends is based from the standard ASTM G109-07 with some alterations. Figures Fig. 4.9 and Fig. 4.10 show the specimen geometry and the preparation of the ends of the rebars.

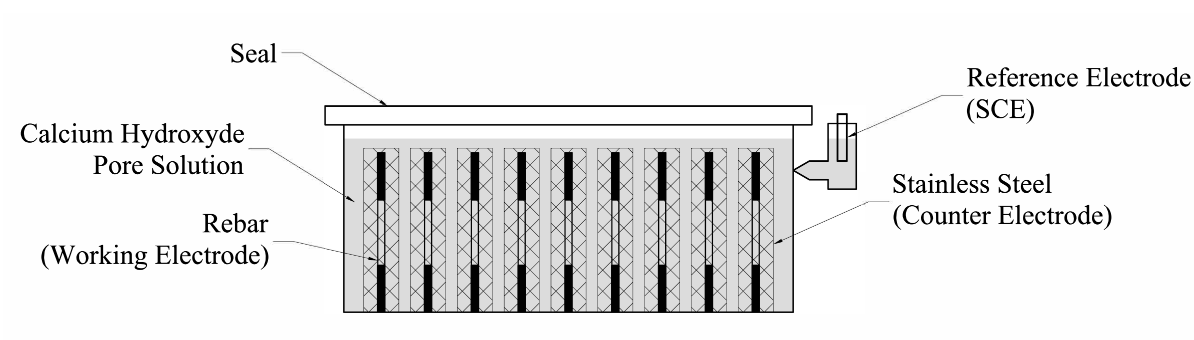

Figure 4.8 Rebars Passivation Process in Calcium Hydroxyde Pore Solution

Accelerated corrosion of Reinforcing Steel

The accelerated corrosion will be done by using a galvanic cell. Different studies [6] has shown that for rebars with pasive films a concentration of 0.3 Moles of sodium chloride ($NaCl$) will start the depassivation process on the rebars. The rebars will be subjected to a current of . This current is sustained for a period of time according to Faraday's Law until the desired level of corrosion is reached:

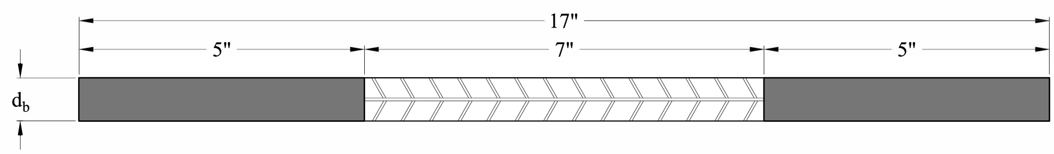

Figure 4.9 Rebar Specimen Geometry

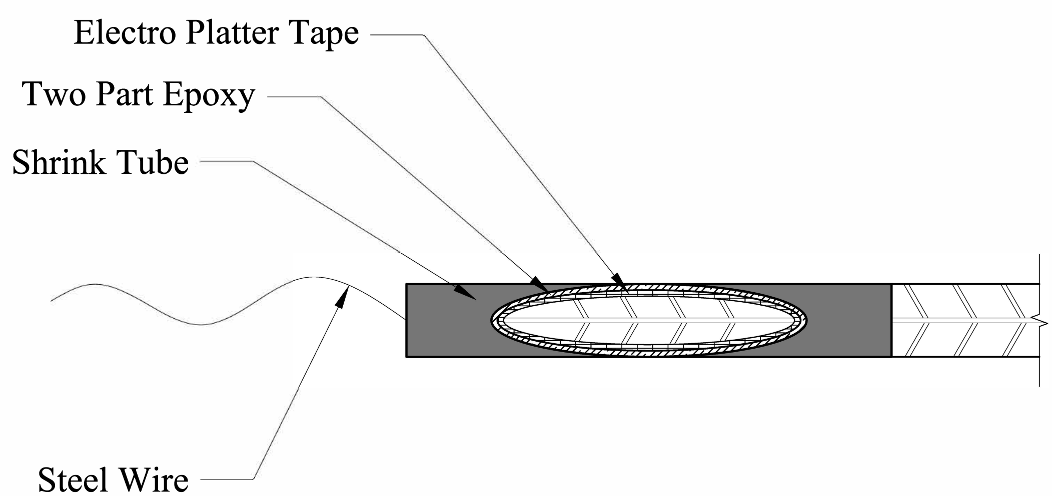

Figure 4.10 Rebars Ends Protection

$$t = \frac{\lambda m_{loss} \eta_{specimen} C_{faraday}}{i M_{specimen}} \tag{4.14}$$

