DEVELOPMENT OF A DESIGN PROCEDURE FOR CONCRETE TRAFFIC BARRIER ATTACHMENTS TO BRIDGE DECKS UTILIZING EPOXY CONCRETE ANCHORS

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DEVELOPMENT OF A DESIGN PROCEDURE FOR
CONCRETE TRAFFIC BARRIER ATTACHMENTS TO
BRIDGE DECKS UTILIZING EPOXY CONCRETE ANCHORS

by

Benjamin James Dickey

A THESIS

Presented to the Faculty of
The Graduate College at the University of Nebraska
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A guide for the design of epoxy anchors for the use in concrete traffic barriers was developed in this study. A series of dynamic bogie tests and static tests was conducted on single and double anchors. The dynamic increase factor for the noted anchors was determined to be 1.06. Protective epoxy coatings for the anchors was observed to contribute to a 9 percent decrease in anchorage strength. The effects of anchors located in close proximity to each other was also investigated. Calibration factors were developed to apply recommended changes to the ACI code for use in bridge rail anchorages.

A load model was developed to estimate the lateral impact loads for non-articulated vehicular crashes into rigid barriers with an impact severity of up to 300 kip-ft (407 kJ). This model was determined from a regression analysis of data obtained from full-scale crash tests. The model consists of a single equation that is a function of the impact severity. This load model was recommended for use with the yield-line analysis procedure to determine the proper anchorage design for bridge railings.


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CHAPTER 1 - INTRODUCTION

1.1 Background

The traditional method of attaching concrete traffic barriers to bridge deck has been to cast concrete around reinforcing bars that protrude out of the top of the slab. The shortcomings of this technique is that it requires hand finishing of large portions of the deck and has less flexibility to be utilized for a variety of different traffic barriers due to the fact that these anchors can only be installed before the slab is poured. Alternative methods of attachments include post-installed mechanical, adhesive, and bolt-through designs. Previous testing has been conducted on bolt-through designs. However, epoxy anchorages have had limited testing for bridge rail applications.

Epoxy anchors are capable of developing the full strength of the surrounding concrete and can provide tensile and shear strengths comparable to any cast-in-place strait bar attachment. The epoxy is stronger than the surrounding concrete and distributes the anchor loads over a larger area of the concrete which can result in higher capacities for epoxy anchors than strait cast-in-place bars with similar embedment depths. However, the cast-in-place bars usually contain bent hooks at the end of the embedment depth to increase their strength to capacities that cannot be normally matched by epoxy anchors with limited embedment.

The rated capacities published by epoxy anchor manufactures are largely based on static tests and contain large safety factors. When used in conjunction with traffic barriers, epoxy anchors can resist much higher capacities under impact loading conditions. Therefore, it is overly conservative to design traffic barrier anchors based solely on their published load ratings.
All anchor components used in bridge rail applications are required to have some sort of corrosion protection in order to ensure long term durability. However, published ratings are based on testing without any corrosion protection. Corrosion protection could lead to different anchor strengths compared to black steel strengths due to the varying frictional resistance of the corrosion protection surfaces. The dynamic testing conducted to determine the dynamic capacity of epoxy anchors needed to include the appropriate corrosion protection as a design consideration.

1.2 Research Objectives

The purpose of this research study was to determine if epoxy masonry anchors can be utilized to anchor crash barriers to bridge decks and create a design guide that can be used to configure epoxy anchorages for a variety of concrete bridge railings. This would allow for the installation of precast aesthetic concrete traffic barriers or in-board cast-in-place traffic barriers without the need to cast reinforcing steel into the deck surface to anchor the barrier. Also, the epoxy anchors could potentially be used to anchor temporary concrete barriers or retrofit permanent bridge railings.

1.3 Research Approach

The research project began with a literature review of previously developed design procedures for estimating the capacity of adhesive anchors for both static and dynamic loading conditions. A dynamic uniform bond stress model was then developed based on the findings of the literature review and the mechanics involved with epoxy adhesive anchors. A series of 16 dynamic bogie tests were conducted to refine and verify the accuracy of the model. A static test was also conducted to investigate the strain rate effects of epoxy anchorages.
CHAPTER 2 - LITERATURE REVIEW

The first phase of the research process consisted of a literature review of publications about the analysis, design, and behavior of adhesive anchors under static and dynamic loading conditions. A review of the manufacturers’ specifications and Pooled Fund State standards for bridge railings was also investigated to identify the anticipated anchor sizes and requirements.

