AN ABSTRACT OF THE THESIS OF

Jill Louise Folkestad for the degree of Honors Baccalaureate of Science in Civil Engineering presented on March 8, 2010. Title: Carbonation of Mid-Twentieth Century Reinforced Concrete Bridges in Oregon

Abstract Approved: _____

Christopher Higgins

Carbonation of concrete is of concern for reinforced concrete bridges. While long-term environmental deterioration of structures is unavoidable, protective design allows a structure to fulfill its full service life without any significant serviceability or strength losses due to deterioration. Understanding the process of concrete carbonation and the effect it has on structural performance allows for the proper design of concrete protection. The tests performed in this project determine in-situ carbonation depths of bridge core specimens retrieved from highway and interstate bridges in Oregon that were constructed in the mid-twentieth century. Further, the development of an accelerated carbonation chamber provides a controlled environment to determine an approximate carbonation rates. Combining field exposure with controlled laboratory results, carbonation can be better characterized in terms of climate, atmospheric carbon dioxide concentration, time, and concrete properties.

Key Words: Carbonation, corrosion, Reinforced Concrete, accelerated testing

Carbonation of Mid-Twentieth Century

Reinforced Concrete Bridges in Oregon

by

Jill Louise Folkestad

A PROJECT

submitted to

Oregon State University

University Honors College

in partial fulfillment of
the requirements for the
degree of

Honors Baccalaureate of Science in Civil Engineering (Honors Scholar)

Presented March 8, 2010
Commencement June 2010

Honors Baccalaureate of Science in Civil Engineering project of Jill Louise Folkestad presented on March 8, 2010.

APPROVED:

_____

Mentor, representing Civil Engineering

_____

Committee Member, representing Civil Engineering

_____

Committee Member, representing Civil Engineering

_____

Chair, School of Civil and Construction Engineering

_____

Dean, University Honors College

I understand that my project will become part of the permanent collection of Oregon State University, University Honors College. My signature below authorizes release of my project to any reader upon request.

_____

Jill Louise Folkestad, Author

Table of Contents

List of Figures

List of Tables

Carbonation of Mid-Twentieth Century Reinforced Concrete Bridges in Oregon

Introduction

Structural engineering consists of more than just the basic mechanics of design. The engineer is responsible for the design of the structure for the entirety of its life-span. This includes the consideration of structural deterioration from external factors such as environmental exposures. In reinforced concrete structural design corrosion of reinforcing steel cause much of the service life deterioration that is seen in modern day structures. Corrosion can be attributed to both chloride exposure and carbonation of the concrete.

In order to design a structure to withstand these environmental degradation mechanisms, it takes an understanding of the material properties and the effects of particular environments. Certain atmospheric conditions in some climates have harsher impacts on concrete than others. For example, when constructing in humid, warm locations such as the tropics, carbonation rates of concrete may be increased (depending on the concentration of atmospheric carbon dioxide gas in the air). Although the general process of carbonation is known, there are many factors that affect the depth and rate of carbonation penetration in concrete. Therefore, developing a better understanding of the impact of all these factors is helpful for structural design. When all the factors are known, a design can be implemented to adequately protect the structure for the entire duration of its service life.

In-situ carbonation depth testing and accelerated carbonation techniques allow a better understanding of the process of carbonation of concrete. Analyzing bridge specimens that have fulfilled their service life in particular climactic locations in the state of Oregon provides insight into the actual amount of carbonation that has occurred in realistic structures. The in-situ carbonation depth in the bridge specimens determines the risk that the bridge had of either being in the initiation phase of structural degradation or in propagation phase, where the reinforcing bar may start to corrode and the structure may begin to lose structural capacity.

The accelerated carbonation test is used to minimize the variability in the atmospheric conditions affecting carbonation (e.g. temperature, humidity, and CO_2 concentration). Once that is established, the main difference for the specimens is the concrete mixture-design and strength. The results from the accelerated carbonation test will determine if the test is a feasible design tool, how the concrete strength relates to carbonation, and how well accelerated tests correlate to field exposure. It is assumed that lower density concrete with more air voids and lower strength may experience substantially more carbonation than concrete of higher quality. Accelerated carbonation testing will determine approximately how substantial that difference in carbonation depth really is.

Background

Reinforced Concrete

Concrete has been used as a building material for many centuries, dating back to before the time of the Roman Empire. Methods for reinforcing concrete were later developed. Today, the use of reinforced concrete has advanced substantially, becoming one of the most popular building materials used world-wide. Concrete is known for its strength in compression, and steel is known for its strength in tension; therefore, providing a very versatile composite material. Also, "the universal nature of reinforced concrete construction stems from the wide availability of reinforcing bars and of the constituents of concrete (gravel, or crushed rock, sand, water and cement)... and from the economy of reinforced concrete compared with other forms of construction" (Wight, 2009, p. 1).

With the understanding that reinforced concrete is a very versatile, economic, and durable composite material, it is important to emphasize careful design and construction procedures to optimize the material's use. Although reinforced concrete is well known for its tremendous durability, many factors may cause premature degradation of the material. For example, freeze-thaw cycling, temperature cycling, moisture, solar radiation, acid attack, sulfates, chlorides, deicing salts, and reactive aggregates all cause deterioration of reinforced concrete structures. However, proper design and detailing protects the structure from these sources of degradation.

Most environments where reinforced concrete bridges exist are harsh on the structures; therefore, rather than avoiding such environments, the structure must be designed to withstand them. In order to protect reinforced concrete structures, many options are implemented. For example, concrete cover protects the steel reinforcing bars from the direct environment and concrete provides a passive chemical protection. Concrete coatings such as silanes and epoxy sealants are sometimes applied to supply additional protection.

Overall, reinforced concrete is a widely used material on a multitude of structures. For the purpose of this project, reinforced concrete bridges will be the focus in terms of structural systems.

Reinforced Concrete Bridge Design

In the late nineteenth and early twentieth centuries, bridges designed of reinforced concrete became more popular. With the advent of the material, more and more possibilities were available for bridge design with reinforced concrete. The composite use of material allows for a very functional, yet aesthetic design. Reinforced concrete allowed the intersection of beauty, economy, and strength (Liebenberg, 1992, p. 11).

There are many benefits to using reinforced concrete for bridge construction. For example, reinforced concrete is a very strong composite material. This material allows for very high compressive strength provided by the concrete and tensile

resistance provided by the reinforcing steel. With the composite section of both concrete and steel, a bridge made of this material can see almost any loading pattern if adequately designed by the engineer. Not only is this material extremely functional for engineering purposes, it is durable against the environment if properly designed to provide the necessary protection against the elements. So far, the functionality of the material has been discussed; however, it also offers aesthetic appeal. Reinforced concrete has the ability to be formed with an architectural façade to allow aesthetic appeal. Also, the simple image of the structural components of a bridge offers a grandiose view (Liebenberg, 1992, p. 52-53).

Reinforced concrete has provided the ability for bridges to span irregular areas, in irregular shapes because of the versatility of the material. The fact that concrete can be formed into any shape opens up many possibilities for design. Many architecturally extravagant designs have been performed with concrete because of the ability to build the forms for pouring, rather than being restricted to any one material section.

Although this material produces a lot of versatility, it can make the design process more extensive. Since many new designs are not "typical," new design strategies have to be formed in order to accurately analyze and design the structure. A certain innovation in design must be established; however, "the adaption of existing design concepts, configurations and details in design, to achieve the objectives and requirements of specific structural projects, constitutes a very large percentage of the work executed in practice" (Liebenberg, 1992, p. 57). It is the job of the engineer to

understand the limits of the structure and design the bridge to fulfill its serviceability requirements for the entirety of its service life.

The economic benefits of reinforced concrete also make it an exceptional building material for bridges. The composite material provides the capability of supporting large loads without excessive material. If the reinforcing steel is placed in such a way as to follow the moment diagram (or follow the tension sections of the structure) then the material cost can be minimized. This was especially important with the advent of the interstate highway system. More and more bridges were needed to build the transportation systems in the United States; therefore, having a very flexible, yet affordable material was essential. The state of Oregon's section of Interstate 5 was opened in 1966. This new passage allowed travel through Oregon much easier and faster. Although this entire section did not open until 1966, certain parts were constructed and used prior to this date. Many reinforced concrete bridges were built in Oregon at this time to provide access for all parts of the highway system.

With the onset of modern techniques, approaches, and philosophies in reinforced concrete bridge design, it is time to evaluate the bridges that were first constructed on the highway corridors. Replacement and renovation of the older bridges is necessary to keep the highway systems safe and functioning. Although the composite material of reinforced concrete is very durable, modern design has worked to perfect the durability and better understand the factors that may cause deterioration in the bridge structures.

Bridge service life is very important to understand in terms of reinforced concrete bridge design. Every bridge is designed with the intent to be used for a particular period of time before rebuilding is necessary. There are two categories in which service life of a bridge can be assessed: functional life and structural life. Functional life describes the capacity of traffic that the bridge can handle. Structural life describes the serviceability and safety of the structure. For the case of this study, structural life is the main focus for bridge service life. The life of a bridge is determined by many factors, such as environment, loading criteria, material selection, etc., and there are many ways in which that life can be optimized (Branco, 2004, p. 11-14).

In terms of environmental deterioration, such as carbonation of concrete, protection to the bridge must be provided to sustain the bridge throughout the entire span of its service life. Therefore, bridge design is an integral process to create a structure that is durable, strong, economical, and aesthetic.

Carbonation of Concrete

There are many factors that affect the deterioration of reinforced concrete structures. A main factor that is becoming increasingly important in urban environments is carbonation. Carbonation occurs with the presence of atmospheric carbon dioxide in the air. Although carbonation does not significantly alter the strength of the concrete itself, it can have detrimental effects on the reinforcing steel.

Carbonation of concrete is the process by which atmospheric carbon dioxide reacts with calcium hydroxide, a hydrated phase of Portland cement paste. With water acting as a catalyst, these two chemical components react to form calcium carbonate and a water byproduct. The chemical reaction is written as follows:

$$CO_2 + Ca(OH)_2 \xrightarrow{\text{H}_2\text{O}} CaCO_3 + H_2O$$
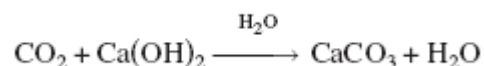
In order for this reaction to take place, the presence of water is essential; however, if the concrete specimen is saturated (meaning there are no air voids) the reaction will be much slower due to the inability of carbon dioxide gas to penetrate the concrete surface. Therefore, carbonation occurs readily when there are plenty of air voids for the carbon dioxide gas to penetrate the pores. Once the carbon dioxide gas has penetrated the concrete, dissolution in water occurs and the reaction takes place.

The process of carbonation happens at a very slow rate; therefore, in most cases it is not an issue. In typical environments, carbon dioxide content "varies from 0.03% by volume in rural air to 0.3% in large cities" (da Silva 2009). Although this is a very slow process, most structures are designed for a very long life span. Throughout this life span, the structure will develop a depth of carbonated concrete on the cover portion of the concrete. In design, a minimum cover of concrete is required to protect the reinforcing bars; however, if this cover is breached, the structure could be at risk. Therefore, it is important to understand the effects of climactic location in determining likely carbonation results that a structure will see in its life.

Factors such as carbon dioxide concentration, humidity, and temperature play a significant roll in the carbonation process. The most ideal situation for carbonation is a relative humidity of 50-75% (Tesfamariam 2008, p. 708). This ensures the air voids are not fully saturated, but also enough water to spur the chemical reaction to form carbonated concrete. Temperature is also very important factor in carbonation. Room temperature (25 degrees Celsius) is a typical value for optimal temperature for concrete carbonation. Different regions have different climates; therefore, it is important to understand that certain structures might see increased effects of carbonation.

The reason that carbonation is such a great concern for reinforced concrete structures is the pH shift that it causes in the concrete. Concrete that is not carbonated typically has a pH value of 12.6. Once the concrete is carbonated, that pH value can fall as low as 9.0 (Da Silva 2009). When carbon dioxide penetrates concrete, carbonic acid forms; this is what lowers the pH value and depassivates the steel. The neutralization of the concrete doesn't necessarily have any adverse affects on the concrete itself; typically the steel reinforcing bar is the major concern.

The rate at which carbonation occurs is a very important aspect to consider in the design of reinforced concrete structures. The service life of a structure depends on all the factors that contribute to the deterioration of a structure. As discussed earlier, relative humidity and temperature play an extremely important part in the process of carbonation. Also, carbon dioxide concentration in the air is an important factor to consider. Several models have been developed to estimate the rate of carbonation in a certain location and climate.

The model presented by Tuutti provides a carbonation depth estimate in terms of time, carbon dioxide concentration in the atmosphere, carbon dioxide which has reacted in the concrete, and a carbon dioxide diffusion coefficient (Andrade, 1995, p. 160). With this model, engineers are able to predict the concrete cover depth that is necessary to protect the reinforcing steel throughout the service life of a structure.

Reinforced concrete is a very durable structural material; however, if not designed properly to consider all deterioration possibilities, the strength and serviceability of that structure will decline immensely. Carbonation of concrete is a very important factor to consider when designing, especially in humid locations that are very conducive for the carbonation reaction.

Determination of Carbonation Depth

There are many methods available to determine depth of carbonation in concrete specimens. This section will explain these methods to gain a better understanding of both carbonation of concrete and how depths can successfully be determined in the laboratory.

pH Indicator Solution

The first method that will be discussed is the use of phenolphthalein indicator solution to determine carbonation depth. This method provides a simple and quick way to determine carbonation depths on freshly faced concrete specimens. If the contact area that has been sprayed has a pH above 9.0 the solution will turn a purple-pink color, while any values of pH below 9.0 will appear colorless when sprayed with the solution.

It is very important to split the specimens rather than saw cut them to get a clean fracture rather than a smeared surface. If saw cutting is used, results may be skewed due to spreading of non-carbonated paste on carbonated concrete or visa versa. A split tensile test (per ASTM C 496) is one method that will provide a clean break and a fresh working surface to determine carbonation depth with an indicator solution.

Drawbacks of using phenolphthalein consist of errors in zones where the carbonation is partial. The indicator cannot clearly detect those zones, which may provide a significant source of error when testing specimens. Also, the phenolphthalein must be sprayed on the specimen immediately after fracture. To gather reasonable results, spraying the concrete within 15 minutes of splitting is essential. Although these drawbacks exist, the phenolphthalein indicator solution is a very economical, simple, and quick way to test for carbonation depth in specimens. The color difference is very vibrant and easy to see for taking measurements.

Thermal-gravimetric Analysis

Carbonation is analyzed by this method using material weight change. The specimens are heated and weighed, and based on the obtained weights, the material properties can be determined.

Using this method, concentrations of calcium hydroxide and calcium carbonate can be determined. Therefore, it's easy to see whether or not the specimen has undergone the transformation from non-carbonated concrete to carbonated concrete. This method requires very careful measurements and calculations and a firm understanding of chemistry. The availability of a drying oven is also necessary.

Other Methods

There are other methods to determine the depth of carbonation in a concrete specimen. For example, x-ray diffraction analysis and fourier transformation infrared spectroscopy are both used to perform this test (Chang, 2004, p. 1760). In choosing a test to perform, it is important to consider time, budget, and scope of the project. For a limited time scale and budget, the phenolphthalein pH indicator solution is a very viable solution. Also, the test specimens themselves may dictate the test that needs to be performed.

Corrosion of Reinforcing Steel

"Corrosion of reinforcing bars occurs in concrete structures due to chloride attack or concrete carbonation. This corrosion causes the reduction of steel section, cracking of concrete cover, and affects the safety and serviceability of the structures" (Broomfield, 1994, p.165). Preventative measures need to be made in the reinforced concrete design to consider the possible effects of corrosive behavior.

Concrete cover provides protection for the reinforcing steel. Concrete mixtures have a naturally protective chemistry for the steel bars; therefore, the cover depth acts as a shield against corrosive environments for the steel. Although this layer is typically designed to be sufficient, there are instances when the cover is too thin and reinforcing steel may be at risk. Certain environments are more vulnerable to the effects of corrosion and concrete penetrability. For example, humid and moist locations provide an ideal environment for corrosion. Humidity accelerates the rate of concrete

carbonation, thus introducing a risk for that concrete protection over the steel. Also, abundant concentrations of environmental carbon dioxide and presence of chloride may lead to increased rates of corrosion of steel. If the steel is exposed to the environment, corrosion is most certain to occur. Therefore, protecting the steel from these environmental impacts is essential in reinforced concrete design.

