

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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DEDICATION

To my parents. To God.

BIOGRAPHY

The author was born in land far away where the earth rocks like a hammock. That land name is El Salvador. . . .

ACKNOWLEDGEMENTS

I would like to thank Dr. Kowalsky for his help. The Staff and Students at The Constructed Facilities Lab of NC State. The Alaska Department of Transportation. . . .

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Chapter 1

MOTIVATION

Bridges are designed based on discrete events with minimal consideration of interactions between hazards/loading or aging and bridge performance. Therefore a Time Dependent Performance Based Design is proposed to take into account the effects of time and damage in the properties of the materials as a function of time like corrosion effects, strain aging, creep and strength aging; Also since there is a high likelihood for a structure to be subjected to more than one main shock through the life of a structure in regions of high seismicity it is dimmed important to consider the effects multiple earthquakes, as a consequence the effects of repairs on the structural response are also of great importance. A procedure to implement these effects into the analysis of a bridge will be proposed and recommendations on changes that may be made in the design phase to improve the behavior of a bridge during its service life.

1.1 Properties that change with time

- Corrosion
- Strain Aging
- Concrete Strength
- Creep
- Fatigue
- Repairs

1.2 Corrosion

A structure that is originally designed to meet code specifications may not have the same margin of safety once the structure has undergone significant corrosion.

Research on the cyclic behavior of Corroded RC Columns [Yua17] have shown that at high corrosion levels as high as 35% the yield strength of the columns decreased by 28% and the ultimate strength decreased by 30%, also the ductility is reduced by as much as 18%. This research was performed by fixing the percentage of corrosion and then run physical tests, then the properties of the materials are modified using empirical equations developed from the tests. While the results appear to correlate well with the tests, this model is not time dependent.

There are time dependent models which first determine the time at which corrosion starts and then a time dependent function in which the diameter of the bar is simulated to reduce its section with time [YR98], [Cho08], [TC], [VS00].

1.2.1 Time to Corrosion

Christensen Model

Christensen model is based on a probabilistic approach to determine the time to corrosion based on the likelihood of the phenomenon to occur at a certain time in the life of a structure.

$$T_{corr} = X_I \left[\frac{d_c^2}{4k_e k_t k_c D_0 (t_0)^n} \left[\text{erf}^{-1} \left(1 - \frac{C_c r}{C_s} \right) \right]^{-2} \right]^{\frac{1}{1-n}} \quad (1.1)$$

X_I Model Uncertainty

d_c Depth of reinforcement

$k_e k_t k_c$ Factors that take into account test methods, curing and environment

D_0 Diffusion

C_s Water to binder ratio $C_s = A_{cs} \frac{w}{b} + \varepsilon_{cs}$

Gosh & Padgett Model

The Gosh & Padgett model is more practical in the sense that it depends on know variables for the concentration of corrosion on the surface of concrete and the internal concentration to calculate the time to corrosion. The model main variable is the Concrete Cover.

$$T_{corr} = \frac{x^2}{4D_c} \left[\text{erf}^{-1} \left(\frac{C_0 - C_{cr}}{C_0} \right) \right]^{-2} \quad (1.2)$$

Liu & Weyers Model

$$T_{cr} = \frac{W_{crit}^2}{2k_p} \quad (1.3)$$

$$W_{crit} = \rho_{rust} \left[\pi \left[\frac{C f'_t}{E_{ef}} \left(\frac{a^2 + b^2}{a^2 - b^2} + \nu_c \right) + d_o \right] D + \frac{W_{st}}{\rho_{st}} \right] \quad (1.4)$$

$$k_p = 0.098 \left(\frac{1}{\alpha} \right) \pi D i_{corr} \quad (1.5)$$

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

$$a = \frac{D + 2d_o}{2}$$

$$b = C + \frac{D + 2d_o}{2}$$

1.2.2 Rate of corrosion (Vu et al.)

To estimate the loss of steel cross section due to corrosion a time dependent corrosion rate model was developed by [VS00], this model implies that corrosion diminishes with time since as corrosion accumulates with time around the steel, it precludes uncorroded steel to react with the environment. The model is shown in Eq. 1.6.

$$i_{corr} = \frac{37.5(1 - w/c)}{d_c} \quad (1.6)$$

w/c : Water Cement ratio d_c : Cover depth

In Fig. 1.1 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.