2.1 Design Standards

A Load and Resistance Factor Design (LRFD) specification for the design of cast-in-place and post-installed mechanical concrete anchors is included in Appendix D of the American Concrete Institute (ACI) publication ACI 318-08, *Building Code Requirements for Structural Concrete and Commentary* [1]. This procedure details the design of single concrete anchors as a function of the material and geometric properties. It also includes procedures to adjust the strength of anchor groups based on the spacing and edge distances from other anchors and concrete edges. An interaction equation is included that allows for the design of an anchor loaded under simultaneous shear and tension. Strength and reduction factors are provided for the various failure mechanisms to ensure a statistically acceptable measure of reliability.

Several of the design procedures for estimating the capacity of concrete anchors presented in ACI 318-08 come from the Concrete Capacity Design (CCD) method. The CCD method is a simpler design procedure than the one contained in ACI 349-85, *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*. For calculation of the concrete breakout strength of anchors in tension, ACI 349-85 assumes a concrete cone shape with the fracture line angled at 45 degrees from the concrete surface
Alternatively, the CCD method assumes a pyramidal concrete shape with the fracture line angled at 35 degrees from the concrete surface to approximate an idealized cone. This allows for easier calculations of the projected failure surface, especially for a group of closely spaced anchors or anchors located near a concrete edge.

A study conducted by Breen, Eligehausen, Fuchs, and Werner evaluated the accuracy of the CCD method and the method presented in ACI 349-85 compared to a database of test data [2]. The CCD method correlated rather well with the mean test results for both shear and tensile loads. The procedure presented in ACI 349-85 was found to be conservative for shallow embedment depths and un-conservative for deep embedment depths. However, the CCD method requires greater spacing and edge distances to develop the full capacity strength for both shear and tensile forces. Based on the simpler design procedure and accuracy obtained by the CCD method, the CCD method was recommended over the procedure presented in ACI 349-85.

The method presented in ACI 318-08 does not include provisions to design adhesive anchors embedded in concrete. ACI is currently working on developing a specification that incorporates a design procedure to account for the mechanics of adhesive bonded anchors. Until this study is complete, The International Code Council Evaluation Services Inc. (ICC-ES) publication AC308, Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements, is being used as an interim design and product approval standard [3]. The design procedure presented in ICC-ES AC308 provides additional and substitutive sections that allow the anchorage procedure in ACI 318-08 to be used in accordance with ICC-ES AC308 to meet the design requirements of adhesive anchors [4].
A uniform bond stress theory is the basis for calculating the pullout strength of anchors in tension in ICC-ES AC308. The equation used in the uniform bond stress model to calculate the mean nominal tensile strength \( N_n \) is shown in Equation (1) and is a function of the uniform bond stress \( \tau_0 \), anchor diameter \( d \), and anchor embedment depth \( h_{ef} \). Due to the similar behavior of adhesive and cast-in-place or mechanical anchors, the shear design procedure in AC308 is nearly identical to the procedure in ACI 318-08.

\[
N_n = \tau_0 \pi dh_{ef}
\] 

(1)

A great difficulty exists in developing a standard for estimating the capacities of adhesive anchors due to the wide variation of many different manufacturers. This is because of the material properties (i.e. bond stress) can vary for each particular product. For this reason, many designers have utilized manufacturers’ specifications based on test data to design adhesive anchors. Essentially ICC-ES AC308 provides a more generalized procedure for designing adhesive anchors based on parameters obtained from test data. However, many of the design parameters require extensive testing for each particular product.

2.2 Previous Research for Static Tensile Loads

Previous research has been conducted on adhesive anchors embedded in concrete and subjected to static tensile loading conditions. Most of these research projects focused on developing a theoretical model for predicting the ultimate tensile strength and conducted tests using a hydraulic ram test machine to validate the proposed theories. Much debate has existed over how the loads are transferred across the adhesive interface.
Two main theories have been proposed: (1) a uniform bond stress distribution over the entire embedment depth and (2) an elastic bond stress distribution.