The geometry and composition of reinforced concrete sections play an extremely important role in the environmental deterioration of the structure. For example, once corrosion of the reinforcing bar occurs, the corroded layer expands. This expansion introduces an internal pressure on the concrete. The expansion can be as much as eight times the volume of the non-corroded steel (Bob, 1990, p.58). If the pressure is great enough, the concrete will begin to crack or even spall. Not only does the concrete lose strength, but the corrosion of the reinforcing steel causes a loss in the usable cross sectional area for the steel strength. Just as stated by Broomfield, this situation will eventually risk the safety and serviceability of the structure.

Figure 1 depicts the corrosion cycle of reinforcing steel.

LOSS OF DUCTILITY

INITIAL SECTION

ATTACK PENETRATION

LOSS IN CONCRETE BOND

RUST

VIRGIN STEEL

CRACKING

$\emptyset(t) = \emptyset(i) - 0.023\, I_{corr}\, t$
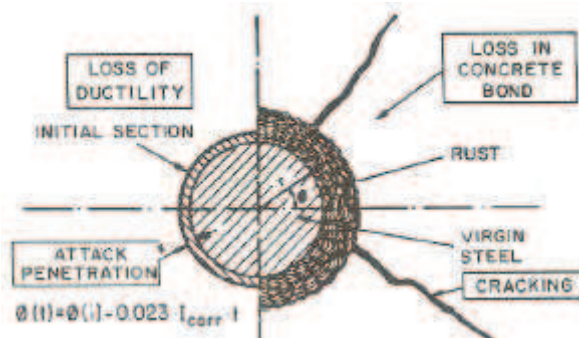
Figure 1—Diagram of Reinforcing Bar Corrosion
(Reprinted from "Progress on Design and Residual Life Calculation   with Regard to Rebar Corrosion of Reinforced Concrete," by C. Andrade & C. Alonso, 1996, *Techniques to Assess the Corrosion Activity of Steel Reinforced Concrete Structures*, p. 32. Copyright 1996 by ASTM)

According to Andrade and Alonso, deterioration in the reinforcement causes both "loss of load-bearing section and… loss of ductility" (1996, p. 32). The interface between the concrete and the reinforcing steel is subject to "loss in bar/concrete bond and… concrete cracking" (1996, p. 32).

Although reinforced concrete structures are well known for durability and unlimited exposure, it is the job of the engineer to design the structure to maintain that durability against an environment causing degradation to the structural materials.

There are different phases to the degradation of reinforced concrete structures. First, unnoticeable changes will occur in the interior sections of the concrete. This phase is called the initiation phase in which no structural weakening has occurred; however, the concrete cover may be penetrated by carbonation. The propagation phase is where the structure actually begins to loose strength, ductility, or serviceability. This is the phase in which corrosion of reinforcing steel occurs. Once this phase begins,

the deterioration of the structure occurs at an accelerated rate, and is typically difficult to slow down or stop (Rostam, 1995, p. 129).

Corrosion of reinforcing bars in concrete is something that can be prevented through proper design. Without the proper protection afforded by concrete cover or other methods, the reinforced concrete structure is potentially at risk of losing strength and/or serviceability. The amount of time and effort to provide concrete protection is very minimal, while the benefits can maximize the life-span of a structure.

Accelerated Carbonation

The use of an accelerated carbonation test allows concrete to be carbonated at a much faster rate than in actual atmospheric conditions. Essentially, the service life exposure (in terms of concrete carbonation) can be achieved in less than a year. This becomes very useful in terms of designing a structure to withstand the full extent of damage caused by carbonation.

In a laboratory accelerated carbonation testing chamber, it is preferable to maintain a carbon dioxide gas concentration of four percent (40,000 ppm). Actual carbonation concentration in real atmospheric conditions is approximately 0.04 percent (400 ppm). With CO_2 concentration being at a magnitude of 100 times greater than that of the atmosphere, the accelerated carbonation chamber is able to induce much larger amounts of concrete carbonation in less time.

The relative humidity of the system is optimal around 60 percent and at room temperature. Maintaining constant humidity is achieved through a liquid solution of

sodium nitrite (NaNO$_3$) and water in pans and a heat lamp. Keeping an air tight

chamber will provide better maintenance of the atmospheric conditions.

Once the accelerated carbonation test has completed the time span desired (one

week of accelerated carbonation can be approximated to one year of real exposure

time), carbonation depth data can be obtained using the phenolphthalein pH indicator

solution.

Objectives

- Develop a database of service life carbonation depths for a data set of reinforced concrete bridge cylinders.

- Develop an accelerated carbonation chamber to determine approximate carbonation rates and how well accelerated tests correlate to field experience.

- Use the results to determine plausible actions to be taken to protect existing bridges from environmental deterioration.

Testing Methodology

In-situ Carbonation Depths

The testing for concrete depth was performed using a pH indicator known as phenolphthalein. This indicator is a clear solution that turns purple when applied to non-carbonated concrete. The concrete that is carbonated will remain uncolored, signifying the depassivation of the concrete cover. This solution can be sprayed onto the fresh face of the concrete specimens to determine the carbonation depth.

The concrete core specimens were first sawn to a length of eight inches. Eight inches is required in order to retrieve split tensile results upon splitting of the cores. The cores are four inches in diameter, and therefore require eight inches of length per ASTM C496—Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens. Once the load at failure (P, lbf) is obtained, the splitting tensile strength in psi can be obtained from the following equation:

$$T = \frac{2P}{\pi LD}$$

Equation 2.1

Where "L" is the length of the specimen in inches and "D" is the diameter of the specimen in inches. Once the splitting tensile strength is found, it can be estimated that compressive strength is approximately (T/6.4)^2 (Wight, 2009, p. 58).

Once all of the cores (including nine control specimens) were cut at the required eight inch length, the cores were split using a split tensile machine. All the tensile strengths were recorded. Further calculations were performed post-testing (i.e. compression strength calculation).

The placement of cylinder in the split tensile machine is essential. Creating a symmetrically loaded area provides the best and most accurate results. The machine used was developed to mainly test the "standard" cylindrical specimens (6" x 12"); therefore, it was necessary to add steel plates in order to develop centroidal loading on the cylinder. It was also important to make sure that the cylinder is snug and well supported throughout the load application. Otherwise, eccentricities cause skewed results.

Observing the cylinder all throughout the load application process was required. Once the core appeared split, the load was removed and the test was complete. The maximum pound-force value was stored in a computer for recording purposes. From this load obtained at failure, the tensile strength was calculated.

Sources of error to note in *Split Tensile Test*:

- Some concrete cylinder specimens taken from the bridges have reinforcing bar remaining in the sample; therefore, the tensile strength of the concrete cannot be determined from this test. The core can still be split to be used for carbonation depth testing, but the strength will not be logged.

- Human error in placing and loading the cylinders results in possibly skewed results.

- The cylinder specimens were cored from in-situ bridges and not cured per ASTM C 511 for curing molded specimens. It is important to note that this test is performed to gather an approximate value for the concrete strength.

After the splitting of the cylindrical specimens, half of each core was tested for carbonation depth. The specimen was sprayed with the pH indicator solution within 15 minutes of splitting to ensure accurate results. The depth measurements were taken at three locations on each long side of the half-cylinder and once on the in-situ exposed face of the concrete. The measurements on the length-wise sides of the cylinder indicate the carbonation that has occurred from the time the cylinder was cored from the bridge. The face depth indicates the in-situ carbonation depth from the time of construction to present. Two concrete depth calculations/measurements were logged in the data—1: The gross carbonation depth of the exposed face and, 2: The net carbonation depth of the exposed face (gross depth less the average of the length-wise depths). It is important to note that the carbonation that has occurred from the time the cylinder was cored is not equal on the length-wise sides and the exposed sides. This is true because once a certain depth of carbonation has been reached the rate of carbonation slows down. It can be understood in terms of a breathable layer of concrete. The outer layer breathes more, and therefore is carbonated much faster. The further into the concrete, less oxygen is available; therefore, it can be said that very

little to no carbonation has happened post coring on the in-situ exposed face. Both results will be logged for completeness purposes; however, the gross results will be considered as realistic depths.

Visualization of testing procedures:

Figure 2 and Figure 3 depict the testing sequence for the concrete cylinder specimens→
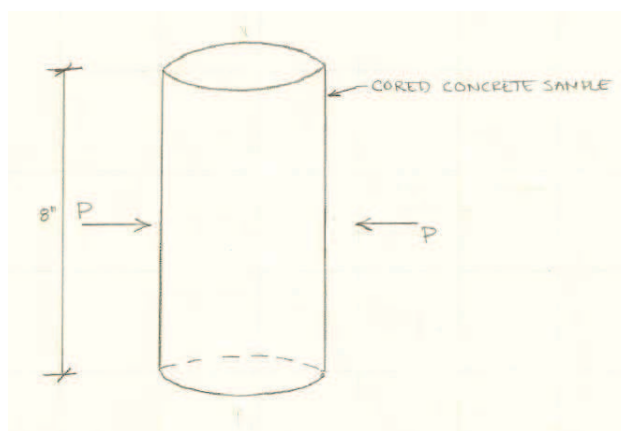
Figure 2—Perform Split Tensile Test
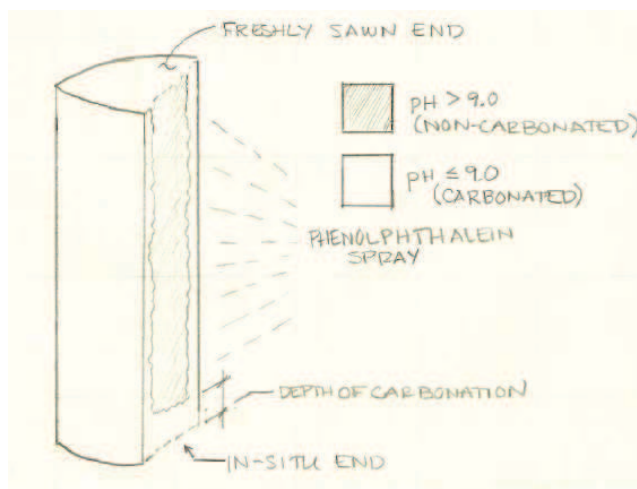
Figure 3—Application of Phenolphthalein indicator solution

Accelerated Carbonation Test

The use of an accelerated carbonation test will provide results on the rate of carbonation. This rate can be determined due to highly controlled conditions provided by a carbonation chamber. The carbonation chamber is constructed using two 55 gallon steel drums that are connected together via tubing. A CO_2 gas supply runs into the first chamber which contains several pans of $NaNO_2$ (Sodium Nitrite) and water solution. A heat lamp is also placed in this first chamber to provide necessary temperature to achieve relative humidity between 50 and 70 percent. The second chamber holds the concrete specimens on wire shelving. A fan is placed in this chamber to provide air circulation. The key to providing a controlled atmosphere is to place a CO_2 sensor in the second chamber. This sensor triggers the solenoid valve to open and let in CO_2 gas if the concentration goes below 4%. With this well-maintained system, carbonation of concrete specimens in the second chamber happens at an accelerated rate (McGrath, 2005, p. 1-8).

It is approximated that a CO_2 concentration of 4% provides an optimal amount for accelerated carbonation. If the CO_2 concentration is too high (i.e. approaching 100%), results for accelerated carbonation are not realistic, because carbonation also depends on the other atmospheric gases that are in the air. With this test, it is approximated that one week of accelerated carbonation is equal to one year of natural atmospheric carbonation (da Silva, 2009, p. 334). Therefore, an accelerated time of 30 days will provide approximately four years of natural carbonation. From this data, the

carbonation rate can be determined. Also, the results will provide a means for looking at how different concrete properties affect carbonation.

After the accelerated test takes place, the specimens will be removed from the chamber. Since the specimens are half-cylinders, splitting them to retrieve a clean surface cannot be done by saw-cutting. Therefore, the samples will need to be split by load application. The half-cylinder will be placed on two supports at either end of the previously split surface (resembling a simply supported beam). A point load will then be applied at the centerline of the beam until failure. Once the split is made, phenolphthalein solution can be applied to the fresh surface to determine accelerated carbonation depth.

The following figure depicts accelerated carbonation depth of a split specimen:
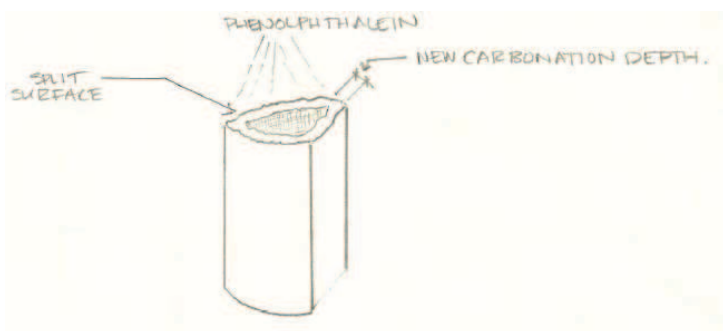
Figure 4—Determination of Accelerated Carbonation Depth

Determining the rate of carbonation from Accelerated Carbonation Results:

To determine the carbonation rate of each specimen, the Tuutti model was used. The following equation provides the "rate of advance of the carbonation front" (Andrade, 1995, p. 160):

$$\frac{C_s}{C_x} = \sqrt{\pi}\, \frac{x\sqrt{t}}{2\sqrt{D}}\, \exp \frac{x^2/t}{2\sqrt{D}}\, erf\left(\frac{x\sqrt{t}}{2\sqrt{D}}\right)$$

Equation 2.2

Where C_s = CO_2 concentration in the atmosphere, mol/kg

C_x = CO_2 which has already reacted with the concrete, mol/kg

D = CO_2 diffusion coefficient, m^2/s

x = Carbonation depth, mm

t = time, s

The error function (erf(x)) is two times the sum of the Gaussian distribution. All variables, except D are known. Therefore, with the results of the accelerated carbonation test, D can be solved.

Also, with time and depth as two variables in the equation, the rate is easily solved for when designing new reinforced concrete structures.

Laboratory Testing and Data

The following table provides the information for the core samples used in this study. The bridges are all in the state of Oregon, along state and United States highway systems.

Bridge No.	Bridge Name	DATE CORED	Construction Date	Location on Bridge	Geographic location	Oregon Region
07863 07863 07863 07863 07863	Row River Road Bridge	6/11/2008	1954	N. Westside Exterior N. Eastside Exterior N. Eastside Exterior Northside Interior Northside Interior	Row River Road, I-5, Cottage Grove	I-5 Valley
06768 06768 06768 06768	OR 58, Lost Creek	N/A	1946	Interior Span 4 Interior Span 4 Exterior Span 4 South Exterior Span 4 North	Lost Creek, 095 MI E HWY 1 JCT, State HWY OR 58 (HWY 018)	Valley
20041 20041 20041 20041 20041	I-5 South Bound	1/3/2008	1956	N/A	I-5 at London Road	I-5 Valley
OR 58 & US 57 OR 58 & US 57 OR 58 & US 57 OR 58 & US 57 OR 58 & US 57	OR 58 & US 57	10/15/2007	N/A	N/A	OR 58 & US 57	Valley
20039 20039 20039 20039 20039	London Rd Bridge, Bundle A04 North Bound	10/8/2007	1956	Interior Girder Interior Girder Exterior Girder Exterior Girder Exterior Girder	London Rd.	I-5 Valley
07491 07491 07491 07491 07491	US 26, Rush Creek Antone, Bundle A51	Summer 2007	1951	Girder Girder Girder Girder Girder	Bridge Creek, US 26 (HWY 041) In Mitchell	East/Central
08738N 08738N 08738N 08738N 08738N	Eagle Mill North Bound	4/13/2009	1962	Interior Interior Exterior Exterior Exterior	I-5 Northbound, Eagle Mill Rd, 0.6 Mile N of Ashland NCL	South West
20376 20376 20376 20376 20376	MJ Hughes Construction, Bundle 312, Brown Creek Bridge	7/24/2009	1958	West Exterior Girder West Exterior Girder West Exterior Girder West Interior Girder West Interior Girder	Wheeler County HWY 26	East/Central
20370 20370 20370 20370 20370	Saginaw Rd Overpass, Bundle 312	8/3/2009	1956	South Exterior Girder South Exterior Girder South Exterior Girder South Interior Girder South Interior Girder	Lane County	I-5 Valley
08742N 08742N 08742N 08742N 08742N	Bear Creek North Bound	4/27/2009	1962	Exterior Exterior Exterior Interior Interior	Bear Creek, 0.8 Mile N of Ashland SCL, I-5	South West
20549 20549 20549 20549 20549	Holm II-I-5 Louse Creek US 199	6/19/2009	1960	N/A	I-5 at Louse Creek	South West
08502 08502 08502 08502 08502	Hilgard	N/A	1958	Right Exterior Girder, Span 4 18' back bent 5 Right Interior Girder, Span 4 20' back bent 5 Left Interior Girder, Span 4 22' ahead bent 4 Right Exterior Girder, Span 3 20' back CL of bent 4 Left Exterior Girder, Span 5 25' ahead bent 4	Hilgard Interchange, OR 244 (HWY 341)	East
20551 20551 20551 20551 20551	Holm II	8/14/2009	1960	Interior Interior Interior Exterior Exterior	I-5 at Louse Creek	South West
07393 07393 07393 07393 07393	Columbia River Highway I-84, Crossing over Mosier Creek	4/29/2009	1951	Interior and Exterior Beams	Mosierr Creek	North (Columbia River)

Table 1—Bridge Core Database

The concrete specified in the bridge drawings under "General Notes" denotes that the concrete used for the bridges should be Class "A" mix with a compressive strength of 3300 psi. Although this is specified in the plans, it does not mean that all of the mixtures used for the various bridges (or even different locations on the same bridge) are guaranteed to have the same properties.