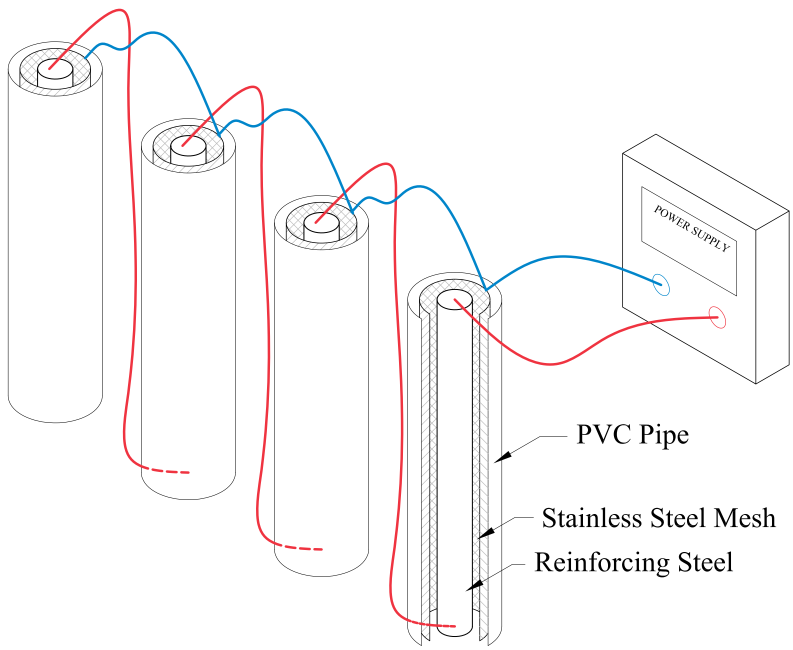

Figure 4.11 Accelerated Corrosion Process

For the different rebar sizes and Corrosion levels the current and the Time of Application is shown in Table 4.1. A current of $5mA$ is applied to the specimen obtaining the time this current needs to be applied for.

Tension Tests

Tension tests will be performed according to ASTM A706. The main objective of this tests is to evaluate differences in the Stress-Strain behavior of corroded Reinforcing Steel. This will help in determining any reduction in the ductility of steel for this condition.

Buckled Bar Tension (BBT) Test

One of the limit states that control Performance Based Design is Buckling of Reinforcing steel, recent tests have been developed to determine the critical bending strain of buckling of reinforced steel [2]. The premise of the BBT Test is a material test to simulate bending and

Table 4.1 Accelerated Corrosion to achieve Corrosion Levels.

Corrosion Level (CL)	Mass loss (g)	time(days)
5%	1.12	9
10%	2.24	18
15%	3.36	27
20%	4.47	36
25%	5.59	45

tension strain demands on a buckled bar.However those results have been developed for rebars in pristine condition, it is therefore necessary to check if available expressions to determine this limit state hold for corroded steel.

The Buckled Bar Tension Test consists in:

1. Compress a rebar specimen up to a certain level of compression strain such that the rebar will show buckling

2. The rebar is then pulled untill rupture

3. process is repeated for different levels of compression strains

This test is proposed for different levels of corrosion such that any changes on the behavior are studied and incorporated into the analysis a sequence of the test procedure is shown in Fig. 4.12. A proposed test matrix is shown in Table 4.2.

4.1.6 Modeling of corrosion for Structural Analysis

The previous elements of corrosion explained in the previous sections are incorporated into the structural analysis mainly at the material level. The application can be outlined as follows:

1. First the time for initiation of corrosion is calculated according to the Gosh and Padgett Model [7]

2. Then the rate of corrosion is calculated according to the Vu et al model [26]

3. Following this the size of the rebar is reduced and the corrosion level is calculated

4. Finally the mechanical properties of the reinforcing steel are modified with the corresponding corrosion level.

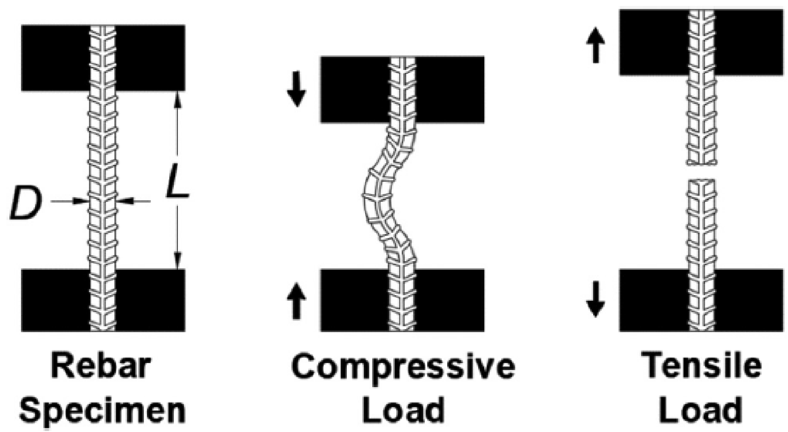

Figure 4.12 BBT Test sequence

Table 4.2 Corroded Rebar Test Matrix

Corroded BBT Test Matrix			
Test	Diameter of Bar	CL (%)	Number of Tests
Tension Test	#6	0	3
		5	3
		10	3
		15	3
		20	3
		25	3
BBT Test	#6	0	6
		5	6
		10	6
		15	6
		20	6
		25	6