In 1984, Luke published a thesis that summarized the findings of 69 reinforcing bar pullout tests that utilized an epoxy adhesive as the bonding agent [5]. Four different failure mechanisms were identified and observed: (1) fracturing/yielding of the dowel bar, (2) pullout/excessive slip of the dowel bar, (3) cone failure of the concrete, and (4) splitting of the concrete. In most cases, a combination of a cone failure and dowel bar pullout was present. Both single and double concrete cone failures were observed (Figure 1). The single cone failures had uniform sloped edges at the concrete failure surface. Double cone failures were similar to the single cone failures except that a flexural cone surface of lesser slope was located near the concrete surface. There was not a noticeable difference in the pullout strengths observed between the two cone types. However, the double cone failure generally occurred on bars with deeper embedment depths.

![Figure 1. Single versus Double Concrete Cone Failures](image-url)
During the study, several cleaning methods were investigated and it was concluded that the drilled holes should be thoroughly cleaned by repeated vacuuming and brushing with a stiff bottle brush or a wire brush. Failure occurred mainly along the epoxy-concrete interface for unclean holes and in some cases the concrete cone did not form. For very clean holes, the failure occurred along either the epoxy-steel or epoxy-concrete interfaces. The cleaner holes generally lead to a high pullout strength which suggests that adhesion plays an important role in the load transfer at the adhesive interface.

In 1989, Doerr and Klingner suggested that an adhesive anchor loaded in tension has three failure modes that include fracture of the anchor steel, pullout of the adhesive core, and cone failure of the concrete (with some core pullout) [6]. A test procedure was conducted that consisted of 105 threaded rod specimens adhesively bonded to concrete with embedment depths between 4 and 8 in. (102 and 203 mm). A bond stress distribution model using an elastic solution accurately predicted the test results. The elastic model is based on Equation (2), which is a function of the maximum bond stress \(\tau_{\text{max}}\), hole diameter \(d_0\), anchor embedment depth \(h_{ef}\), and the adhesive stiffness parameter \(\lambda'\).

\[
N_n = \left(\frac{\pi \tau_{\text{max}} d_0^{1.5}}{\lambda'}\right) \tanh\left(\frac{\lambda'(h_{ef} - 2)}{\sqrt{d_0}}\right) \tag{2}
\]

A uniform stress distribution model was reasonably consistent with the test results for short embedment depths (less than 8 in. (203 mm)), but grossly overestimated the capacity of longer embedment depths. The most common failure mode was the formation of a shallow concrete cone accompanied by the pullout of the adhesive core, but the
concrete cone did not significantly increase the anchor strength. It was concluded that for short embedment depths the capacity of a fully bonded anchor could be closely approximated by the capacity of a partially bonded anchor with the adhesive length equal to the embedment depth less the height of the concrete cone. The height of the concrete cone had an average depth of 1 to 2 in. (25 to 51 mm).

Also in 1989, Collins, Klingner, and Polyzois published a report on the study of several different types of cast-in-place and post-installed anchors [7]. The load transfers for adhesive anchors were found to be dependent on the mechanical interlock and chemical bond between both the adhesive and the concrete and the adhesive and the anchor steel. The failure modes of tensile pullout tests included fracture of the anchor shank, cone failure of the concrete, pullout of the anchor, and pullout of the anchor accompanied by a concrete cone. The anchor pullout behavior occurred with the failure of the bond surfaces between both the adhesive-concrete and adhesive-steel interfaces. However, only a few of the anchors tested failed at the bond surface between the adhesive and the steel.

Two different bond stress models were developed to predict the pullout capacity of adhesive anchors and the concrete cone depth. The first assumed a uniform bond stress over the entire embedment depth while the second assumed a linear stress distribution with the bond stress equal to zero at the bottom of the embedded end of the anchor and the maximum bond stress at the concrete surface. Most of the test specimens failed by the formation of a concrete cone radiating outward from the anchor head with a depth between 1 and 2 in. (25.4 and 50.8 mm). Based on the test data, the height of the concrete cone decreased with an increase in the embedment depth. Because of this phenomenon,
non-uniform bond stress model was suggested because the test data indicated that a non-uniform stress distribution was present. Analysis using finite element methods suggested that the bond stress of adhesive anchors is not only non-uniform, but non-linear with the highest bond stress near the surface of the concrete and a bond stress near zero at the embedded end of the anchor.

Cook reviewed several models for predicting the strength of adhesive anchors and developed a new model that was based on three modes of failure which varied with the anchor embedment depth [8]. Two models of bond failure, a uniform and an elastic stress distribution, were analyzed with a database of test data to determine the proper use of each model. The elastic bond stress model matched well with the uniform bond stress model up to bonded length of 40 times the square root of the hole diameter in millimeters. A graph of the two models verses embedment depth is shown in Figure 2.