Each bridge had a select amount of core specimens tested for split tensile and then sprayed with a phenolphthalein indicator solution to determine in-situ carbonation depth. The following tables give the results that were obtained from the testing procedures.

Bridge No.	P, load at failure (lb)	fct, Tensile Strength (psi)	f'c, Compressive Strength (psi)
20370	10660	212.07	1098.03
20370	11725	233.26	1328.39
20370	14040	279.32	1904.73
20370	15390	306.17	2288.64
20376	17570	349.54	2982.94
20376	17570	349.54	2982.94
20376	17575	349.64	2984.63
20376	17580	349.74	2986.33
06768	12380	246.29	1480.95
06768	17565	349.44	2981.24
06768	18120	360.49	3172.61
06768	19275	383.46	3589.96
07393	20390	405.65	4017.30
07393	23285	463.24	5239.05
07393	27170	540.53	7133.12
07491	10600	210.88	1085.71
07491	11315	225.10	1237.11
07491	14575	289.96	2052.66
07863	11140	221.62	1199.14
08502	14270	283.89	1967.65
08502	14975	297.92	2166.88
08502	17010	338.40	2795.82
08502	17030	338.80	2802.40
08502	17175	341.69	2850.32
08738N	14215	282.80	1952.51
08738N	15135	301.10	2213.43
08738N	15610	310.55	2354.54
08738N	15820	314.73	2418.32
08742N	12615	250.97	1537.71
08742N	12935	257.33	1616.71
08742N	13890	276.33	1864.25
08742N	14835	295.13	2126.55
20041	13045	259.52	1644.33
20041	13740	273.35	1824.21
20041	16225	322.79	2543.72
20549	17185	341.88	2853.64
20549	17580	349.74	2986.33
20549	18760	373.22	3400.68
20549	20650	410.82	4120.41
20551	16895	336.12	2758.14
20551	17675	351.63	3018.69
20551	18350	365.06	3253.66
20551	20640	410.62	4116.42
OR 58 & US 57	16180	321.89	2529.63
OR 58 & US 57	20055	398.98	3886.38
OR 58 & US 57	20410	406.04	4025.19
OR 58 & US 57	28850	573.95	8042.52

Table 2—Concrete Strength of Core Specimens

	Carbonation Depth (in)						Face Depth	Face Depth
	Side 1			Side 2			(Gross)	(Net)
Bridge No.	1.1	1.2	1.3	2.1	2.2	2.3		
07863	0.55	0.3	0.4	0.38	0.35	0.3	1.45	1.07
06768	0.3	0.35	0.3	0.3	0.35	0.3	0.9	0.58
06768	0.3	0.4	0.4	0.3	0.35	0.3	0.95	0.61
06768	0.4	0.4	0.35	0.35	0.35	0.3	1.1	0.74
06768	0.4	0.4	0.4	0.3	0.35	0.35	0.9	0.53
20041	0.25	0.25	0.25	0.25	0.25	0.25	0.4	0.15
20041	0.2	0.2	0.2	0.2	0.2	0.2	0.5	0.30
20041	0.1	0.15	0.1	0.1	0.15	0.2	0.35	0.22
OR 58 & US 57	0	0	0	0	0	0	1.1	1.10
OR 58 & US 57	0	0	0	0	0	0	0.75	0.75
OR 58 & US 57	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
OR 58 & US 57	0	0	0	0	0	0	1.85	1.85
20039	0.55	0.35	0.5	0.3	0.3	0.5	1.5	1.08
20039	0.4	0.3	0.4	0.3	0.5	0.3	1.4	1.03
20039	0.4	0.35	0.35	0.3	0.5	0.3	1.35	0.98
20039	0.3	0.3	0.3	0.35	0.35	0.5	1.7	1.35
07491	0.3	0.25	0.2	0.25	0.25	0.25	1.85	1.60
07491	0.25	0.3	0.35	0.25	0.2	0.2	2.05	1.79
07491	0.35	0.3	0.35	0.35	0.25	0.3	1.35	1.03
08738N	0.2	0	0.2	0.1	0.15	0.2	1.45	1.31
08738N	0.15	0.2	0.2	0.15	0.05	0.05	1.6	1.47
08738N	0.05	0.05	0.25	0.05	0.2	0.2	1.85	1.72
08738N	0.2	0.25	0.2	0.15	0.1	0.25	1.7	1.51
20376	0.3	0.3	0.3	0.25	0.25	0.2	1.5	1.23
20376	0.3	0.25	0.35	0.35	0.4	0.45	1.45	1.10
20376	0.25	0.2	0.3	0.3	0.25	0.2	1.5	1.25
20376	0.2	0.2	0.2	0.2	0.2	0.25	1.5	1.29
20370	0.2	0.1	0.15	0	0	0.1	1.3	1.21
20370	0.12	0.15	0.12	0.1	0.1	0.1	1	0.89
20370	0.15	0.1	0.15	0.2	0.1	0.15	1.1	0.96
20370	0.1	0.15	0.05	0.15	0.2	0.1	1.6	1.48
08742N	0.2	0.3	0.3	0.15	0.2	0.25	0.85	0.62
08742N	0.2	0.15	0.25	0.1	0.15	0.2	0.9	0.73
08742N	0.15	0.2	0.2	0.2	0.2	0.15	0.9	0.72
08742N	0.2	0.1	0.2	0.2	0.2	0.15	0.85	0.68
20549	0.2	0.15	0.1	0.2	0.15	0.1	1.6	1.45
20549	0.3	0.3	0.3	0.1	0.1	0.1	1.7	1.50
20549	0.25	0.2	0.2	0.1	0.15	0.1	1.5	1.33
20549	0.1	0.1	0.1	0.2	0.2	0.2	1.6	1.45
08502	0.15	0	0	0.15	0.25	0.1	1.35	1.24
08502	0.5	0.4	0.4	0.4	0.35	0.35	1.6	1.20
08502	0.1	0.1	0.1	0.1	0.1	0.1	1.4	1.30
08502	0.1	0.1	0.1	0.05	0.05	0.05	1.3	1.23
08502	0.3	0.3	0.35	0.25	0.3	0.35	0.98	0.67
20551	0	0	0	0	0	0	1.2	1.20
20551	0.1	0.1	0.1	0.15	0.2	0.2	1.15	1.01
20551	0.1	0.1	0.25	0.2	0.15	0	1.1	0.97
20551	0.2	0.2	0.2	0.15	0.1	0.2	1.5	1.33
07393	0.1	0.1	0.1	0.2	0.2	0.2	0.5	0.35
07393	0	0	0	0.1	0	0	0.95	0.93
07393	0.1	0.2	0.1	0.1	0.2	0.1	0.85	0.72
Control							0.1	
Control							0.1	
Control							0.1	
Control							0.1	
Control							0.1	
Control							0.1	
Control							0.1	
Control							0.1	
Control							0.1	
Control							0.1	

Table 3—Carbonation Depth of Core Specimens

Results Analysis

Split Tensile

Figure 5—Split Tensile Test

Based on the approximate results obtained by the split tensile test, the core specimens are able to be classified in terms of concrete strength. The following figure shows the graphical depiction of the bridge cores and their corresponding concrete strengths.

Tensile Strength Distribution
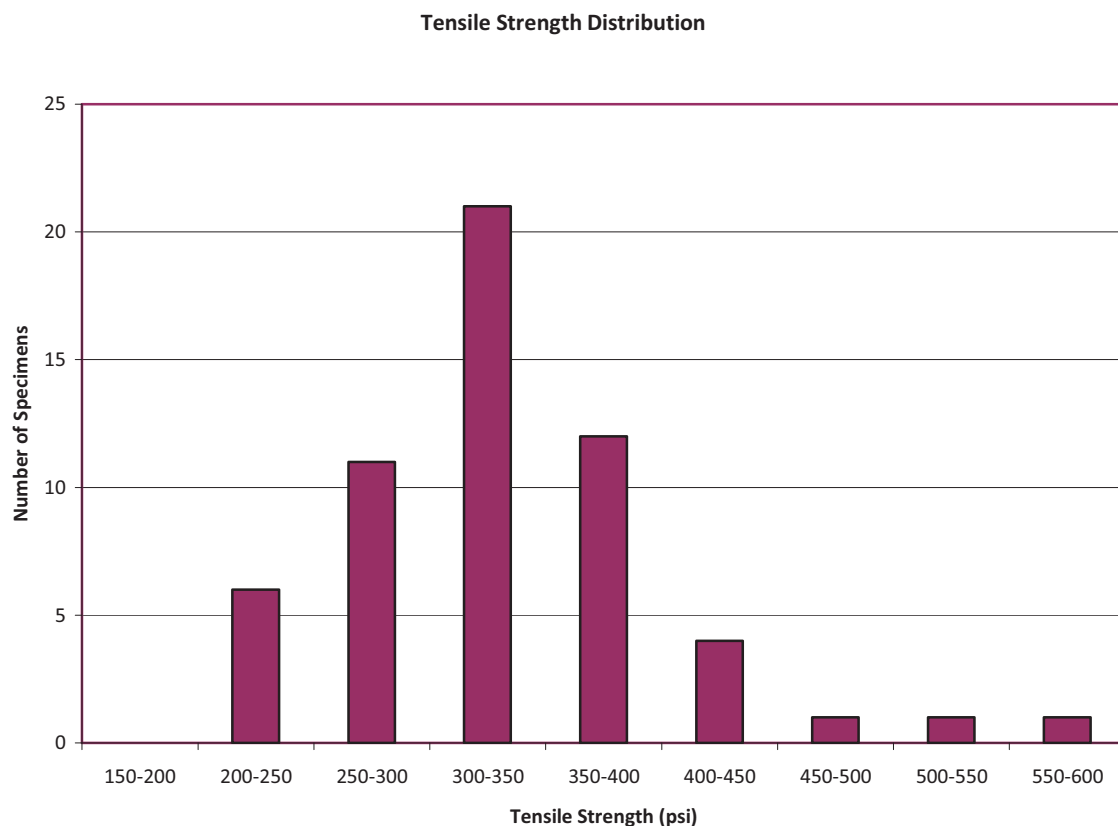
Figure 6—Concrete Tensile Strength Distribution of Bridge Cores

Carbonation Depth

From the results that were gathered and displayed above in Table 3.2, the following plots provide graphical depiction of the carbonation depths versus several different variables such as the geographic regional location in Oregon, time, and a generalized plot of carbonation depth showing the difference between gross and net values.

Gross Versus Net Carbonation Depth:

Gross vs. Net Carbonation Depth
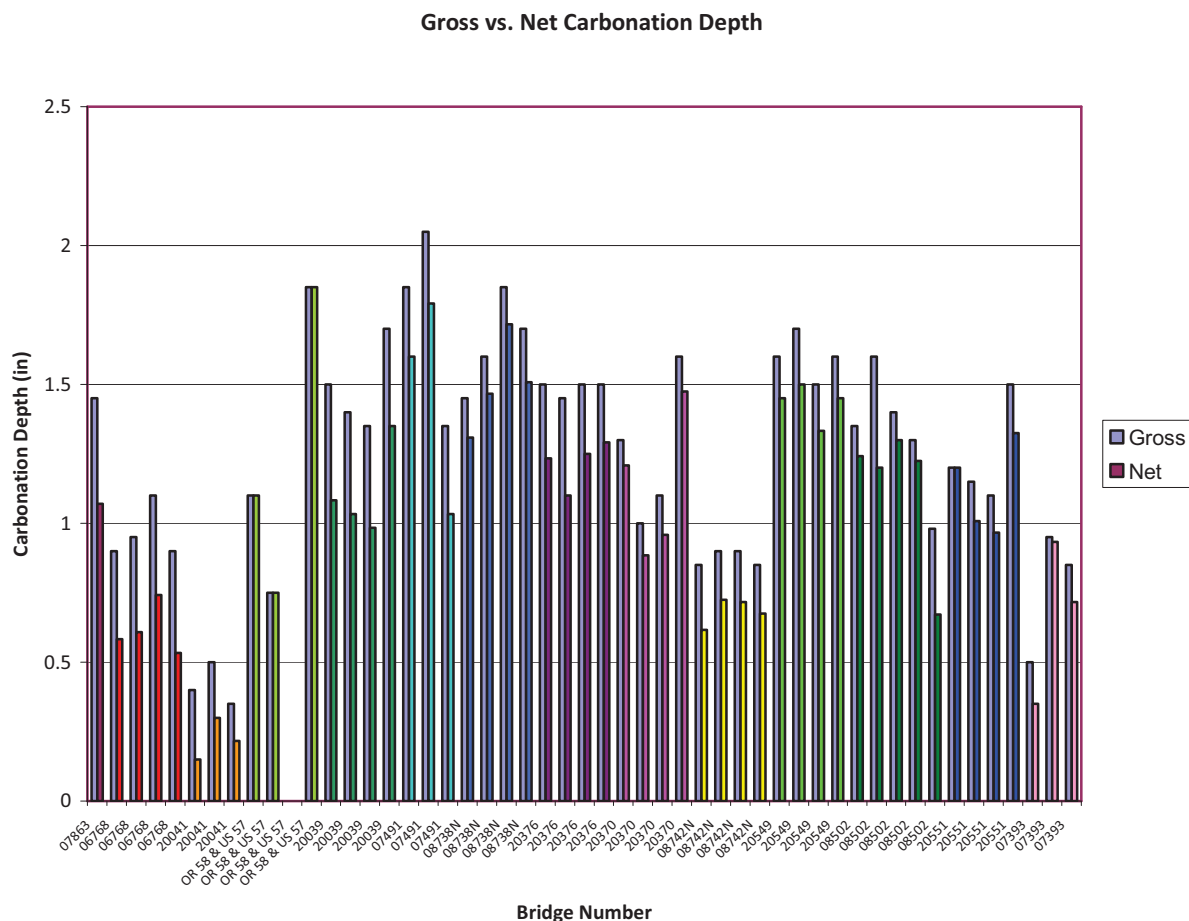
Figure 7—Gross vs. Net Carbonation Depth

The photographs of the carbonation depths of all the bridge specimens are in Appendix A. The Phenolphthalein indicates the non-carbonated portion of the concrete with a purple-magenta coloring.