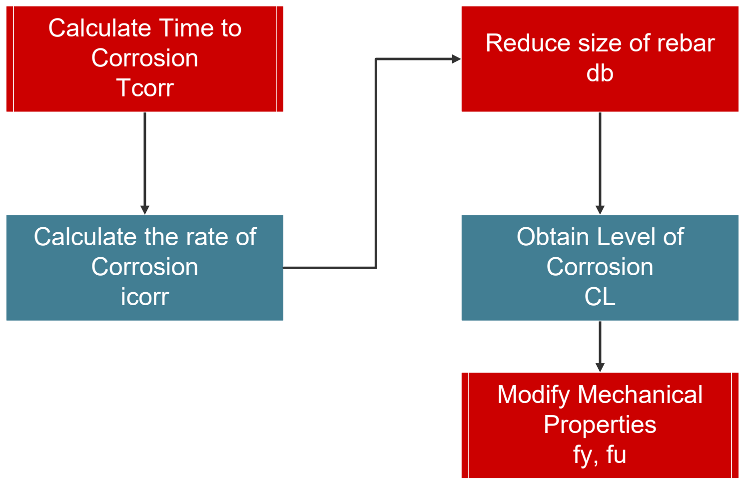

Figure 4.13 Corrosion Modeling for Structural Analysis

This modeling can be better seen in Fig. 4.13

This is later incorporated into the Nonlinear Structural Analysis Framework using the package OpenSees [17], the framework of this analysis is explained in [**Chapter-5**].

4.2 Steel Strain Aging

4.2.1 Metallurgical Process

It is generally accepted that strain aging is due to the diffusion of carbon and/or nitrogen atoms in solution to dislocations that have been generated by plastic deformation. Initially, an atmosphere of carbon and nitrogen atoms is formed along the length of a dislocation, immobilizing it. Extended aging, however, results in sufficient carbon and nitrogen atoms for precipitates to form along the length of the dislocation.

These precipitates impede the motion of subsequent dislocations, and result in some hardening and loss in ductility. The extent of strain aging, which is a thermally activated process, depends primarily on aging time and temperature. In general, extended aging results in a saturation value above which further aging has no effect.

A second strengthening mechanism occurs when cold deformation (alone) is applied to steels. When dislocations break away for their pinning interstitial atoms and begin the movement causing slip they begin to intersect with each other. A complex series of interactions between the dislocations occurs, causing them to pin each other, decreasing their mobility. The decreased mobility also results in higher strength, lower ductility and lower toughness. As a result, cold deformed steels already have lowered ductility and toughness before any strain aging occurs and when heating follows cold deformation, the loss in ductility and toughness is greater. It is this combination of events that is the most damaging to the toughness of structural steels.

4.2.2 Strain aging effects in structures

Since it has already been established that strain aging is the process in which steel after being subjected to large strains develops an increased strength and reduced ductility with time and therefore important to include it in a time dependent analysis, considering the fact that plastic hinges will form in a ductile structure and the steel could reach high strains in this regions of the structure. Furthermore strain aging will cause an increased in the strength of the plastic hinge and as a consequence plastic hinges might be formed in regions of the structures that have not been designed for such demands. The effects of strain aging may also alter the transverse reinforcement due to both cold bending, making them susceptible to brittle failure.

According to [23] most strain aging occurs in the first 37 days. Also [20] studied strain aging effects with respect to time for different levels of pre-strains that ranged from $2\varepsilon_y - 10\varepsilon_y$ and for a time frame of 3 days to 50 days, from this study it was determined that a significant effect of strain aging took place from pre-strains $5\varepsilon_y$ and on. Strains higher than $15\varepsilon_y$ indicate a performance level in which substantial damage has been induced in the structure such that it is deemed unrepairable and therefore pre-strains higher that $15\varepsilon_y$ are unpractical and not studied by Montahan et al[20].