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

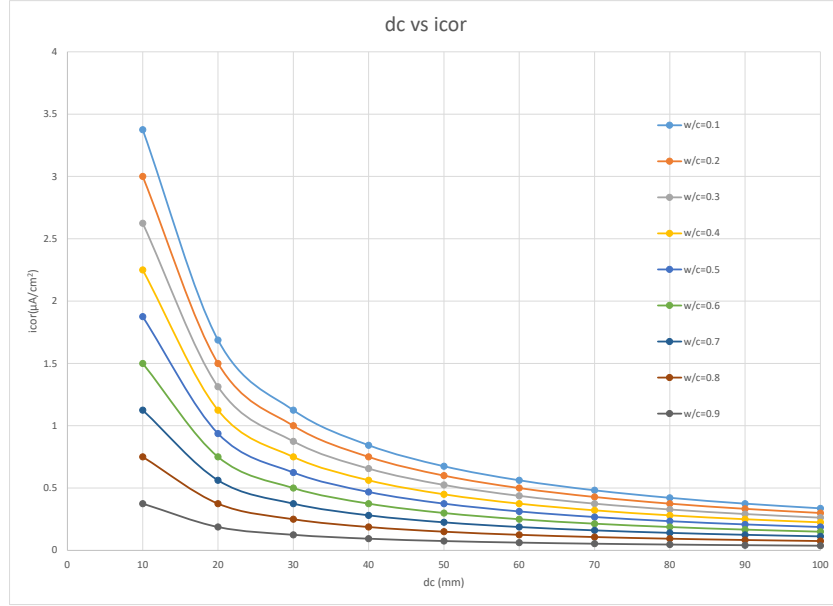


Figure 1.1 Concrete cover depth vs rate of corrosion

$$d_{corr} = d_{bi} - \frac{1.0508(1 - w/c)}{d_c} (t - t_{corr})^{0.71} \quad (1.7)$$

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

The diameter is plotted in Fig. 1.2.

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:

$$C = \frac{G_o - G}{g_o l_o} * 100 \quad (1.8)$$

Then the Corrosion level is plotted as a function of time in Fig. 1.3

1.2.3 Corrosion modified properties of reinforcing steel bars

In a study presented by Yuan et al [Yuan2000] it was shown from experimental results that the mechanical properties of steel for different levels of corrosion could be modified for analysis as follows:

$$f_{y,C} = f_{yo}(1 - 0.021C) \quad (1.9)$$

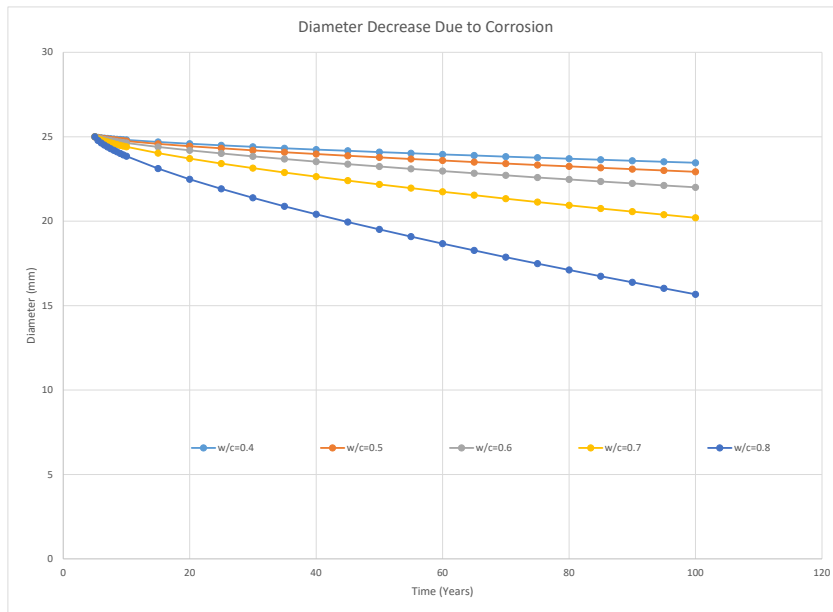


Figure 1.2 Diameter decrease due to corrosion

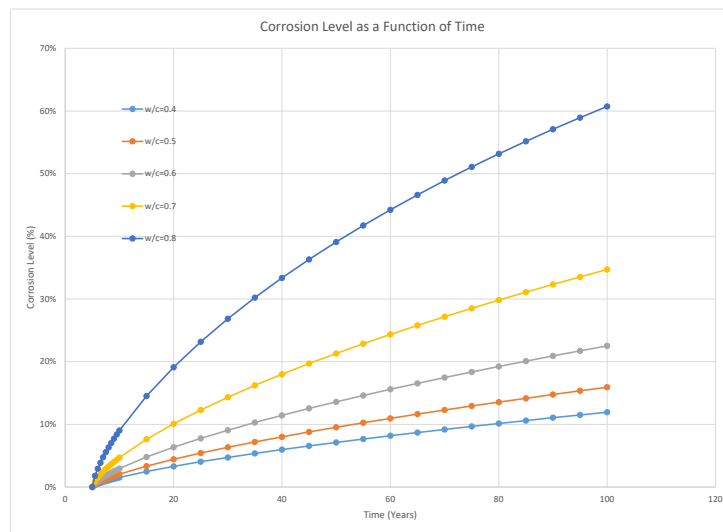


Figure 1.3 Corrosion Level vs Time (years)

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

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

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

1.3 Steel Strain Aging

1.3.1 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 from 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.

1.3.2 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 these regions of the structure. Furthermore strain aging will cause an increase 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 [RP94] most strain aging occurs in the first 37 days. Also [Mom09] 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 than $15\varepsilon_y$ are unpractical and not studied by Montahan et al [Mom09].

Montahan et al Strain Aging Effects in Yield Strength of Steel

Montahan 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:

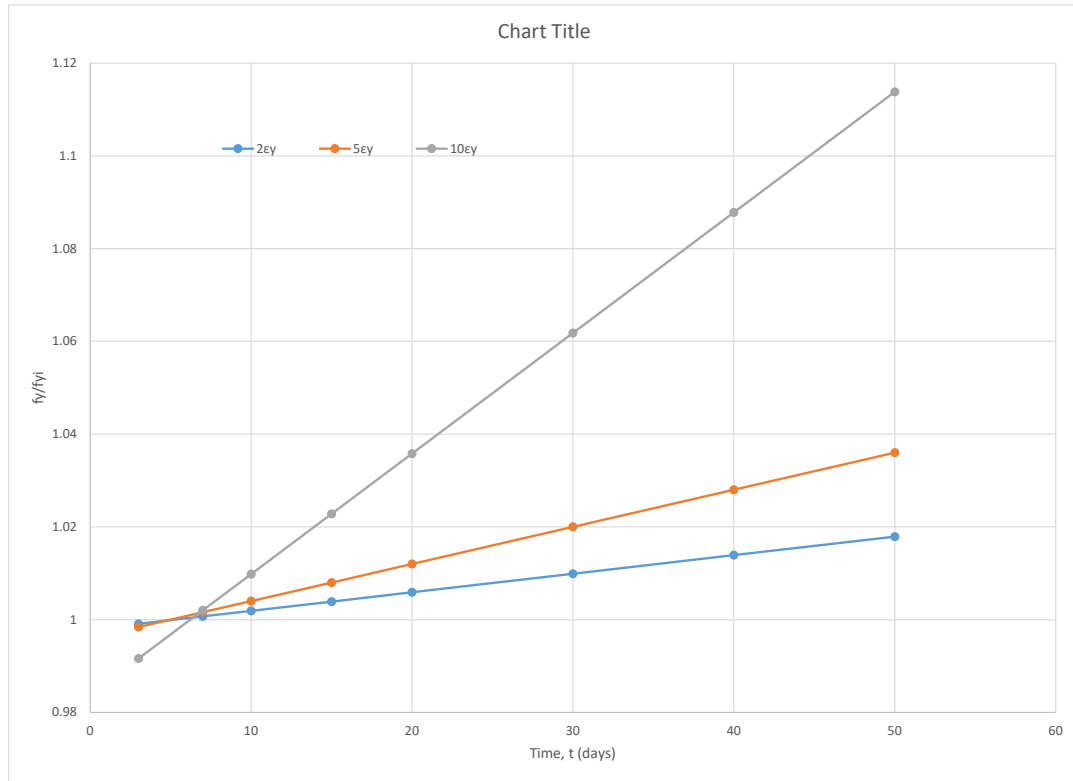


Figure 1.4 Strain Aging effect on Yield Strength vs Time (days)