Strength Correspondence:
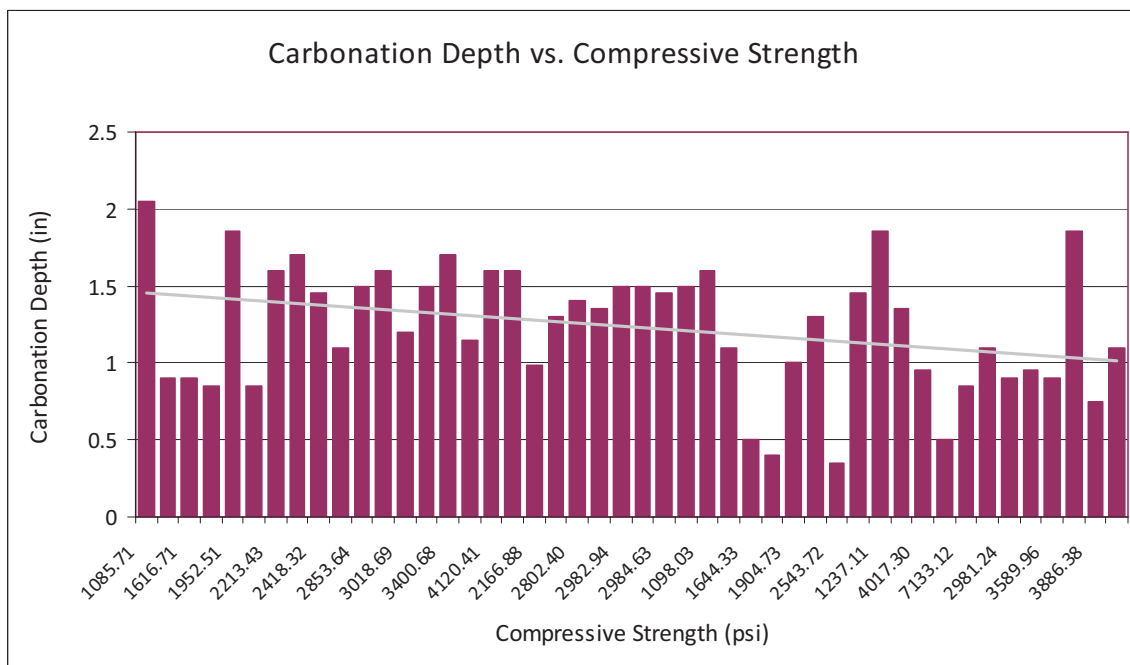
Figure 8—Carbonation Depth vs. Compressive Strength

Regional Correspondence:

Carbonation Depth vs. Geographic Location
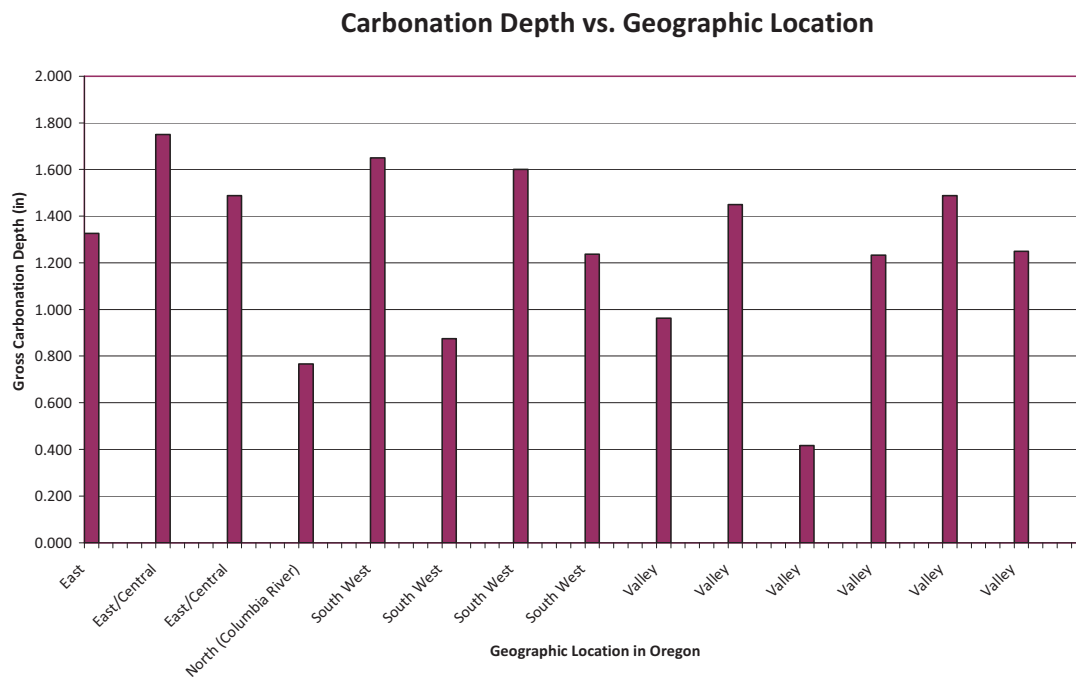
Figure 9—Carbonation Depth vs. Geographic Regional location in Oregon
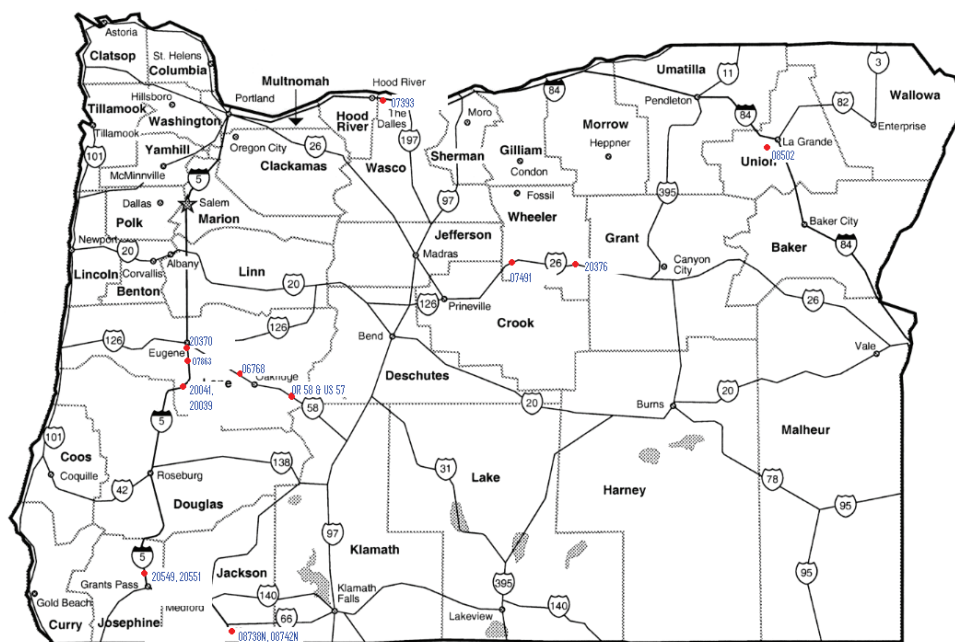
Figure 10—Oregon Map with Bridge Locations

Carbonation of Mid-Twentieth Century Reinforced Concrete Bridges in Oregon

Service Life Analysis (Time History):
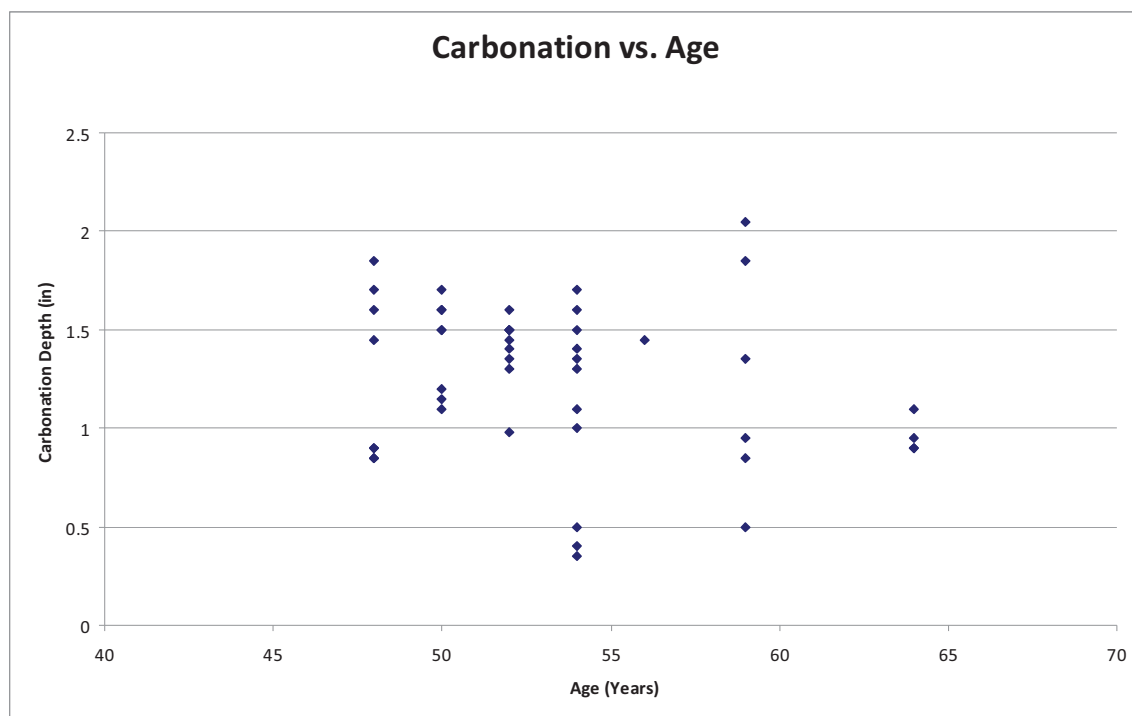
Figure 11—Carbonation Depth vs. Time

3-Dimensional plot of Carbonation vs. Age & Compressive Strength:
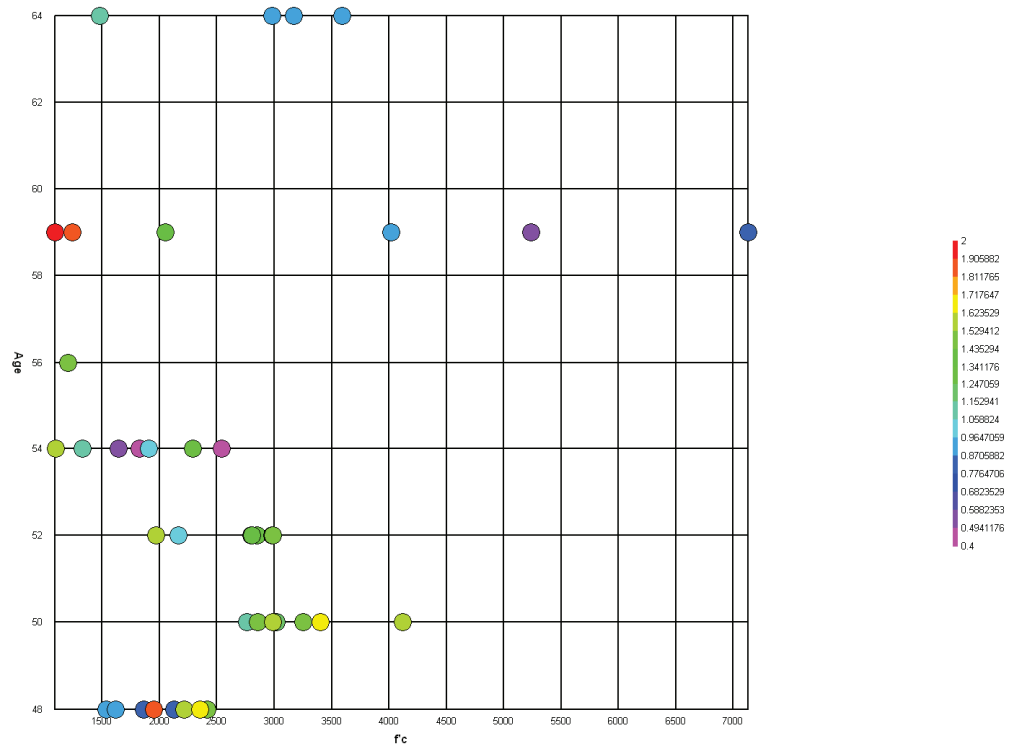
Figure 12—Age vs. Compressive Strength with Carbonation Depth in color scale
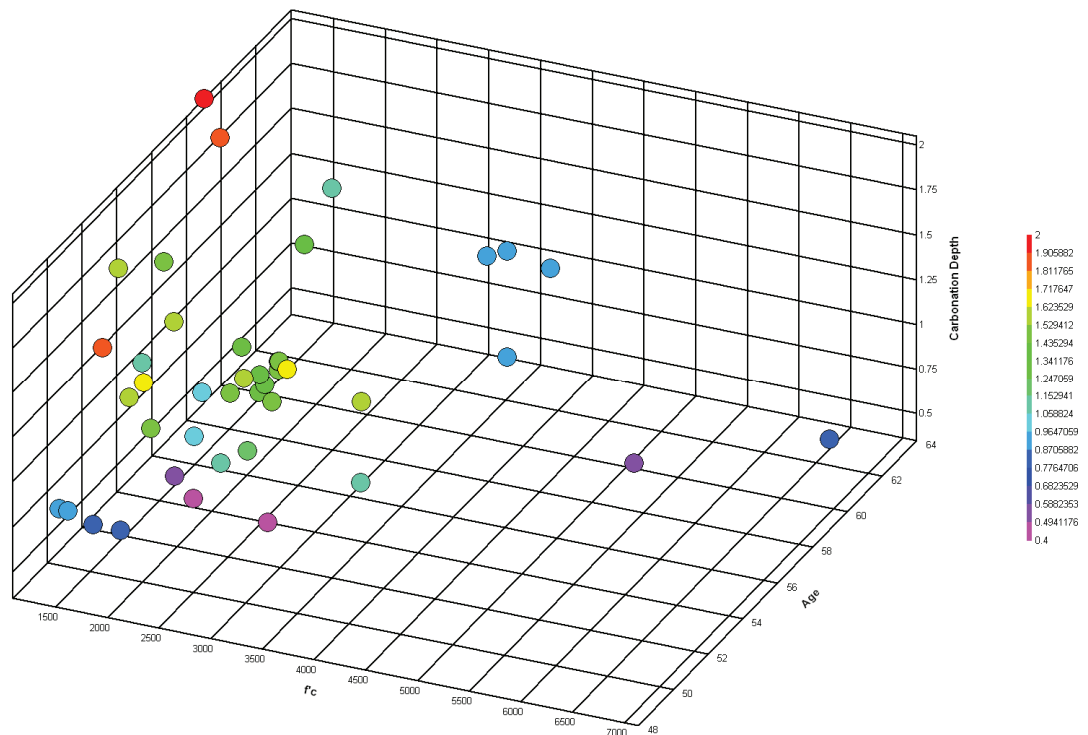
Figure 13—3D view of Age, Compressive Strength, and Carbonation Depth

Carbonation of Mid-Twentieth Century Reinforced Concrete Bridges in Oregon

Discussion

The results obtained for the carbonation depths of the individual cores gives relevant and useful data for a specific bridge; however, when looking at the group of specimens from all the different bridges, it is hard to draw conclusions on the carbonation depths versus geographic location, time, and other factors. The main cause for the inconclusiveness is the variety of parameters that affect carbonation of concrete. The bridges are diverse enough in mixture design, concrete strength, construction date, concrete curing time, geographic and climactic locations, and atmospheric carbonation content.

In order to draw more conclusive results and determine feasible carbonation rates for the individual bridge specimens, the accelerated carbonation results are necessary. Providing consistent and equal parameters for all concrete specimens in the accelerated carbonation chamber limits the differing factors.

Despite the existence of inconsistent group results, each bridge provides relatively good internal consistency for carbonation depth. Each core specimen from its particular bridge correlates exceedingly well with the carbonation rates found in the other specimens. Therefore, it is reasonable to assume that the differing locations on the bridge girders have relatively no significance in the carbonation results. However, it is important to note extremes or inconsistent results among the cores obtained from a single bridge. For example, a cored sample from bridge number 07393 had a deep carbonation spot of 2.7 inches in one location of the specimen. Although the typical

carbonation depth appeared to be 0.95 inches, it's important to note locations where it is vastly different.

Figure 14—Location of Deep Carbonation Depth in core from Bridge No. 07393

If this behavior is typical in not only this sample, but elsewhere on the bridge, due to air voids or cracking, the structural integrity of the bridge structure may be at risk.

Another example of differing test results came from bridge number 08742N. All of the core specimens from this bridge had a very distinct purple area where the concrete was non-carbonated; however, there was also a region that was lighter purple, which might suggest that the specimen was partially carbonated in those regions. On average, just over an inch of depth was seen in the "partially carbonated" zones of these core specimens.