Momtahan et al Strain Aging Effects in Yield Strength of Steel

Momtahan et al was able to correlate the increase in yield strength as a function of time and the pre-strain in reinforcing steel bars. The proposed equations are shown below:

For $10\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0026t + 0.9838 \tag{4.15}$$

For $5\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0008t + 0.996 \tag{4.16}$$

For $2\varepsilon_y$

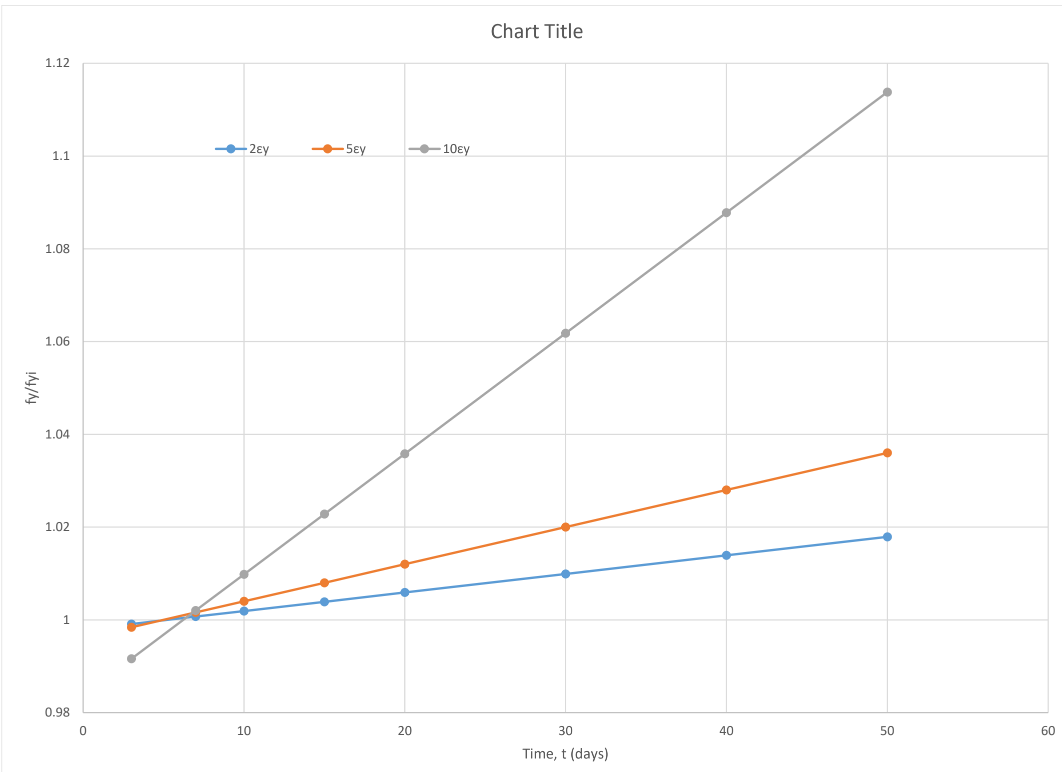

Figure 4.14 Strain Aging effect on Yield Strength vs Time (days)

$$\frac{f_y}{f_{yi}} = 0.0004t + 0.9979 \tag{4.17}$$

It is proposed to limit the increase in yield strength to the one obtained at 50 days. These equations are plotted in Fig. 4.14

4.3 Multiple Seismic Events

The evaluation of multiple seismic events is a topic that has been scarcely studied, however their effects have been felt in numerous earthquake sequences such as El Salvador, North Ridge, Chi-Chi among others. The main thought is that after a series of earthquake the structures accumulate damage and would eventually fail, this has been attempted as it was shown in [**Chapter-1**].

For this study it has been determined that not all damage in structures are dependent on multiple events but rather their condition when an event occours as is the case for corrosion. Other damage related phenomenons such as Strain Aging depend on the loading history and are therefore dependent on the history of extreme loading events. It is therefore proposed to study corrosion on a discrete modeling of Main Shocks each independent of the other and to study the effect on Strain Aging by using a sequence of Main Shocks.

4.3.1 Earthquake Selection

For this study the NGA2 West Database of earthquake records provided by the Pacific Earthquake and Engineering Research Insitute (PEER) [1] is used. This database consists of 599 different Earthquake events that characterize theground motions on the west coast of the contiguous United States. The data was filteres to according to the following criteria:

- Earthquake sequence
- Moment Magnitude $M_w \geqslant 5$
- $PGA > 0.04$
- $PGV > 1$ cm/s
- $Vs_{30} > 100m/s$ & $Vs_{30} < 1000$ m/s
- Lowest usable frequency is less than 1Hz

- $R_{rup} < 60km$

From this data the major earthquakes found are the following, the earthquakes can be sumarized in Fig. 4.15 which show this earthquakes as moment magnitude Mw vs rupture distance (R_{rup}).