For $10\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0026t + 0.9838 \quad (1.10)$$

For $5\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0008t + 0.996 \quad (1.11)$$

For $2\varepsilon_y$

$$\frac{f_y}{f_{yi}} = 0.0004t + 0.9979 \quad (1.12)$$

It is proposed to limit the increase in yield strength to the one obtained at 50 days. These equations are plotted in Fig. 1.4

1.4 Concrete Strength

1.5 Welding and Fatigue in Steel Structures

1.6 Repair Effects

1.7 Multiple Seismic Events

1.7.1 Main Shock Series

1.7.2 Main Shock - After Shock Series

1.7.3 Main Shock - After Shock Series - Repair Series

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APPENDIX

Appendix A

LOREM IPSUM

A.1 A First Section

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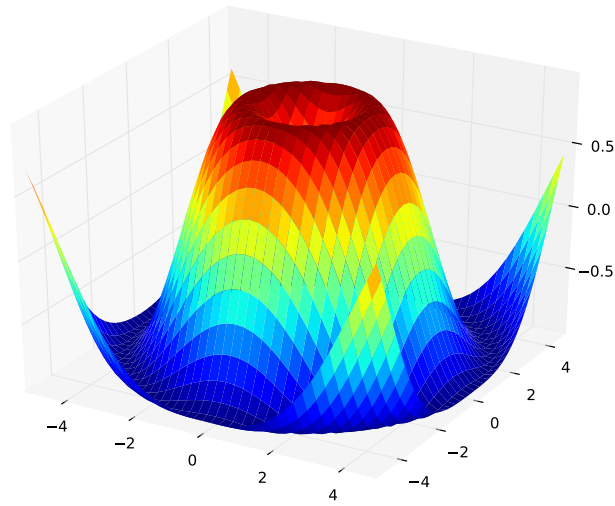


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A.2 A Second Section

Etiam ac leo a risus tristique nonummy. Donec dignissim tincidunt nulla. Vestibulum rhoncus molestie odio. Sed lobortis, justo et pretium lobortis, mauris turpis condimentum augue, nec ultricies nibh arcu pretium enim. Nunc purus neque, placerat id, imperdiet sed, pellentesque nec, nisl. Vestibulum imperdiet neque non sem accumsan laoreet. In hac habitasse platea dictumst. Etiam condimentum facilisis libero. Suspendisse in elit quis nisl aliquam dapibus. Pellentesque auctor sapien. Sed egestas sapien nec lectus. Pellentesque vel dui vel neque bibendum viverra. Aliquam porttitor nisl nec pede. Proin mattis libero vel turpis. Donec rutrum mauris et libero. Proin euismod porta felis. Nam lobortis, metus quis elementum commodo, nunc lectus elementum mauris, eget vulputate ligula tellus eu neque. Vivamus eu dolor.

Nulla in ipsum. Praesent eros nulla, congue vitae, euismod ut, commodo a, wisi. Pellentesque habitant morbi tristique senectus et netus et malesuada fames ac turpis egestas. Aenean nonummy magna non leo. Sed felis erat, ullamcorper in, dictum non, ultricies ut, lectus. Proin vel arcu a odio lobortis euismod. Vestibulum ante ipsum primis in faucibus orci luctus et ultrices posuere cubilia Curae; Proin ut est. Aliquam odio. Pellentesque massa turpis, cursus eu, euismod nec, tempor congue, nulla. Duis viverra gravida mauris. Cras tincidunt. Curabitur eros ligula, varius ut, pulvinar in, cursus faucibus, augue.