The following figure shows the partially carbonated zone on the concrete between the distinct purple and the gray.
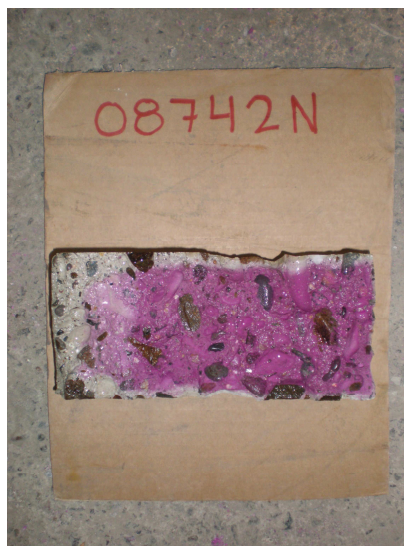
Figure 15—Partially Carbonated Zone in core from Bridge No. 08742N

The partially carbonated zone can be extremely important if it has a pH value low enough to cause depassivation of the reinforcing steel. If that concrete layer is no longer to act as a passivating material for the steel, corrosion may be an issue. Although depassivation doesn't always mean immediate corrosion of the steel, it is a serious problem if not properly dealt with.

The length of total carbonation depth (colorless carbonated layer plus light purple partially carbonated zone) is as much as 2.2 inches on one of the specimens of this bridge. With a concrete cover of only 2 inches for all the bridges, it is clear to see that the steel was subject to depassivation in this region.

Other bridges indicate moderate to severe carbonation depths in the bridge specimens. If the net values for the carbonation depths are used, the in-situ carbonation depth decreases dramatically; however, carbonation does not happen at a linear rate. This means that the carbonation that occurred post-coring on the in-situ face is not equal to the carbonation that occurred on the sides of the core. The rate is

much faster on the sides that have not seen any previous carbonation because carbonation occurs much faster near the surface of the concrete (or breathable layer), whereas carbonation at increasing depth occurs much slower due to the slower diffusion rate of water and carbon dioxide gas.

In order to draw more conclusive results on the climactic impact, a larger sample size is necessary. The bridges considered have too many variables to draw reasonable results based on the location those bridges had been in Oregon. However, it is still useful information to know where that bridge came from, and the environmental factors that were affecting the concrete throughout the life-span of the bridge.

Due to the time variance from bridge construction to bridge demolition, many of the bridges were exposed to atmospheric conditions longer than others. This impacts the results for the rate of carbonation. Although linear results are not clear, the data does somewhat resemble longer exposure periods to be associated with carbonation depth.

Further Testing and Research

Accelerated carbonation tests are the main focus of further testing and research on this project. The same samples from the same bridges will provide results from the accelerated carbonation tests that will be compared to the in-situ carbonation results as well as the group results obtained from all the bridges tested under the same controlled environment. The newly faced half-cylinders are used to have a clean, non-carbonated face at the beginning of the test.

The goal of the accelerated carbonation test is to develop a relatively simple procedure for obtaining the rate of concrete carbonation that can be implemented in the design of reinforced concrete structures. The safety of the structure should be held paramount when designing; therefore, developing a system to minimize hazards caused by environmental degradation will enhance the engineer's ability to design a safe structure.

The accelerated carbonation chamber provides a controlled environment for all the samples to be tested. A picture of the finished Chamber can be seen in Figure 16.

Figure 16—Accelerated Carbonation Chamber

Once results are obtained from the accelerated carbonation test, the plots from in-situ versus those from accelerated carbonation can be compared. It will be interesting to see if the same trends occur among the different specimens.

Several approaches can be taken when doing laboratory testing with accelerated carbonation. Changing parameters and input variables allows results to be obtained for different aspects or objectives. Understanding how these variables affect each other and implementing a test henceforward provides for a successful laboratory experiment.

Conclusion

The results obtained for the bridge core specimens provide significant information on the carbonation depths of the reinforced concrete bridges. With real data from bridge core specimens, it is assumed that the general depth of carbonation for the entire bridge relates to the carbonation depth reported by the core specimens. After further analysis using an accelerated carbonation chamber, the properties of the bridge cores can be more accurately defined. The development of a rate of carbonation for the different bridges can be idealized through the use of the field exposure carbonation results and the accelerated carbonation results for each bridge core specimen. Furthermore, feasibility of an accelerated carbonation chamber can be determined based off the comparison of the field exposure versus controlled laboratory carbonation depths.

Upon completion of this project, the state of the objectives set for this project is as follows:

- Database of service life carbonation depths was developed for a data set of reinforced concrete bridge cylinders.
- An accelerated carbonation chamber was developed to determine approximate carbonation rates and how well accelerated tests correlate to field experience.
- The results can be used to determine plausible actions to be taken to protect existing bridges from environmental deterioration.

Bibliography

Andrade, C. and Alonso, C. (1996). Progress on Design and Residual Life Calculation with Regard to Rebar Corrosion of Reinforced Concrete. In Burke, N., Escalante, E., Nmai, C., Whiting, C. (Eds.). *Techniques to Assess the Corrosion Activity of Steel Reinforced Concrete Structures, ASTM STP 1276* (23-40). West Conshohocken, PA: ASTM.

Andrade, C. (1995). Quantification of Durability of Reinforcing Steel: Methods and calculation procedures. In Aquado, A., Gettu, R., Shah, S. (Eds.). *Concrete Technology: New Trends, Industrial Applications* (157-175). London: E & FN Spon.

ASTM Standard C496, 2004, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens," ASTM International, West Conshohocken, PA, 2004, DOI: 10.1520/C0496_C0496M-04E01.

ASTM Standard C511, 2009, "Standard Specification for Mixing Rooms, Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in the Testing of Hydraulic Cements and Concretes," ASTM International, West Conshohocken, PA, 2009, DOI: 10.1520/C0511-09.

Bob, C. (1990). Some Aspects Concerning Corrosion of Reinforcement. In Dhir, R. & Green, J. (Eds.). *Protection of Concrete* (53-61). London: E & FN Spon.

Branco, F., Brito, J. (2004). *Handbook of Concrete Bridge Management.* United States: American Society of Civil Engineers.

Broomfield, J., Rodriguez, J., Ortega, L., Garcia, A. (1994). Corrosion Rate Measurements in Reinforced Concrete Structures by a Linear Polarization Device. In Weyers, R. (Ed.). *Concrete Bridges in Aggressive Environments* (163-181)*.* Michigan: American Concrete Institute.

Chang, C., Chen, J. (2004). The Experimental Investigation of Concrete Carbonation Depth. *Cement and Concrete Research, 36*(2006) 1760-1767. doi: 10.1016/j.cemconres.2004.07.025

Da Silva, F., Helene, P., Castro-Borges, P., Liborio, J. (2009). Sources of Variations When Comparing Concrete Carbonation Results. *Journal of Materials in Civil Engineering, 21*(7), 333-342. doi: 10.1061/(ASCE)0899-1561(2009)21:7(333)

Liebenberg, A. (1992). *Concrete Bridges: Design and Construction.* New York: Longman Scientific & Technical.

McGrath, P.F., (2005) A Simple Test Chamber for Accelerated Carbonation Testing of Concrete. *Submitted to ConMat'05 3 rd Int Conf on Construction Materials, Vancouver, BC , Aug 2005*. Available from http://www.mcgratheng.com/publications/McGrathPF-carbonation.pdf.

Rostam, S. (1995). Design for Durability: The Great Belt Link. In Aquado, A., Gettu, R., Shah, S. (Eds.). *Concrete Technology: New Trends, Industrial Applications* (127-155). London: E & FN Spon.

Tesfamariam, S., & Martín-Pérez, B. (2008). Bayesian Belief Network to Assess Carbonation-Induced Corrosion in Reinforced Concrete. *Journal of Materials in Civil Engineering, 20*(11), 707-717. doi:10.1061/(ASCE)0899-1561(2008)20:11(707).

Tuuti, K. (1982). Corrosion of Steel in Concrete. *Swedish Cement and Concrete Research Institute*. P. 17-21.

Wight, J., MacGregor, J. (2009). *Reinforced Concrete: Mechanics and Design.* New Jersey: Pearson Prentice Hall.

Appendix A

Photos of Carbonated Core Specimens

Controls:

Bridge Number: 20376

Bridge Number: 06768

Bridge Number: <u>OR 58</u>

Bridge Number: <u>07491</u>

07491

07491

Bridge Number: <u>20041</u>

20041

20041

Bridge Number: <u>20370</u>

20370

20370
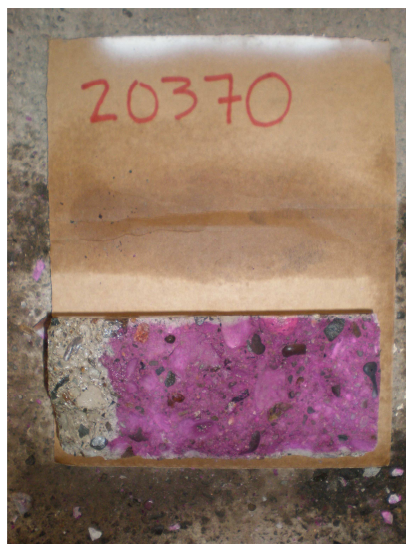
Bridge Number: <u>08742N</u>
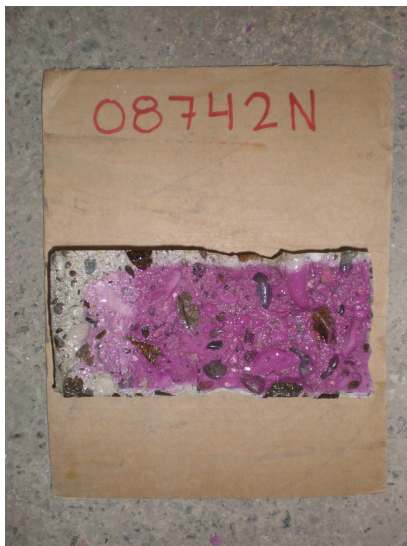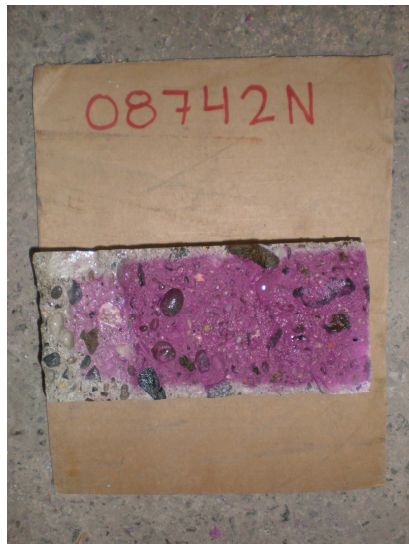
Bridge Number: <u>08738N</u>
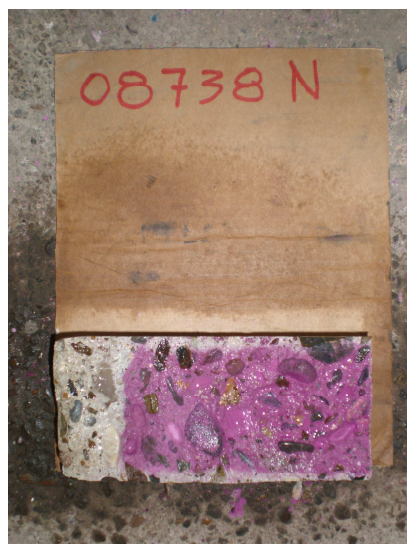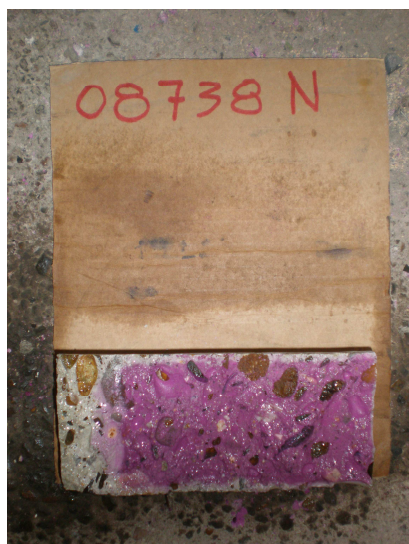
Bridge Number: 07863

Bridge Number: 08502

Bridge Number: 20551

Bridge Number: 20549

Bridge Number: 07393

Bridge Number: 20039

Appendix B

Technical Information for CO_2 Sensor
(For use in Accelerated Carbonation Chamber)

an ISO9001 and 14001 company

FIGARO

Technical Information for the CDM4160 CO$_2$ Module

CDM4160 is a new unit which uses TGS4160, Figaro's durable solid electrolyte CO$_2$ sensor. Due to Figaro's proprietary idea for signal processing with a microcomputer, no maintenance is required for this module. By application of DC voltage to the module, an analog output voltage proportional to CO$_2$ concentration can be obtained. The module can generate a control signal based on a user-selectable threshold concentration. The features mentioned above make this module ideal for air quality control applications in homes, offices and factories as well as in the field of agriculture.

1. Basic Information

1-1 *Features*

* High selectivity to CO_2
* Long life
* Maintenance free
* Pre-calibrated
* Low cost
* Wide detection range
* Excellent durability

1-2 *Applications*

* *Indoor air quality control in offices, homes, and factories*
* *CO_2 monitors*
* *Agricultural applications*

1-3 *Specifications*

The specifications of CDM4160 are contained in Table 1. Depending on the customer's target concentration range, Figaro offers three versions of this module as indicated in Table 1. Customized modules are available according to special requests from customers. Please consult with Figaro.

1-4 *Dimensions (see Fig. 1 below)*

u/m: mm

Fig. 1 - CDM4160 dimensions

2. Structure and Functions

2-1 *Solid electrolyte CO_2 sensor TGS4160*

The sensor changes its output voltage in response to exposure to CO_2 gas. The sensor should not be directly connected with low-input impedance equipment. To read sensor output, amplified voltage (CP4) should be used. Please refer to *"TGS4160 Technical Information"* for details as to the characteristics of the sensor.

2-2 *Sensor's output signal voltage (CP4)*

The sensor's output can be read as an amplified voltage (as taken from the differential amplifier). Voltage should be read from this pin.

2-3 *Microprocessor*

The microcomputer takes data and renews it once per second. The processor calculates CO_2 concentrations based on the difference between current sensor output and a baseline value (which represents the sensor output value in fresh air).

2-4 *Thermistor signal output voltage (CP3)*

The thermistor's output signal voltage, which is used for compensation of the sensor's temperature dependency, can be read at this pin.

2-5 *Concentration setting for control signal (JP1, JP2)*

Concentrations utilized for control signals (i.e. the *control concentration*) can be changed by modifying the settings of JP1 and JP2 as shown in Table 3. The module is pre-set at Mode I before shipment.

2-6 *Baseline reset switch (SW1)*

The baseline value may be manually reset using this switch. When this switch is pushed, the sensor's output at that moment is memorized as 400ppm of CO_2 (the baseline value in fresh air). When the switch

Pin No.	Name	Description
1	V_{IN}	Power supply input
2	V_{CONC}	CO_2 concentration output
3	CTRL	Control signal output
4	TRBL	Trouble signal output
5	GND	Common ground

Table 2 - CDM4160 pin designations of CN1
NOTE: CN1 should be MB5P-90S, mfg. by JST.
Recommended receptacle for connector: 05JQ-BT, mfg. by JST.

Product name	Carbon dioxide (CO_2) sensor module		
Model No.	CDM4160-L00	CDM4160-M00	CDM4160-H00
Detection range	400 to 4,500ppm	400 to 9,000ppm	400 to 45,000ppm
Sensor (principle)	TGS4160 (Solid state electrolyte)		
Accuracy (*1)	Approx. ±20%		
Power supply	DC5.0±0.2V regulated		
Power consumption	1.3W (typical)		
Inrush current	600mA (max)		
Operational temperature & humidity range	-10˚~+50˚C, 5~95%RH (avoid condensation)		
Storage temperature & humidity range	-20˚~+60˚C, 5~90%RH (pack in a moisture proof bag)		
Warm up time	2 hours		
CO2 concentration signal (*2)	Continuous analog output proportional to CO2 concentration		
	Vconc = CO2 concentration/1,000 (DC 4.5V full scale)	Vconc = CO2 concentration/2,000 (DC 4.5V full scale)	Vconc = CO2 concentration/10,000 (DC 4.5V full scale)
Control signal	ON: HIGH output (when CO2 conc. exceeds threshold) OFF: LOW output		
	800/ 1,000/ 1,500/ 2,000 (ppm)	1,000/ 2,000/ 5,000/ 8,000 (ppm)	5,000/ 10,000/ 20,000/ 40,000 (ppm)
Trouble signal	ON: LOW output (sensor trouble) OFF: FLOAT NC		
LED display	Green LED: Lights while power is on (blinks during warm up)		
	Yellow LED: Blinks during trouble		
	Red LED: Lights when CO_2 concentration exceeds the threshold		
Reset switch	Establishes the ambient CO_2 concentration as 400ppm when pushed		
Dimension	45 x 60 x 31mm (45 x 67 x 31mm incl. CN1)		
Weight	approx. 22g		

Note 1: Assumes benchmark is set accurately at 400ppm of CO_2. This value does not contain long term drift.