- Chi-chi
- Managua
- Livermore
- Northridge
- Duzce
- Mammoth lake

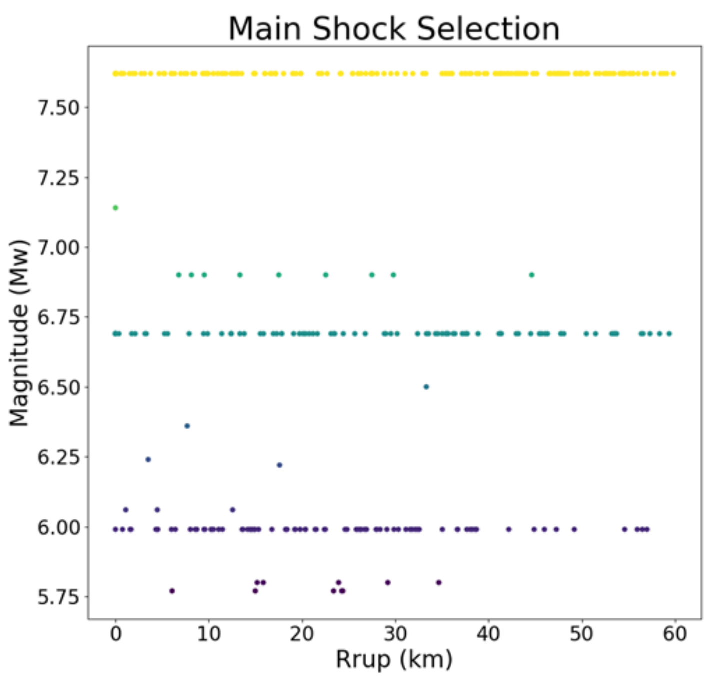

Figure 4.15 Mainshock selection from PEER NGA West2 Database

4.3.2 Discrete Modeling of Main Shock Series

The discrete modeling of mainshocks consists in using individual earthquakes that accour at different times thorughout the life of the structures which correlate to a Corrosion Level (CL), this can be done for each of the main shocks selected after which the follogin data is obtained and later analyzed:

- Maximum axial strain in Confined Concrete, Cover and Reinforcing steel Strains
- Obtain the probability of exceeding a given limit state $P(\varepsilon > \varepsilon_{LS}, IM)$
- The earthquakes are characterized according to an intensity measure

4.3.3 Multiple Main Shock Series

To simulate the life of a structure a Mainshowck series consisting of 3 Mainshocks for a the life of a structure is considered, three phases are considered:

1. at time $t = 0$ the structure has pristine conditions
2. Mainshock 1 occours
3. Mainshock 2 occours
4. Mainshock 3 occours

This is shown graphically in Fig. 4.16

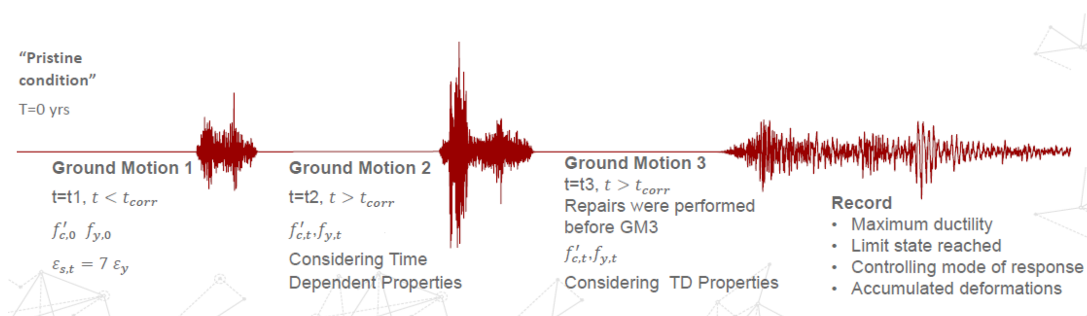

Figure 4.16 Mainshock sequence example

Then from the analysis the same results can be recorded

- Maximum axial strain in Confined Concrete, Cover and Reinforcing steel Strains
- Obtain the probability of exceeding a given limit state $P(\varepsilon\varepsilon_{LS}, IM)$
- The earthquakes are characterized according to an intensity measure

4.4 Future Topics

- Concrete Strength Aging
- Welding and Fatigue in Steel Structures
- Repair Effects
- Main Shock - After Shock Series - Repair Series
- Degree of damage effect on confined structures behavior

Chapter 5

Analytical Model and Preliminary Results

In this chapter first a framework for the analysis that is performed is presented, later the basic model that is used for the analysis is presented and later calibrated and verified with experimental data available in the literature. Finally preliminary results are presented that will define the general view for the proposed research

5.1 Analytical Model

5.1.1 Cantilever Column

This study focuses on the behavior of a Single Degree of Freedom System representing a Cantilever Reinforced Concrete Column. The column is modeled as shown in Fig. 5.1 This structure is modeled in OpenSeesPy [17][31] using the $forceBeamColumn$ element [24]. The forceBeamColumn element is used with two-point Gauss-Radau integration applied in the hinge regions and two-point Gauss integration applied on the element interior for a total of six integration points [24]. The force based formulation requires only a single element to accurately represent the full nonlinear deformation of the member and the integration scheme selected prevents the loss of objectivity during softening response while also providing integration points at the member ends [3],[24]. The element requires the length of plasticity be defined at each end of the member, for which the tension based rectangular plastic hinge length is calculated using the following expressions [9]:

$$L_{pc} = k * L_{eff} + 0.4D \tag{5.1}$$

$$k = 0.2 * (Fu/Fy - 1) \leqslant 0.08 \tag{5.2}$$

$$L_{pt} = L_{pc} + \gamma * D \tag{5.3}$$

For Single Bending

$$\gamma = 0.33 \tag{5.4}$$

The two-point Gauss-Radau integration is applied such that each end node integration is weighted equal to the specified plastic hinge length, as illustrated in Fig. 5.2. Therefore, strains recorded at the end sections represent accurate values even in the case where deformation localizes to the ends from strain softening behavior. For the case of the cantilever column considered, only one plastic hinge length is defined, and the opposite end is given an arbitrary unit length.

The section of the column is shown in Fig. 5.3, the section is discretized with concrete and steel material fibers. Concrete fibers are modeled using the $Concrete01$ material, modified for confined material strength based on the Mander confined concrete model [16]. The $Steel02$ material, based on the Giuffre-Menegotto-Pinto model [5] and it is used for the longitudinal reinforcement with recommended parameters ($b = 0.01, R0 = 20, cR1 = 0.925, cR2 = 0.15$).

5.1.2 Strain Penetration Component

The strain penetration is necessary to be considered to take into account the additional deformation due to anchorage of the reinforcement into the foundation, since the strains of tension in the reinforcement will drop to zero at a depth equal to the true development length of the rebar [22]. Experimental studies have generally reported that this end rotation contributes up to 35% to the lateral deformation of flexural members[30] and it is therefore important to incorporate into the analytical model. A way to capture this effect is by using a zero-length section element implemented in nonlinear fiber-based analysis of concrete structures, this is available in the material library of OpenSeesPy as $BondSP1$ [30] this is material model used for the steel fibers of the zero-length section element.

The required parameters for this model are:

- F_y Yield strength of the reinforcement steel
- S_y Rebar slip at member interface under yield stress (see Eq. 5.5)