Cook developed an equation that estimated the height of the concrete cone ($h_{cone}$) that was a function of the uniform bond stress ($\tau_0$), the diameter of the hole ($d_0$), and the compressive strength of the concrete ($f'_c$) as shown by Equation (3). Note, this equation did not agree with the observations by Collins, Klingner, and Polyzois that the height of the concrete cone varied with the anchor embedment depth [7]. Cook suggested that for embedment depths less than the calculated cone height, the concrete cone model should be utilized. The primary variable in the equation to calculate the mean nominal tensile strength ($N_{nt}$) for the concrete cone model of adhesive anchors was the embedment depth ($h_{ef}$) as can be seen in Equation (4).

$$h_{cone} = \frac{\tau_0 \pi d_0}{1.84\sqrt{f'_c}}$$  \hspace{1cm} (3)
Figure 2. Comparison of Uniform and Elastic Models versus Embedment Depth [8]

\[ N_n = 0.92h_{ef}^2 \sqrt{f'_c} \]  \hspace{1cm} (4)

For embedment depths greater than the height of the concrete cone but less than 40 times the square root of the hole diameter plus the height of the concrete cone (in millimeters), a uniform stress distribution model with the concrete cone was suggested, shown by Equation (5). For greater embedment depths, the elastic bond stress model with the concrete cone should be used. Equation (6) was suggested to calculate the mean nominal tensile strength for the elastic model and utilizes the following additional adhesive properties: the maximum bond stress \( \tau_{max} \) and the adhesive stiffness parameter \( \lambda' \). The cone breakout strength is not included in this equation because it has a negligible effect on the capacity of anchors with deep embedment depths.
This method fit well with the results from the test database and agreed well with the conclusions made by Doerr and Klinger that the uniform bond stress model fit the test data for short embedment depths [6]. The equations are based on the geometric variables of the anchor and three basic bond properties: (1) the uniform bond stress, (2) the maximum bond stress, and (3) the adhesive stiffness parameter.

A report by Biller, Cook, Fagundo, and Richardson in 1991 detailed test procedures for determining those three adhesive bond properties [9]. The uniform bond stress was calculated as the failure load obtained from a confined tensile test using a hydraulic ram divided by the bonded area. A confined tensile test consisted of placing a bearing surface closely around the anchor to prevent a concrete cone breakout. The applied load and the displacement were measured at increments great enough to develop a load-displacement graph. The failure load was determined as the point on the load-displacement plot where the graph began to deviate from a straight line. The stiffness parameter of the adhesive was based on the slope of the linear portion of the load-displacement graph on a specimen where the steel was loaded past it yielding point.

In 1993, Cook, Doerr, and Klinger published a journal article that verified the accuracy of the elastic model with experimental data [10]. A procedure for calculating the maximum bond stress was proposed that consisted of conducting a pullout test of a partially bonded anchor with the top two inches not bonded to the concrete. This lowered the point of load transfer so that a concrete cone did not form, and the capacities of the
partially bonded anchors were only dependent on the adhesive bond. The maximum bond stress was calculated as the ultimate load divided by the bonded area. An alternate method for calculating the stiffness parameter of the adhesive was determined by a least-squares fit between the test data and Equation (6).

A simultaneous combined cone and bond failure model was derived based on the elastic model which is shown in Equation (7) [10]. This equation included the approximate angle of the concrete cone fracture line relative to the concrete surface (α) and the effective concrete tensile stress over the projected area of the cone ($f_t$).

$$N_n = f_t \pi \left( \frac{h_{cone}}{\tanh(\alpha)} \right) \frac{\sinh \left( \lambda' \frac{h_{ef}}{\sqrt{d_0}} \right)}{\sinh \left( \lambda' \frac{h_{ef}}{\sqrt{d_0}} \right) - \sinh \left( \lambda'(h_{ef} - h_{cone}) \right) \sqrt{d_0}}$$

(7)

This model overestimated the tensile capacities compared to the experimental data. Some possible reasons for the inaccuracy of this model are that the bond strength did not appear in the equation, and the equation included the tensile strength of the concrete, which was highly variable. The elastic bond failure model, shown by Equation (6) proved to be more accurate based on the experimental data. Since this model assumed that the bond failure occurred after the concrete cone failure, the capacity of the anchor was only dependent on the bond stress below the concrete cone.