Note 2: In this module, the CO_2 concentration is calculated by measuring the relative change of sensor output at the measuring point from sensor output in clean air (assumed to be 400ppm of

Table 1 - Specifications
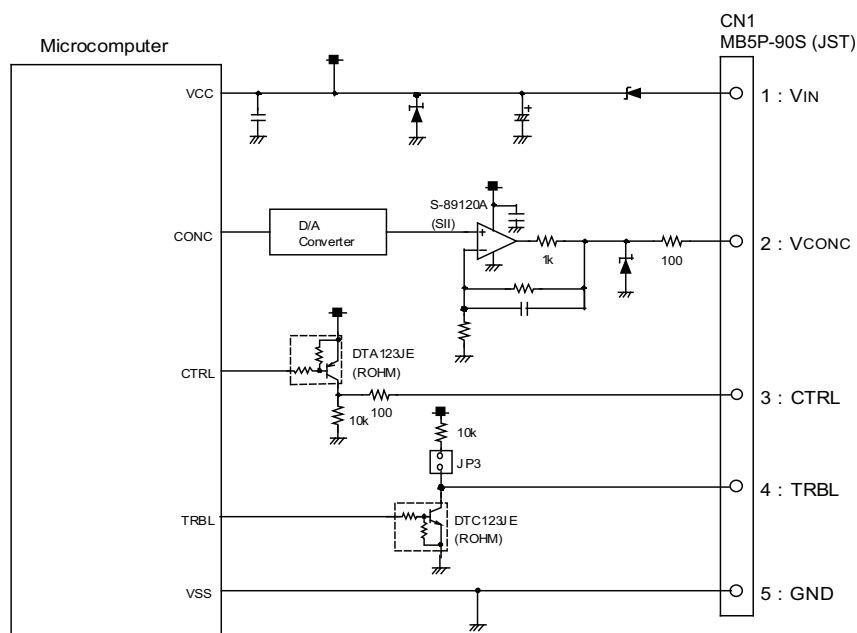
Fig. 2 - Circuit diagram of interface for circuit

is pushed during warm-up time (within the first two hours after the module is powered on), the baseline voltage is read at that moment and the module immediately goes into operation mode.

Caution: If the baseline reset switch is pushed while in a polluted environment where the actual CO_2 concentration is higher than 400 ppm (ambient levels), the accuracy of readings may become adversely affected.

2-7 *Input-output signal (CN1)*

Please refer to *Table 4-Rated input and output voltages* for rated values. Also please refer to *Fig. 2-Circuit diagram of interface for circuit.*

2-7-1 *Vin (Pin No. 1)*
Regulated voltage should be input into this port. The sensor's output may vary according to the sensor's voltage dependency characteristics if the input voltage fluctuates. Please refer to *"TGS4160 Technical Information"* for details.

2-7-2 *Concentration output (Pin No. 2)*
An analog voltage corresponding to CO_2 concentration is output from this port according to the following equation:

CDM4160-L00 : Vconc=CO_2 conc/1000
CDM4160-M00 : Vconc=CO_2 conc/2000
CDM4160-H00 : Vconc=CO_2 conc/10000

2-7-3 *Control signal output (Pin No. 3)*
When CO_2 concentrations exceed the control concentration, output from this port will be "ON".

Mode	JP1	JP2	Threshold CO_2 Concentration of Control Signal					
			CDM4160-L00		CDM4160-M00		CDM4160-H00	
			ON	OFF	ON	OFF	ON	OFF
I	OPEN	OPEN	800ppm	720ppm	1,000ppm	900ppm	5,000ppm	4,500ppm
II	OPEN	SHORT	1,000ppm	900ppm	2,000ppm	1,800ppm	10,000ppm	9,000ppm
II	SHORT	OPEN	1,500ppm	1,350ppm	5,000ppm	4,500ppm	20,000ppm	20,000ppm
IV	SHORT	SHORT	2,000ppm	1,800ppm	8,000ppm	7,200ppm	40,000ppm	36,000ppm

Table 3 - CDM4160 jumper pin settings

2-7-4 *Trouble signal output (Pin No. 4)*
This port will output "ON" in the following situations:
* when sensor output is abnormal
* when the connection for control concentration setting is broken

2-8 *LEDs (refer to Table 5)*

2-8-1 *Green LED (LED 1)*
The green LED indicates the module's power condition. It will be lit when the power is on. The LED blinks on and off during warm-up time (for the first two hours after power is turned on).

2-8-2 *Yellow LED (LED 2)*
The yellow LED indicates a trouble condition. It blinks on and off when trouble signal output is "ON".

2-8-3 *Red LED (LED 3)*
The red LED indicates that CO_2 concentration is over the control concentration level. It lights when control signal output is "ON".

Pin No.	Item		Minimum	Typical	Maximum	u/m
1	Power supply input (V_{IN})		3.5	5.0	5.5	V
2	CO_2 conc. output (V_{CONC})	Output voltage	GND	-	4.6	V
3	Control signal output (CTRL)	Output voltage (trouble) I_{OL}=1mA	GND	-	0.2	V
		Output voltage (warmup) I_{OH}=1mA	V_{IN}-0.6	-	V_{IN}	V
		Allowable current	-	-	25	mA
4	Trouble signal output (TRBL)	Output voltage I_C=5mA	-	-	0.3	V
		Allowable voltage	-	-	50	V
		Allowable current	-	-	100	mA

Table 4 - Rated input/output voltages

Condition	Green LED	Yellow LED	Red LED	Control signal (CTRL)	Trouble signal (TRBL)	CO_2 concentration signal		
						CDM4160-L00	CDM4160-M00	CDM4160-H00
Warm up period	Blink ON/OFF	OFF	OFF	OFF	OFF	0.4V	0.2V	0.04V
CO_2 Conc. < Threshold	ON	OFF	OFF	OFF	OFF	CO2 conc./1000V	CO2 conc./2000V	CO2 conc./10000V
CO_2 Conc. ≥ Threshold	ON	OFF	ON	ON	OFF	CO2 conc./1000V	CO2 conc./2000V	CO2 conc./10000V
Trouble	ON	Blink ON/OFF	OFF	OFF	ON	4.5V	4.5V	4.5V

Table 5 - Signal output in operation mode

3. Operation modes

3-1 *Warm up*

The sensor is warmed up for two hours after the module is powered on. The green LED blinks on and off and a constant voltage (0.4V for -L00, 0.2V for -M00, 0.04V for -H00) is output from the concentration output port during this period. The green LED will be lit continuously after the warm-up period ends unless a power outage occurs.

3-2 *CO_2 concentration lower than threshold level*

The green LED will be on if the calculated CO_2 concentration is lower than the threshold level.

3-3 *CO_2 concentration exceeds threshold level*

The red LED will be lit and the control signal output is turned "ON" if the concentration exceeds the threshold level.
The control signal output is turned to "OFF" and the red LED will be off when CO_2 concentration drops to 90% of the threshold level.

3-4 *Trouble*

When the sensor's output is abnormal, the yellow LED blinks on and off and the TRBL signal is set to "ON.

4. Cautions

1) By assuming that the baseline level represents fresh air (400ppm of CO_2), actual CO_2 concentrations are calculated based on the difference between the baseline level and the current sensor output. As a result, the following cautions should be noted:

a) Accurate readings cannot be expected if an accurate baseline could not be acquired.

b) The sensor should be exposed to fresh air periodically to properly renew the baseline level. Performance shown in the specifications cannot be achieved if the module was used in an environment where CO_2 concentrations increased slowly and steadily for a long period of time.

c) The module should be located in fresh air during the warm-up period. Accurate readings cannot be expected until the baseline is acquired in fresh air. If the module is warmed up in an environment where CO_2 concentration is higher than normal fresh air, the baseline will represent a polluted level and the device will not be able to clean the air sufficiently.

d) Power should be on at all times. Since the baseline is memorized in a microcomputer, if the power should be cut off, the memory would be lost and operation would resume from the warm-up process.

e) The module is not intended for usage in life saving equipment. If the module is incorporated into life saving equipment, an alternative and secure measure for calculating CO_2 concentration should be used be used for the life safety function.

2) This module is designed only for indoor usage. The module should be protected from exposure to rain, wind, sun, heat radiation, etc.

3) Please apply a regulated voltage, otherwise the accurate reading cannot be expected. Application of excessive and/or reverse voltage would cause damage to the module.

4) The module does not include a circuit for protection from excessive current. An excessive current protection circuit should be added to a peripheral circuit of the module.

5) The sensor may deteriorate if it is stored without power in a high humidity environment for a long period of time. Please keep the sensor in a humidity-proof bag with a desiccant if the sensor is to be stored without power for a long period of time.

6) The sensor has dependency on oxygen concentration. Under environments where the oxygen concentration

varies from ambient conditions (i.e. 21% O2), the sensor may not function properly and may not exhibit the chracteristics described in this brochure.

7) Please refer to *"TGS4160 Technical Information"* for other handling precautions of TGS4160.

5. Important Notice

Figaro Engineering Inc. (Figaro) reserves the right to make changes without notice to any products herein to improve reliability, functioning or design.

Information contained in this document is believed to be reliable. However, Figaro does not assume any liability arising out of the application or use of any product or circuit described herein; neither does it convey any license under its patent rights, nor the rights of others.

IMPORTANT NOTE
This product is not designed and authorized for use as a component in life support applications wherein a failure or malfunction of the products may result in injury or threat to life. Figaro Engineering Inc. reserves the right to make changes to this product without notice to improve reliability, functioning, and/or design.

FIGARO GROUP

HEAD OFFICE

Figaro Engineering Inc.
1-5-11 Senba-nishi
Mino, Osaka   562-8505   JAPAN
Tel.: (81) 72-728-2561
Fax: (81) 72-728-0467
email: figaro@figaro.co.jp
www.figaro.co.jp

OVERSEAS

Figaro USA Inc.
121 S. Wilke Rd. Suite 300
Arlington Heights, IL 60005 USA
Tel.: (1) 847-832-1701
Fax.: (1) 847-832-1705
email: figarousa@figarosensor.com
www.figarosensor.com

Appendix C

Bridge Plans

GENERAL NOTES

All concrete shall be class "A" mix and shall have a breaking strength of 3300 lbs. per sq. in. in 28 days. ($f_c = 1100^\#/sq.$ in.)

All reinforcing steel shall be intermediate grade deformed bars placed 2" clear of the nearest face of concrete unless noted or shown otherwise.

Bridge designed for H-20 S-16 loading.

Footing elevations may be varied at the discretion of the Engineer to secure suitable foundation.

All materials and workmanship shall conform to the specifications for bridges of the Oregon State Highway Commission.

Elevations shown are top of finished deck. 1.11 Lift of P.E. and are 1.63 above profile grade.

Approved:

Bridge Engineer

Chief Engineer

Grades Revised Feb. 2, '47
Revised Feb. 20, 1947
Sidewalk replaced by curb.

ELEVATION
Scale: 1/8" = 1'-0"

Sta. 564+18 El. 628.49
Bent #1 Sta. 564+34 El. 628.50
Bent #2 Sta. 564+84 El. 628.55
Bent #3 Sta. 565+42 El. 628.61
Bent #4 Sta. 565+92 El. 628.66
Sta. 566+08 El. 628.68

El. 608.0

Solid Rock

Hinge
Proposed Channel Change
Existing Ground
High Water El. 622.7 (W.E. 5-6-40 616.7)
Proposed Channel Change
Hinge
Grade + 0.18%

PLAN
Scale: 1/8" = 1'-0"

NOT TO SCALE
35'-0"
2'-6" 30'-0" Roadway 2'-6"
PT 564+11.99
16'-0" 50'-0" 58'-0" 50'-0" 16'-0"
190'-0"

Install Standard Curb Marker
Bm. D
Bm. A
Bm. B
Bm. E
Drain
Handrail Details shown on Drwg. No. 8417
Bm. C
Bm. F
€ Roadway
Install Standard Curb Marker

To Goshen 10.7 mi.
To Lowell 3.75 mi.

PLAN

39'-3" o-o
4'-7½" 30'-0" Clear Rdwy. 4'-7½"
1'-6" at ₵ Bents 27'-0" c-c Outside Bms. 1'-6" at ₵ Bents
1'-0" 3'-6" 9'-0" 4'-6" 4'-6" 9'-0" 3'-6" 1'-0"
Beam spacing

Bm C
Bm D
Bm A
Bm B
Bm D
Bm E

25'-0"

Elev. 101.10
Elev. 103.38
Sta. 1521+09
Bm A 161+43"
Sta. 1521+65
Elev. 103.50
Elev. 101.10
Bm B 161+43"

15'-0" ₵ Drain (See Detail)

Sta. 1522+35
Elev. 103.50
Sta. 1522+91
Elev. 103.38
Elev. 101.10
Bm A 161+43"
Bm C

2'-6" 2'-0" 2'-0" 8'-0" 8'-0" 2'-0" 2'-0" 2'-6"

17'-0" ₵ Drain 15'-0" ₵ Drain

6 equal spaces 6 equal spaces 6 equal spaces
6" 6" 6"

Inside rail - 69'-3¼"

Outside rail - 56'-6⅜" Inside rail - 55'-5¼" Outside rail - 20'-8¼"

See drawing No. 9233 for handrail details.

Scoring 4" deep thru side walk ₵ curb & completely thru rail base

Test hole No.1   Test hole No.2

18'-10"
182'-0" c-c End bents on ₵ Rdwy.

2'-0"

56'-0"
56'-9" 55'-10"

On bridge ₵
On bridge ₵

To The Dalles
17 miles

ELEVATION

Scale: ⅛"=1'0"

Outside rail - 56'-6⅜" Inside rail - 55'-5¼" Outside rail - 20'-8¼"

5 equal spaces 6 equal spaces 5 equal spaces
6" 6" 6"

Extreme high water, Elev. 91.8 (1948)?
Low water, Elev. 22.37

Original ground surface

Hydraulic fill
Treated piling (Bearing 21 tons/pile)
Hydraulic fill

Steel H Bearing Pile

0.0%   0.0%

El. 49.9

Elev. 47
Elev. 64
Test hole No.1
El. 21.0 Av. El.

Elev. 47
Elev. 64
Test hole No.2
El. 21.0 Av. El.

Av. El. 6.0
El. 94.3

Hardpan
Soft silty clay
Untreated piling approx. 20 ft long (Bearing 25 tons/pile)
Sand and gravel
Sand and gravel

LINE DIAGRAM
Not to Scale

P.S. Sta. 1509+25.02
P.S.C. Sta. 1515+25.02
P.C.S. Sta. 1528+87.24
P.T. Sta. 1534+87.24

3° C.L.
Ta = 59'.01"
Ts = 1395'.11"

600'.59'
s=9°
3°.06

600'.59'
s=9°
3°.06

Approved:
Bridge Engineer

△ 8-18-71 As Constructed

GENERAL NOTES:

Bridge designed for H20-S16-44 loading.
Bearing piling shall be class "A" and shall attain a breaking strength of 3300 p.s.i. in 28 days. (fc = 1320 p.s.i.)
All reinforcing shall be intermediate grade deformed bars. All reinforcing bars shall be placed 2" clear of any surface of concrete unless otherwise noted. All spaces shall be 1⅓" unless otherwise noted.
Elevations shown are finished grade at gutters. Finished grade is 1.8" above subgrade.
Tons per pile for piling is 25 tons per pile.

Subgrade = 100.02
101.14 ₵

BRIDGE No. 7490
ELEVATION
ACCOMP. BY DWGS No. 10127, 10132 & 583
PLAN

BRIDGE No. 7491
ELEVATION
ACCOMP. BY DWGS No. 10127, 10133 & 583
PLAN

BRIDGE No. 7492
ELEVATION
ACCOMP. BY DWGS No. 10127, 10130
PLAN

GRADE LINE DIAGRAM
No Scale

Note: Elevations of grade line diagram are to sub-grade.
right of %. Finished grade 1/29' above sub-grade.