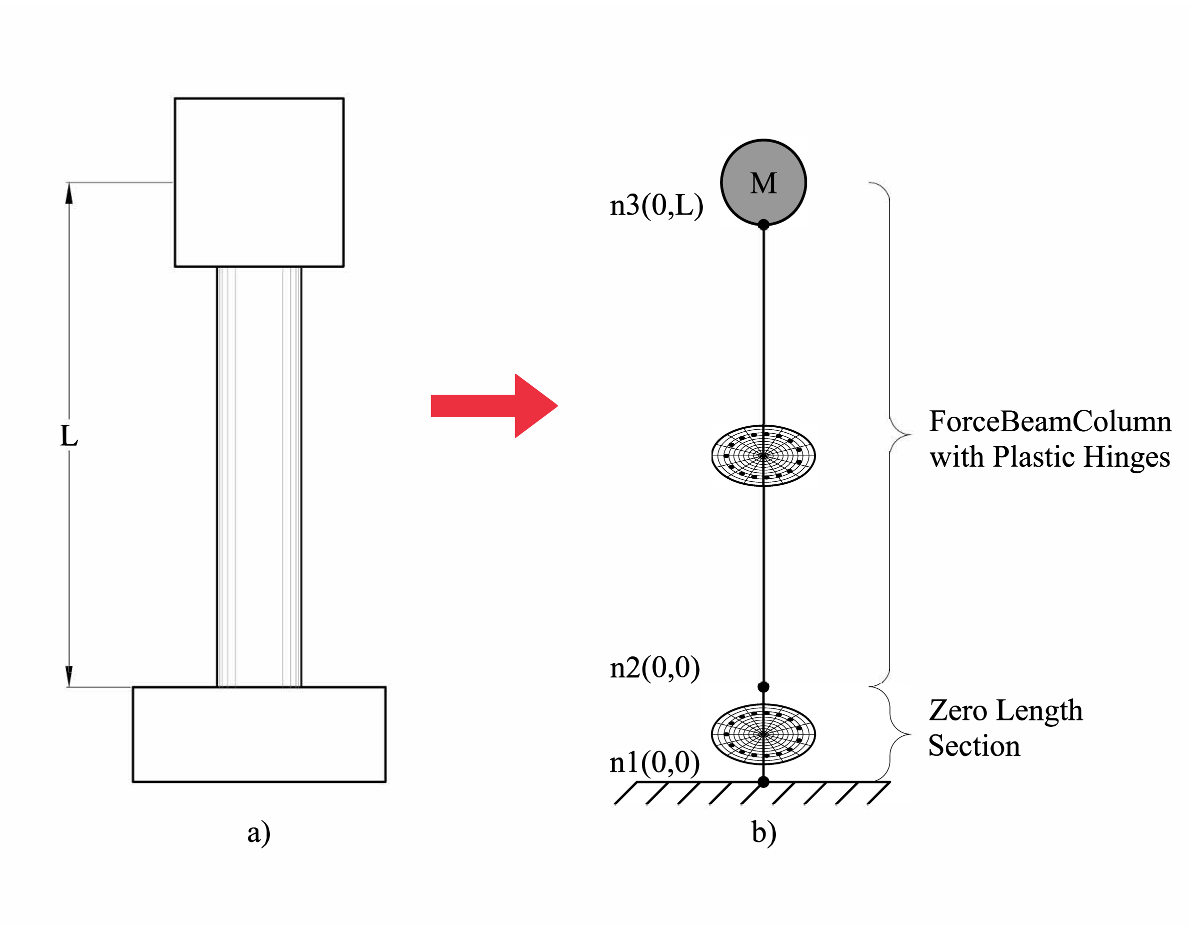

Figure 5.1 Structural Model a) SDOF Column b) Structural Model

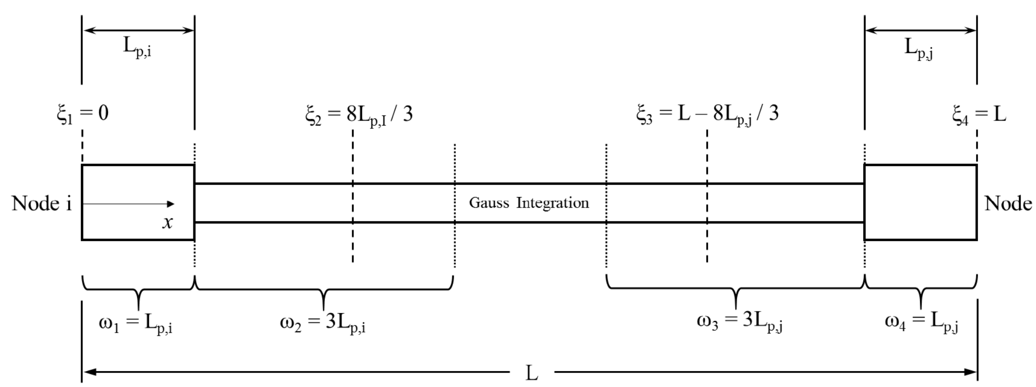

Figure 5.2 End point plastic hinge method [24]

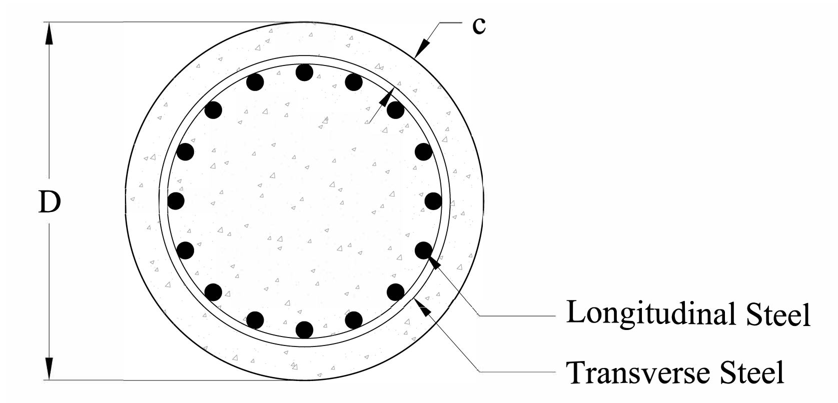

Figure 5.3 Section of the RC Column

- F_u Ultimate strength of the reinforcement steel
- S_u Rebar slip at the loaded end at the bar fracture strength a value of $35S_y$ is recommended [30]
- b Initial hardening ratio in the monotonic slip vs. bar stress response $b = 0.45$ is recommended [30]
- R Pinching factor for the cyclic slip vs. bar response $R = 1.01$ is recommended [30]
- d_b Rebar diameter
- $f'c$ Concrete compressive strength of the adjoining connection member
- α Parameter used in the local bond-slip relation and can be taken as $\alpha = 0.4$ in accordance with CEB-FIP Model Code 90 [4]

Bar slip is calculated as:

$$S_y(in) = 0.1\left(\frac{d_bF_y}{4000\sqrt{f'_c}}(2\alpha+1)\right)^{\frac{1}{\alpha}} + 0.013(in) \tag{5.5}$$

5.1.3 Design Limit States

Design limit states are defined on the basis of strains in the material since they can more accurately represent the different performance level of a structure. Structure limit states are defined for tension strains in the rebars or compression strains in the concrete core. The values recommended in typical performance based design of reinforced concrete bridge columns are shown in Table 5.1. The serviceability limit states correspond to the compression strain at which concrete cover begins to crush and the peak tension strain which results in residual crack widths of approximately 1 mm. These limits are generally accepted as nominal limit states for RC members. The compression limit state for damage control is defined by the expresion shown in Eq. 5.6 and it refers to the compression strain in the confined concrete at which fracture of the transverse reinforcement confining the core occurs [22]. This equation is obtained using the strain-energy balance between that absorbed by the confined core concrete and the capacity of the confining steel. The tension damage control limit state is defined by the strain at the onset of buckling which can be expressed according to 5.7, this equation demonstrated accurate predictions of the onset of bar buckling on physical tests in SDOF Concrete Column [10].