Further, strength reduction factors were suggested based on the calculated capacity in relation to the horizontal asymptote of the elastic model, calculated by Equation (8) [10]. A higher strength reduction factor of 0.80 was utilized when the calculated capacity was greater than or equal to 95 percent of the horizontal asymptote, while a smaller reduction factor of 0.65 was utilized when the calculated capacity was
below 95 percent of the horizontal asymptote. The more conservative reduction factor was suggested to be used with shorter embedment depths because a greater drop in the capacity was observed on the elastic bond stress model with decreasing embedment depth.

\[
\text{Elastic Model Horizontal Asymptote} = \frac{\pi \tau_{\text{max}} d^{1.5}}{\lambda'}
\]  

(8)

The effect of anchor spacing was also investigated in the same study. Closely spaced, fully bonded anchors had small overlaps of the concrete cones that contributed to only a small reduction in capacity. Therefore, it was suggested that anchor spacing had a negligible effect on the capacity of a group of anchors. However, for anchor groups with hole diameters between 0.5 and 1.0 in. (13 and 25 mm) and spacings less than 8 in. (203 mm), a capacity reduction of 15 percent should be utilized to account for the uncertainty that the effects of the overlapping cones were negligible and should be used in the design procedure until more extensive testing confirms this assumption. No reduction in capacity was required for spacings greater than 8 in. (203 mm) for anchors with hole diameters between 0.5 and 1.0 in. (13 and 25 mm).

In 1996, Cook, Krishnamurthy, and McVay reviewed previous empirical and theoretical methods for predicting the failure of chemically bonded anchors loaded in tension, developed an elasto-plastic finite element model of an adhesive anchor, and compared the numerical results to experimental data [11]. The results of the numerical analysis indicated that the elastic model corresponded closely to anchors with relatively low loads, and the bond stress distribution at higher loads resembled a somewhat uniform bond stress. This occurred because at high loads the epoxy adhesive and the concrete began to yield, which redistributed the stress toward the bottom of the adhesive layer.
Figure 3 shows a printout of the shear stress distribution obtained from the finite element model along the epoxy-concrete interface of an adhesive anchor with an embedment depth of 5 in. (127 mm). Five different solutions are shown with increasing applied loads. The left-most line shows the elastic solution that corresponds to a relatively low applied load while the right-most line shows the solution that corresponds to a high applied load. A transition from an elastic bond stress distribution to a relatively uniform bond stress distribution is shown by the middle lines as the materials begin to yield when the load is increased. A uniform average bond stress applied over the entire embedded anchor area did an excellent job of predicting the tensile failure capacity of the chemically bonded anchors studied.

In 1998, Cook, Fuchs, Konz, and Kunz published an article that reviewed several previously developed models for predicting the tensile capacities of adhesive anchors.
The models were statistically compared to a worldwide database of test data to determine the accuracy and precision of each method based on varying concrete strength. A new model was then developed that statistically better fit the database of pullout tests that were analyzed. This model was based on the uniform bond stress model with an added coefficient to account for the effect of the concrete strength. Equation (9) shows the modified equation as a function of the uniform bond stress \( (\tau_0) \), anchor diameter \( (d) \), embedment depth \( (h_{ef}) \), and the modification factor for concrete strength \( (\psi_c) \). The modification factor for concrete strength was based on a function of variables determined by tests of individual adhesive products in various concrete strengths.

\[
N_n = \tau_0 \pi d h_{ef} \psi_c
\]  

(9)

The concrete cone model, shown in Equation (4), provided the worst fit to the database because of the inherent differences between the behavior of adhesive and mechanical anchors. The uniform bond stress model with and without the shallow concrete cone, shown in Equation (10), provided a good fit to the test data. However, the bond model that neglected the stress at the top of the anchor was considered not viable as the stress distribution was not correct compared to finite element studies. The cone with a bond model consisted of either Equation (1) or Equation (4) that could be utilized based on the mode of either cone or bond failure. This method was ruled out as the uniform bond equation fit the data better than the cone equation when a cone failure occurred. The combined concrete cone and bond failure model, shown in Equation (11), provided the best theoretical analysis of adhesive anchors since it accounted for both the failure modes present (partial concrete breakout and partial bond failure), but did not provide as good of a fit to the database as the uniform bond stress model, which was also easier to
implement. The two-interfaced bond model consisted of two equations that could be utilized based on the bond failure mode of either the adhesive-steel or the adhesive-concrete interfaces. This model provided the best fit to the database with significant variation in concrete strength. However, it was difficult to adequately distinguish between an adhesive-steel and an adhesive-concrete failure.