Note: Elevations are given to finished grade at gutter.

Note: Res. Engr. to check Grade Elevations before construction.

GENERAL NOTES:
All material and workmanship shall conform to the specifications of the Oregon State Highway Commission.
All concrete shall be class "A" unless otherwise noted.
Reinforcing steel shall be intermediate grade deformed bars conforming to A.S.T.M. specifications A305-49.
Allowable foundation bearing taken as 3 tons/☐.

Revised Dec. 27, 1951

Approved:
Bridge Engineer

ELEVATION VIEW

Scale = 8" = 1'-0"

12"×53" H-Piles
Ld/Pile = 24 Tons

Sta. 747+50.08
Bent 1

8" Cast iron pipe - connect to installed Catch Basin

Clay & Boulders

65'-0"

El. 648.00
Ld/Pile = 42 Tons
12"×53" H-Piles

Clay & Boulders

226'-0" ± to ℄ of end beams measured along Rdwy. ℄

Dimensioned along Rdwy. ℄

83'-0"

carry to Catch Basin

Profile @ 14'-0" Rt.

El. 648.00
Ld/Pile = 42 Tons
12"×53" H-Piles

Sta. 749+00
Test Hole
5'-15'-0" Lt. ℄
4'-15'-0" Rt. ℄

Hard Decomposed Rock
El. 630.00
El. 635.00
El. 670.00 Clay
Grey Clay
Clay

1.5 to 1 Slope Normal

65'-0"

El. 664.00
@ 84.5 Tons
Ld/Pile = 42 Tons

13'-0"

Hinges

12"×53" H-Piles

Fill

Sta. 749+76.08

Grade + 94.9%

Approved:
Bridge Engineer

State Highway Engineer

7-79 Deck Production: Dense low Slump P.C.C. resurface. (Ion. System/membrane placed Jul '78.)

Revision 06/11/84

General Notes:

Bridge designed for 2 lanes H20-S16-44 loading except longitudinal beams, which are designed for 3.0 lanes with load equally distributed to all longitudinal beams. All reinforcing steel shall be intermediate grade deformed bars, Bars from #3 to #11 inclusive shall conform to A.S.T.M. spec's. A305-50T and shall be lapped 20 dia unless otherwise marked. #14 bars shall be lapped 60 dia at #11 spaces unless otherwise noted. All concrete shall be class "A" and shall attain a breaking strength of 3,300 p.s.i. in 28 days (fc=1320 p.s.i.)

PLAN VIEW

34'-10" o. to o.
30'-0" Clear Roadway
26'-0" to top of slope
1'-6"   1'-6"
1.5 to 1 slope to drain
℄ of Sta 5-73

4 Beams @ 8'-10" ctrs.
Catch Basin - Dwg. 5-195

Bm. "E" Cap
90°-0"
Bm. "A" 14½"×48"
Bm. "F" 8"×46"
Bm. "A" 14½"×48"
Bm. "A" 14½"×48"
Bm. "F" 8"×46"
Bm. "F" 8"×46"
Bm. "G"

Sta. 748+15.08
Bent 2

Curve Data
1°C.R.
Ta = 21'-12"
T5 = 272.47
L = 2520.00
All Bents lie
on Radial lines.

Clear pt. "A": Min clearance at pt. "A".
℄ line bent of Bm. El. 676.27
D-line Gutter El. 15'-0" Lt 655.13
Vert. Cl. = 21'-4½"

"L"-line - Sta. 748+56.58 P.O.T.
"O"-line - Sta. 4+72.93 P.O.T.

Bm. "B" 14½"×48"
Bm. "F" 8"×46"
37°52'2760"
Bm. "B" 14½"×48"
Bm. "F" 8"×46"
Bm. "B" 14½"×48"
Bm. "B" 14½"×48"
Bm. "F" 8"×46"
Bm. "G"

30'-0" Clear Roadway

Sta. 748+98.08
Bent 3

Bm. "G"

Bm. "C" 14½"×48"
Bm. "F" 8"×46"
Bm. "C" 14½"×48"
Bm. "C" 14½"×48"
Bm. "F" 8"×46"
Bm. "C" 14½"×48"
Bm. "F" 8"×46"

Catch basin not to be installed by Bridge Contractor

Sta. 749+63.08
Bent 4

Bm. "D" 14½"×48"
Bm. "F" 8"×46"
Bm. "D" 14½"×48"
Bm. "D" 14½"×48"

Existing 1" water main

Note: Elevations shown at top of concrete 1½" below finish grade.

26'-0" ± to top of slope

Bm. "K" 12"×50"

Sta. 749+76.08

Top of slope

PLAN VIEW
Scale 1"=15'-0"

ELEVATION VIEW
Scale 1"=15'-0"

"L" LINE SECTION
Scale: 1"=10'-0"

SUPER ELEVATION DIAGRAM
No Scale

"C" GRADE LINE
No Scale

"L" GRADE LINE
No Scale

General Notes:
Structure designed for H20-S16-44 loading.
All concrete shall be Class "A" and shall attain a breaking
strength of 3300 psi in 28 days (f'c 1320 psi).
All reinforcing steel shall be intermediate grade deformed
bars and shall conform to A.S.T.M. Specification A 305. Bars
from #3 to #11 inclusive shall be lapped 20 diameters at all
splices unless noted or shown otherwise. Bars #14 & #18 shall
not be spliced except as shown. All bars shall be placed 2"
clear of nearest face of concrete unless noted or shown
otherwise. f's 20,000 psi.
Footing elevations are subject to change depending
on foundation material encountered. Reinforcing steel
for columns shall not be fabricated until final footing
elevations have been determined in the field.
All workmanship and materials shall conform to specifications
for bridges of the Oregon State Highway Commission.

Note:
Structure elevation shown 17'-4" Rt. of "L" Line &.
Finish "L" Line profile shown @ "C" Line &.

Span 1   Span 2   Span 3   Span 4

34'-8" o-o
30'-0" Rdwy.

326'-6"

Reinforced Concrete
Retaining Wall
See Dwg. 14824 & Dwg. 14824-A

GRADE LINE DIAGRAM
OAK STREET

Elevations shown are profile grade 10' Left or Right of ₵ and are finish grade.

B.V.C. Sta."O" 20+00 El.1787.52

P.I. Sta."O" 21+00 El.1776.41

200' V.C.

E.V.C. Sta."O" 22+00 El.1769.11

-7.30 %

ELEVATION—NORTH BOUND STRUCTURE
Scale 1"=20'-0"

Sta."LN"1407+63.00 Bent 1N
Sta."LN"1407+90.00 Bent 2N
Sta."LN"1408+46.00 Bent 3N
Sta."LN"1408+80.64 P.O.C. = Sta."O"20+97.39 P.O.T. Bent 4N
Sta."LN"1409+16.00
Sta."LN"1409+72.00 Bent 5N

NOTE
Deck elevations shown are top of concrete at gutter line

ELEVATION—SOUTH BOUND STRUCTURE
Scale 1"=20'-0"

Std. Type "A" Catch Basin, carry 8" corr. metal drain pipe to toe of slope. See Dwg. #14660.

PLAN
Scale 1"=20'-0"

Sta."LS"1406+28.00 Bent 1S
Sta."LS"1406+84.00 Bent 2S
Sta."LS"1407+40.00 Bent 3S
Sta."LS"1407+74.94 P.O.C. = Sta."O"22+36.66 P.O.T.
Sta."LS"1408+10.00 Bent 4S
Sta."LS"1408+66.00 Bent 5S
Sta."LS"1408+93.00 Bent 6S

NOTE
All Bents placed parallel to Oak Street ₵

TEST HOLE DATA

Test Hole #5 2"/1406+45.0 50' Rt.
Test Hole #4 2"/1407+60.0 45' Rt.
Test Hole #2 2"/1407+66.0 50' Lt.
Test Hole #1 2"/1408+65.0 54' Rt.
Test Hole #3 2"/1409+10.0 60' Lt.

VICINITY MAP

Sta."LS"1406+28.00 Begin structure
Sta."LN"1407+63.00 Begin structure
Sta."LS"1408+93.00 End structure
Sta."LN"1409+72.00 End structure

To Ashland
Pacific Highway (North Bound)
Pacific Highway (South Bound)
Proposed Structures
Oak Street (Relocation)

GRADE LINE DIAGRAM

Sta."LS"1406+90.63 Elev.1793.12
Sta."LN"1400+90.63 Elev.1793.60

Fin. Grade 12'/Ft. (South Bound) (+0.3379%)
Fin. Grade 12'/Ft. (North Bound) (+0.3031%)

MILITARY LOADING

24K axle
24K axle
4'-0"

Lane Load Width
2'-0" Min.
6'-0"
2'-0" Min.

DESIGN DATA 56-70-56

	A	B	C	D	E
M	90	+900	-1247	-1250	+550
V		125	124		

M = ft kip/bm   V = kip/bm

REVISION

8/2/61 — Dwg #1816 added ₵ no. of sheets changed.
8/2/61 — Dwg #1812G added 8"x 36'-9" pipes added.
8"x 36'-9" pipes added
Bent 3N Cols.
Bent 5S Cols.
Bent 5S Ftgs. elev.

Revised as constructed 3.27.64

GENERAL NOTES

Bridges designed for H20-S16-44 and Military loading. Longitudinal beam loads are assumed to be distributed in accordance with Article I-31 (b) Case I of the specifications for bridges.

All concrete shall be class "A" and shall have a breaking strength of 3300 p.s.i. in 28 days (f'c=1320 p.s.i.).

All reinforcing steel shall be intermediate grade deformed bars. Bars #3 to #11 inclusive shall conform to A.S.T.M. specification A-305 and shall be lapped 20 diameters at all splices unless noted or shown otherwise. Bars #14 shall conform to A.S.T.M. specification A-408 and shall be lapped 30 diameters at all splices unless noted or shown otherwise. All reinforcing bars shall be placed 2" clear of the nearest face of concrete unless noted or shown otherwise. (f's = 20,000 p.s.i.)

All materials and workmanship shall conform to the specifications for bridges of the Oregon State Highway Commission.

MILITARY LOADING

24K axle
4'-0"
24K axle

2'-0" Min.
Lane Load Width
6'-0"
2'-0" Min.

ELEVATION N.B. LANE
Scale: 1"=20'

Fill
2'-0" thick rip rap El. 1828.0
High Water Elev. 1824.5 Dec. '55
Rip rap placed 3'-0" above H.W. mark
El. 1828.0
Ground Line & 15' Lt.
Ground Line & 15' Rt.

Elev. 1828.7 Lt. 1813.7 Ch. 1813.7 Rt. 1812.8
Elev. 1826.2 Lt. 1810.5 Ch. 1809.7 Rt. 1809.3
Bent 1 N
Elev. "L" 1530+79.0
Elev. "L" 1531+19.0 Bent 2 N
"L" 1531+750 Bent 3 N
"L" 1532+450 Bent 4 N
"L" 1533+01.0 Bent 5 N
"L" 1533+46.0 Bent 6 N
Sta. "L" 1530+83.35 P.O.C. Ch. Ch. 10+72.98
"L" 1532+09.80 P.O.C. Ch. Ch. 9+28.22
Sta. "3" = 3+00 Elev. 1825.0 -0.8%
30'-0" Rdwy. Width
15'0" 15'0"

BEAR CREEK CHANNEL CHANGE PROFILE
No Scale

PLAN N.B. & S.B. LANES
Scale: 1"=20'

"1" Line 50'-0" 50'-0" "1" Line
Test hole #1
40'-0" Span 1
56'-0" Span 2
70'-0" Span 3
56'-0" Span 4
45'-0" Span 5
Note: Deck elevations shown are top of concrete at gutter line see Dwg #4440 for details
& Channel Change 62.0 Wide
Std. Drain Type "N" see Dwg #4440 for details
8" corr. pipe
267'-0" R.C.D.G. c/c-c/c end bents along "1" line
LINE & Ls
Ta = 25° 04'
Ts = 857.33
Lmc = 855.33
Sta. "L" 1529+58.0 Bent 1 S
Sta. "L" 1529+98.0 Bent 2 S
"L" 1530+54.0 Bent 3 S
"L" 1531+24.0 Bent 4 S
"L" 1531+80.0 Bent 5 S
"L" 1532+25.0 Bent 6 S
"L" 1531+50 P.O.C. Ch. Ch. 10+00 P.O.T.
Bar Probe #3
Bar Probe #4
Test hole #2
Note: All bents are parallel to channel change &
30'-0" Rdwy. Width
15'0" 15'0"
Tan. to curve

ELEVATION S.B. LANE
Scale: 1"=20'

Fill
2' thick Rip rap El. 1828.0
High Water Elev. 1823.0 Dec. '55
Rip rap placed 3'-0" above H.W. mark El. 1826.0
& Ch. Ch.
62'-0" Channel bottom
Rip rap 15' Lt.
Ground Line & 15' Lt.
Fill
Ground Line & 15' Rt.
Ground Line &
No Scale

Ch. 1811.6 Rt. 1811.3
Ch. 1806.6 Rt. 1807.3
Ch. 1806.7 Rt. 1805.3
Ch. 1813.0 Rt. 1812.2

DESIGN DATA 56'-70'-56'

	A	B	C	D	E
M	+ 920	-1.297	-1.291	+ 870	
M = Ft-kip	92				
V	—	/28	/27	—	
V = kip					

TEST HOLES PROFILE
No Scale

A B C D E

Hole #1 Sta. 152+00 1810
Probe #3 Sta. 1529+50 Sta. 1531+70 50' Rt.
Probe #4 Sta. 1529+50 60' Rt.
Hole #2 Sta. 1531+70 Sta. 152+70 &
Sand, gravel & cobbles
Rock
Soil
1820

General Notes:

Bridges designed for H20-S16-44 and Military loading. Longitudinal beam loads are assumed to be distributed in accordance with Article 1-3.1(b) Case 1 of the specifications for bridges.

All concrete shall be class "A" and shall have a breaking strength of 3300 p.s.i. in 28 days. (f'c =1320 p.s.i.)

All reinforcing steel shall be intermediate grade deformed bars. Bars #3 shall be supplied in accordance with specification noted. All reinforcing bars #4 shall conform to A.S.T.M. specification A-408 and shall be lapped 30 diameters at all splices unless otherwise shown. All reinforcing bars shall be placed 2" clear of the nearest face of concrete unless noted or shown otherwise. (fs=20,000 p.s.i.) Column steel shall not be fabricated until final footing elevations have been determined in the field.

All materials and workmanship shall conform to the Oregon State Highway Commission Standard Specifications for bridges of the Oregon State Highway Commission.

GRADE LINE DIAGRAM
No Scale

Sta. "N" 1516+00
Sta. "N" 1516+00
Sta. 1526+00.0 P.I. North & South Bound Elev. 1820.79 N Bound Elev. 1820.31 S. Bound
2000' V.C. N & S Bound
Sta. "S" 1536+00
Sta. "N" 1536+00
Elevations shown are finish grade 12' Rt. N.B. & S.B.

Vicinity Map
Scale: 1"=400'

OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION

BEAR CREEK BRIDGE
N. ASHLAND INTCH.-S. ASHLAND INTCH.
PACIFIC HIGHWAY-JACKSON COUNTY

PLAN & ELEVATION

DATE 6/12/62
DRAWING NO. 1806.9
SHEET

Revised as constructed 3.26.64
Revise Deck Elev.
7/11/62

PLAN
Scale 1"=15'-0"

ELEVATION
Scale 1"=15'-0"

GRADE LINE DIAGRAM
No Scale

Sta. 0+00 Elev. 709.90
+0.555%
Sta. 3+50 Elev. 711.84
200' V.C.
Sta. 4+50 Elev. 712.40
Sta. 5+50 Elev. 711.70
-0.70%

Elevations shown are finish grade at Rdwy. ℄.