$$\varepsilon_{c,spiralyield} = 0.009 - 0.3\frac{A_{st}}{A_g} + 3.9\frac{f_{yhe}}{E_s} \tag{5.6}$$

$$\varepsilon_{s,BB} = 0.03 + 700\rho_s\frac{f_{yhe}}{E_s} - 0.1\frac{P}{f_c' A_g} \tag{5.7}$$

Table 5.1 Design Limit States

Limit State	Concrete Limit State $\varepsilon_c(in/in)$	Reinforcing Steel Limit State $\varepsilon_s(in/in)$
Serviciability	0.004	0.015
Damage Control	Eq. 5.6	Eq. 5.7

5.2 Comparison with existing physical Tests

5.2.1 Pristine Condition Columns

One of the tests performed at NC State by Goodnight et al [9] is used to calibrate the parameters used in the analytical model. The

5.2.2 Accelerated Corrosion Columns

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5.3 Analytical Framework

An overall analytical framework is established such that several analysis can be performed. From this analysis it is possible to determine the effects of damage in the performance of structures. The proposed analytical framework consists in:

1. Geometrical Properties of the SDOF column
2. Properties of the material are evaluated (i.e. water to cement ratio, cover)
3. For equal periods of time the Time Dependent Properties are modified
4. Nonlinear Time History Analysis are performed for discrete events or sequence of events
5. Results are obtained and evaluated

This procedure has been summarized in the form of a flow chart presented in Fig. 5.4

5.4 Earthquake selection

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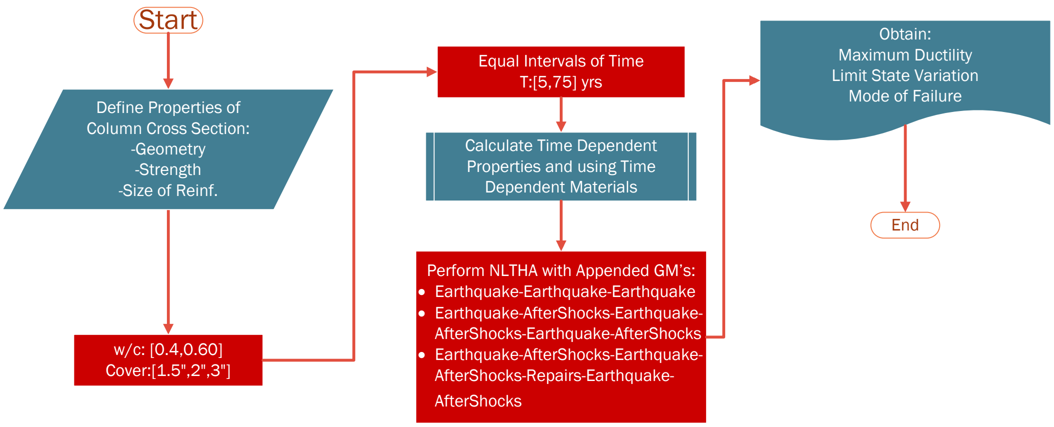

Figure 5.4 Analysis Framework Flowchart

5.5 Results from NLTHA

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5.5.1 Effect on structure response

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5.5.2 Effect on material response

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5.5.3 Preliminary results

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APPENDIX

Appendix A

LOREM IPSUM

A.1 A First Section

A.1.0.0.1 Filler Text

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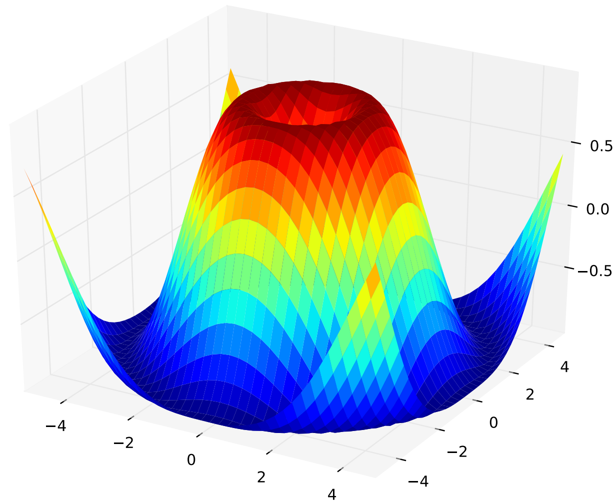

Figure A.1 A figure in the appendix.

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A.2 A Second Section

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