\[ N_n = \tau_0 \pi d(h_{ef} - 3d) \]  

\[ N_n = 0.92 \cdot \frac{h^2}{f_c'} + \tau_0 \pi d(h_{ef} - h_{cone}) \]

Implementation of the coefficient for the concrete strength in the uniform bond stress model reduced the overall coefficient of variation from 0.218 to 0.203. This modified uniform bond stress model exhibited the best fit to the database of all the previously reviewed methods and was suggested for implementation in future specifications. It also agreed with nonlinear analytical studies of the adhesive anchor system.

In 2006, Appl, Cook, and Eligehausen published an article that proposed a behavioral model for predicting the average failure load of adhesive bonded single anchors and groups of anchors loaded in tension [13]. The method developed was similar to the method presented in Appendix D of ACI 318-08 and the CCD method based on the square concrete cone assumption. Several numerical analyses were conducted using a three-dimensional nonlinear finite element code and was compared to the predicted loads of the model as well as a database of test results.

The design provisions in ACI 318-08 for the steel strength in tension were considered to be applicable to adhesive anchors. A new equation was developed for the pullout capacity of adhesive anchors was based on a uniform bond stress model and is
shown in Equation (12). This equation utilized the uniform bond stress at the adhesive-steel interface ($\tau$) instead of previous studies where the uniform bond stress was based on the adhesive-concrete interface. The ratio of the projected concrete failure area of a single or group of anchors ($A_{Nc}$) to the area of the projected concrete failure area of a single anchor ($A_{Nco}$) was used to account for the overlapping of the concrete cones. These projected areas are shown in Figure 4.

$$N_n = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{g,N} \tau \pi d h_{ef}$$  \hspace{1cm} (12)

![Projected Concrete Failure Areas for Adhesive Anchors](image)

(a) Single anchor away from edges and other anchors  
(b) Four anchor group with close spacing and located near a corner

Figure 4. Projected Concrete Failure Areas for Adhesive Anchors [13]

The factor used to modify the tensile strength of anchors based on the proximity to the edges of a concrete member ($\psi_{ed,N}$) and the factor used to modify the tensile strength of adhesive anchors based on the number and spacing of anchors in a group and the mean bond strength ($\psi_{g,N}$) were utilized in the developed model. This model agreed
with the results obtained from the test database and closely resembles the design procedure that was adopted in ICC-ES AC308 and ACI 318.

The critical anchor spacing \( s_{cr} \) was defined as the minimum spacing between anchors where the strength of the anchor group was not influenced by the close proximity of the anchors. Equations that calculated the critical anchor spacing were derived by a regression analysis of several anchor tests where the spacing was varied. They are shown in Equation (13) for both English and Metric units.

\[
s_{cr,\text{in}} = 20d \left( \frac{\tau}{1450} \right)^{0.5} \quad (13) (a)
\]

\[
s_{cr,\text{mm}} = 20d \left( \frac{\tau}{10} \right)^{0.5} \quad (13) (b)
\]

According to the results obtained from the test program, it was observed that the failure load of adhesive anchors was limited to the concrete breakout failure load of post-installed mechanical anchors. An equation for the maximum bond strength was derived by setting the equation for the capacity of post-installed mechanical anchors equal to the uniform bond stress equation and solving for the bond stress. The resulting equation for the maximum bond stress is shown in Equation (14) for both English and Metric units.

\[
\tau_{\text{max,psi}} = \frac{11.1 \sqrt{f_c} \sqrt{h_{ef}}}{d} \quad (14) (a)
\]

\[
\tau_{\text{max,MPa}} = \frac{4.7 \sqrt{f_c} \sqrt{h_{ef}}}{d} \quad (14) (b)
\]

### 2.3 Previous Research for Static Shear Loads

Bickel and Shaikh conducted a study to determine the differences in capacities of concrete headed and adhesive anchors loaded in shear [14]. Two different design methods based on the shear strength of headed studs were statistically analyzed to
determine if the models could be used to predict the shear strength of adhesive anchors. One was based on the Precast/Prestressed Concrete Institute (PCI) Handbook and the other was based on the CCD method. The CCD method consists of calculating a failure surface area to determine the concrete shear strength while the PCI method is a function of the distance from a free concrete edge.