Elevation labels / soils (left profile):
Road Bed
Soft Mud
Ftg. Press. 2.757 T/ꟹ"
Clay-Tuff
Test Hole #4
Elev. 687
Elev. 692.0
Shale-Tuff
Elev. 680
Bent 1
Slope Pave Dwg. S-215
Ftg. Press. 2.537 T/ꟹ"
Bent 2
Elev. 684.0
Elev. 672.1
℄ South Bound Proposed
15'-0" Min. Cl.
Pt. "B"
Ftg. Press. 2.727 T/ꟹ"
Test Hole #2
Elev. 684
Bent 3
Elev. 674.1
℄ North Bound Future
Rockers
15'-1¼" Min. Cl.
Pt. "A"
Ftg. Press. 2.531 T/ꟹ"
Test Hole #3
Elev. 684.0
Elev. 68.3
Bent 4
Slope Pave Dwg. S-215
Ftg. Press. 2.757 T/ꟹ"
Soft Mud
Hard Clay-Tuff
Elev. 692.7
Elev. 687
Soft Blue Clay
Shale-Tuff
Test Hole #1
Elev. 690
Bedrock
Bent 5
Fill Road Bed

Plan dimensions (top):
35'-3" o-o
4'-7½" | 26'-0" Rdwy. | 4'-7½"
3'-7½" | 8'-8" 8'-8" 8'-8" 8'-8" | 3'-7½"

310'-0" ℄ to ℄ of end bents
15'-0" Concrete | 60'-0" Concrete span | 80'-0" ℄ to ℄ bent Steel span | 80'-0" ℄ to ℄ bent Steel span | 60'-0" Concrete span | 15'-0" Concrete

Station / elevation callouts:
Sta. 1+83.47 Elev. 710.92
Sta. 1+98.47 Elev. 711.00
Sta. 2+58.47 Elev. 711.34
Sta. 3+00 London Rd= Sta. 854+81.23 L3 P.O.C.
Sta. 3+38.47 Elev. 711.78
Sta. 3+76.94 London Rd= Sta. 855+54.60 L" P.O.C.
Sta. 4+18.47 Elev. 712.07
Sta. 4+78.47 Elev. 712.03
Sta. 4+93.47 Elev. 711.99

Pacific Hwy. South Bound
Pacific Hwy. North Bound
Std. catch basin Dwg. S-195
8" corr. metal drain pipe
S. 5°-18'-W)

GENERAL NOTES:

Bridge designed for H20-S16-44 loading. All longitudinal beam loads are assumed to be equally distributed among all beams. Live loads are calculated according to Case 1-3-3-1 and Case A-3-3-2 of the Oregon State Highway Specifications for bridges.

All concrete shall be class A and shall attain a break-ing strength of 3300 p.s.i. in 28 days (n=320 p.s.i.). All reinforcing steel shall be intermediate grade deformed bars. Bars from no. 3 to no. 11 inclusive shall conform to A.S.T.M. specification A 305-50T and shall be lapped 20 diameters at all splices unless shown or noted otherwise. 1⅛" bars shall be lapped 60 diameters at all splices unless shown or noted otherwise. All bars shall be placed 2 clear at nearest face of concrete unless shown or noted otherwise.

All materials and workmanship shall conform to the specifications for bridges of the Oregon State Highway Department.

All structural steel shall conform to specification for "Structural Steel for Welding" of the A.S.T.M. designation A-373-54T except as noted otherwise.

PLAN
Scale 1/8"=1'-0"

ELEVATION
Scale 1/8"=1'-0"

Note:
Elevations shown are
at gutters, 1/8" below
finished grade to allow
for future A.C.W.S.

Bent 1 — Elev. 600.0
Sta. 9+30.0 G Elev. 626.93

Bent 2 — Elev. 600.0
Sta. 9+58.0 G Elev. 626.78

Bent 3 — Elev. 600.0
Sta. 10+04.0 G Elev. 627.04
P.I. Sta. 10+20.0

Bent 4 — Elev. 600.0
Sta. 10+32.0 G Elev. 627.09

Bent 5 — Elev. 600.0
Sta. 10+78.0 G Elev. 626.83

Bent 6 — Elev. 600.0
Sta. 11+06.0 G Elev. 626.48

Hinge
Fill
Slope 1½:1
Std. slope paving, see dwg. S-215
Std. handrail (1/120)
15.60' min. cl.
Solid Rock
Ground line
1½:1 slope
Hinge
Std. catch basins (Dwg. S-195)
¾" mastic joint thru rail and curb
Edge of Pvmt.
South bound ₵
Point of minimum clearance
Future North bound ₵
8" cast iron pipe to toe of fill.
90°

30'-10" o-o
26'-0" cl. rdwy.
1'-6" 7'-8" 3'-10" 3'-10" 7'-8" 1'-6"
28'-0" 46'-0"
116'-0" c/c-c/c end bents
74'-0" c-c roadways
10'-0" 12'-0" 12'-0" 10'-0"

2 equal panels
3 equal panels
4 equal panels
2 equal panels
3 equal panels

Bm. "C" Bm. "W" Bm. "X" Bm. "D" Bm. "E" Bm. "B" Bm. "G"

GENERAL NOTES:
All concrete shall be class "A" mix and shall attain a breaking
strength of 3300 p.s.i. in 28 days (f'c=1320 psi).
All reinforcing steel shall be intermediate grade deformed bars.
Bars from number 3 to number 11 inclusive shall conform to
A.S.T.M. specifications A305-50T. (fs=20,000 psi.) All bars
shall be placed 2"clear of nearest face of concrete unless
noted or shown otherwise and shall be lapped 20 diameters at
all splices unless shown or noted otherwise.
Design Load=H20-S16-44. Longitudinal beams are designed
for 2 G. lanes with loads assumed as being equally distributed
among all four bms.

Note:
Details revised January 20, 1955 from 3 span structure
to ultimate 5 span structure.

Elev. are finished grade 12" left or right of roadway ₵
"L" LINE

B.V.C. Sta. 7+20.0 Elev. 620.0
Sta. 550+00 Elev. 590.52
+0.30%
+.50%
600 V.C.
P.I. Sta. 10+20.0 Elev. 635.00
Sta. 672+50.0 Elev. 625.77
Sta. 11+06.0 G
E.V.C. Sta. 13+20.0 Elev. 620.0
-.50%

OREGON STATE HIGHWAY DEPARTMENT
BRIDGE DIVISION

McKENZIE RIVER OVERFLOW BRIDGES
AT STA. 402+32.50 & STA. 40G+03.50
McKENZIE RIVER-WILLAMETTE RIVER SECTION
PACIFIC HIGHWAY-LANE COUNTY

PLAN & ELEVATION

DATE JULY 23/59	BRIDGE NO 8180
ACCOMPANIED BY DWGS	
SEE "PLAN & ELEVATION"	SHEET OF

DRAWING NO. **15283**

APPROVED
STATE HIGHWAY ENGINEER
BRIDGE ENGINEER
ASST. HIGHWAY ENGINEER

| DESIGNED F.G. | CHECKED |
| DRAWN F.G. | CALC BOOK |

BRIDGE No. 8180
ACCOMPANIED BY DWG'S 15284, 15286, 15287, 14660 & 14382

E L E V A T I O N
SCALE 1"=15'-0"

P L A N

Bent No.1 Bent No.2 Bent No.3 Bent No.4
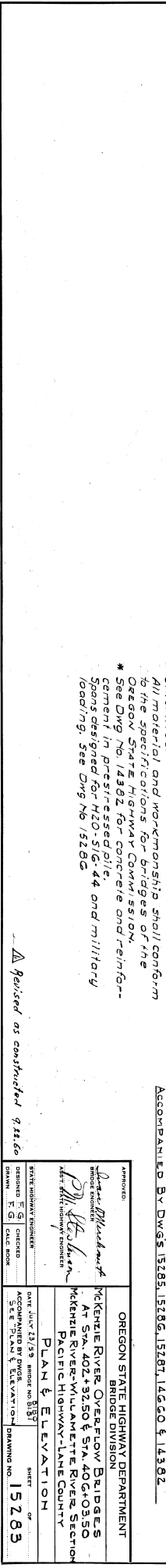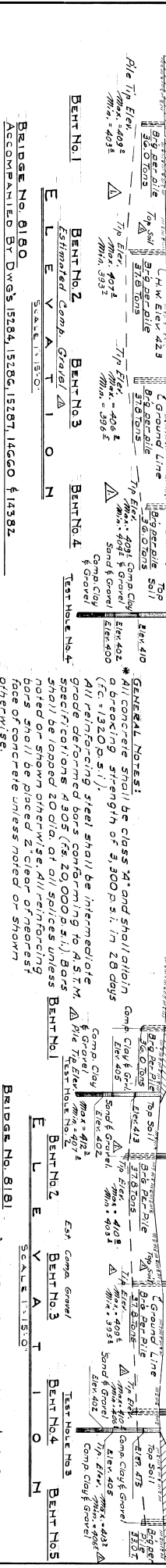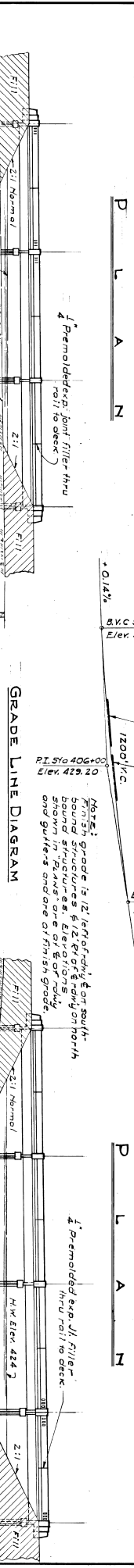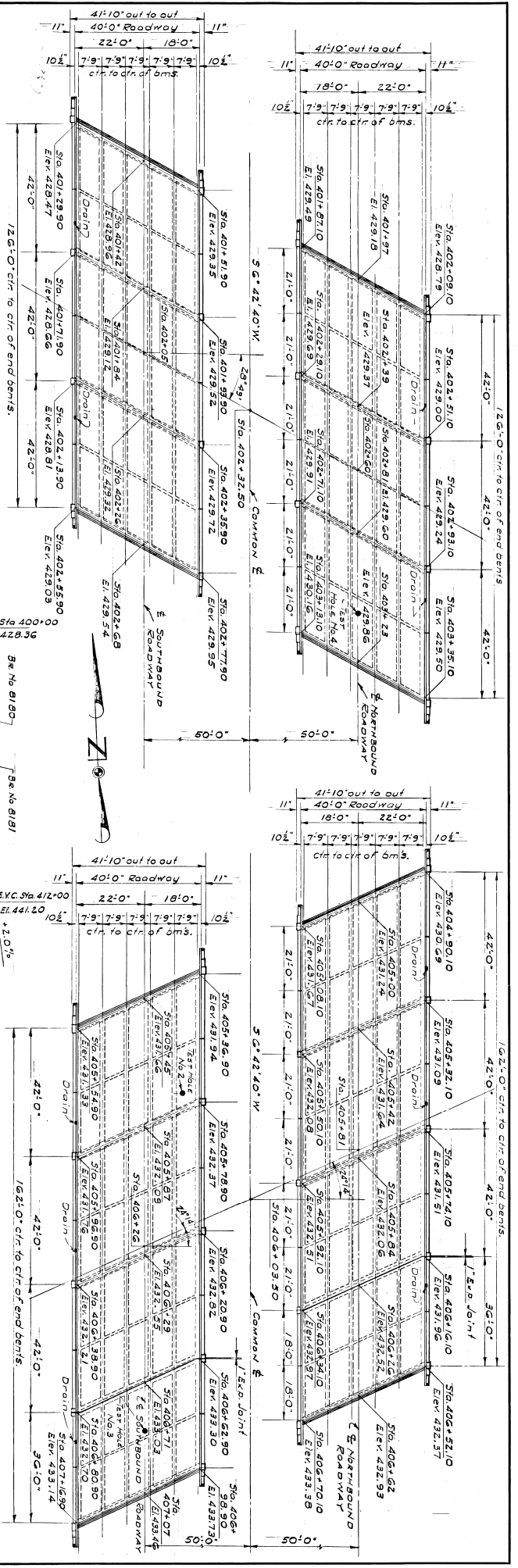
BRIDGE No. 8181
ACCOMPANIED BY DWG'S 15285, 15286, 15287, 14660 & 14382

E L E V A T I O N
SCALE 1"=15'-0"

P L A N

Bent No.1 Bent No.2 Bent No.3 Bent No.4 Bent No.5

GRADE LINE DIAGRAM

B.V.C. Sta 400+00 Elev. 428.36
P.I. Sta 406+00 Elev. 429.20
E.V.C. Sta 412+00 El. 441.20
Br. No 8180
Br. No 8181
+0.14%
+2.0%
1200' V.C.

N

GENERAL NOTES:
All concrete shall be class "A" and shall attain a breaking strength of 3,300 p.s.i. in 28 days (f'c=1320 p.s.i).
All reinforcing steel shall be intermediate grade deformed bars conforming to A.S.T.M. specifications A305 (fs 20,000 p.s.i.) Bars shall be lapped 20 dia. at all splices unless noted or shown otherwise. All reinforcing bars shall be placed 2" clear of nearest face of concrete unless noted or shown otherwise.
All material and workmanship shall conform to the specifications for bridges of the Oregon State Highway Commission.
See Dwg No. 14382 for concrete and reinforcement in prestressed pile.
Spans designed for H20-S16-44 and military loading. See Dwg No. 15286.

41'-10" out to out
40'-0" Roadway
22'-0"   18'-0"
ctr. to ctr. of bms.

C. NORTHBOUND ROADWAY
C. SOUTHBOUND ROADWAY
Common C
50'-0"

PLAN & ELEVATION

EAST GRANTS PASS INTERCHANGE
GRANTS PASS ~ EVANS CREEK SECT.
PACIFIC HIGHWAY
JOSEPHINE CO.

DATE 4-19-60 BRIDGE No. 834I
SHEET 1 of 15
DRAWING No. 15802

GENERAL NOTES

Bridge designed for H20-S16-44 and military loads.
All concrete shall be class "A", and shall attain a breaking strength of 3300 p.s.i. in 28 days.
All reinforcing steel shall be intermediate grade deformed bars. Bars #3 thru #11 shall conform to A.S.T.M. Specification A-305, and shall be lapped 20 diameters at all splices unless noted or shown otherwise. #14 bars shall conform to A.S.T.M. Specification A-408 and shall be lapped 35 diameters at all splices unless noted or shown otherwise. All bars shall be placed 2" clear of nearest face of concrete unless noted or shown otherwise. (f's = 20,000 p.s.i.)

Spread Footings ----- 5 T/□

10 BP 42 w/Reinforced Tip-50% pile

10 BP 42 Plain Tip ---- 37T/pile

Footing elevations are subject to change depending on foundation material encountered, and reinforcing for columns shall not be fabricated until final footing elevations have been established in the field.

All materials and workmanship shall conform to the specifications for bridges of the Oregon State Highway Commission.

△ As constructed 2-22-73
△ Grades revised 7-26-60 to fit profile from field dated 7-18-60
△ Revised 5-13-60 - Slope paving removed from fill slope.

SUMMARY OF DESIGN DATA

Moments in ft. kips - Shears and Reactions in kips

MILITARY LOADING

Lane Load Width
6'-0 Min.   2'-0 Min.

24 K axle
4'-0"
24 K axle

"A" LINE PROFILE

Sta. 153+50.00 Elev. 1025.00
Sta. 163+10.00 Elev. 1082.60
300 V.C.
1100 V.C.
+ 6.00 %
- 2.816 %

SUPER ELEVATION DIAGRAM

"L" LINE GRADE

Finish Profile grade 32'Lt.
155+45 Begin Structure
158+80 End St.
- 2.816 %
2000' V.C.

Notes:
Profile grade is .004 below grade shown on paving plans because of change from A.C. to P.C.C. surfacing and reduction of 3" in base thickness.

PLAN
Scale 1"=20'

ELEVATION
Scale 1"=20'-0"

NOTE
Bent and intersection angles tangent to curve.

7'-0"± Rock broken by ripper 15'Rt. of ℄ to 47'Lt. of ℄.

Maintain drainage during construction. Restore 6" earth-pipe upon completion of bent.

See Dwg.#15812 for Median Barrier Bolt details

Type "A" Deck Drains at L.H. gutter, Bents 3 and 4, see Dwg.#4660 - Extend drain pipe down column to ditch

Std. Pipe Parapet Handrail See Drawing No. 14657

PROFILES
35 Rt.
47' Lt.

TEST HOLE DATA
Solid rock at 2 to 5 feet below ground surface.
See Plan View for test hole & bar locations.