The failure behaviors of adhesive and headed stud anchors loaded in shear are similar because both anchors bear on the concrete. However, adhesive anchors generally had higher shear capacities compared to headed studs because the adhesive allowed for the stresses to distribute more uniformly over a larger portion of the embedment depth. Based on statistical analysis, both the PCI and the CCD methods were more accurate and more conservative in predicting the adhesive anchor capacities than the headed stud capacities.

A regression analysis of the test data was performed for each method. It was suggested to change the calibration coefficient in the PCI method from 12.5 to 15 to better predict the capacity of adhesive anchors. The resulting shear strength is shown in Equation (15) where \( d_e \) is the distance from the anchor to a free concrete edge.

\[
V_n = 15\sqrt{f_c} d_e^{1.5}
\]  

(15)

The CCD equation was modified by changing the calibration coefficient and the exponents of the variables. It was suggested that the modified equation in the CCD method would accurately and conservatively predict the strength of adhesive anchors loaded in shear. This study was limited to single anchors only and did not include an investigation of whether the modification factors presented in the models accounting for anchor locations near free edges could be used.
2.4 Previous Research for Dynamic Tensile Loads

Most manufacturers publish their rated capacities based on static testing or specifications developed to estimate the static load capacities. Further, the methods for estimating the capacities presented by ICC-ES AC308 and ACI 318 were developed based on static behavior. However, the loading rate has an influence on the behavior of adhesive anchors bonded to concrete. Dynamic capacities of adhesive anchors are generally higher than the rated static capacities. In previous research, many have attempted to correlate the static load capacities with dynamic capacities.

In 2003, Fujikak, Ishibashi, Mindess, Nakayama, and Sato conducted several tests on chemically bonded anchors subject to various tensile dynamic loading rates [15]. A dynamic increase factor (DIF) was defined as the ratio of the average dynamic ultimate bond strength to the average static ultimate bond strength. It was observed that the dynamic increase factor increased as the loading rate increased. An empirical equation based on an exponential regression analysis of the test data was developed to estimate the dynamic increase factor as a function of loading rate. The dynamic increase factor was multiplied by Equation (1) to calculate the mean dynamic nominal pullout capacity ($N_{n,d}$) as seen in Equation (16). The last factor in this equation is the dynamic increase factor which is the ratio of the dynamic loading rate ($\dot{p}$) to the static reference loading rate ($\dot{p}_s$) raised to the 0.013 power.

$$N_{n,d} = \tau_0 \pi d_0 h_{ef} \left( \frac{\dot{p}}{\dot{p}_s} \right)^{0.013}$$  \hspace{1cm} (16)

The most common failure mode during testing was adhesive bond stress failure combined with the formation of a concrete cone. The test results indicated that the dynamic pullout strengths were closely related to the calculated values based on the
ultimate uniform bond strength. It was observed that the cone failure was fully developed before the bond failure occurred; therefore the capacity was most commonly controlled by the bond strength of the anchor below the cone failure. This agrees with the theory proposed by Cook, Doerr, and Klingner [10] but under dynamic conditions instead of static conditions. It was noted that the behavior of chemically bonded anchors under dynamic loading is strongly dependent on the particular bonding agent.

In 2005, Solomos and Berra utilized a Hopkinson bar technique to determine the effect of dynamic loading rates on post-installed anchors [16]. The static and dynamic test results were compared to the values predicted by the design codes of ACI 349-97 and the CCD method. For static loading conditions, the experimental capacities were always higher than the predicted ones, especially compared to ACI 349-97. The capacities under dynamic loading conditions were substantially higher as the experimental capacities were between 1.59 and 2.39 times as high as the predicted values for static conditions. A dynamic increase factor of 1.25 is permitted to increase the axial concrete strength for impact loads according to ACI 349-97. It was concluded that this dynamic increase factor was reasonable for chemical adhesive anchors. However, the dynamic increase factor of concrete in tension could be as high as 3 or 4 for very high strain rates.

In 2009, Braimah, Constestabile, and Guilbeault conducted several “mass drop” tests on epoxy adhesive anchors and compared the dynamic capacities to results obtained from a static test program [17]. It was concluded that the dynamic capacity of adhesive anchors could be increased by minimum factors of 1.2 and 3.2 for normal loads and loads applied at a 45 degree angle, respectively, compared to static capacities.

2.5 Material Properties of Structural Epoxy Adhesives
Kruger and Lin conducted several tests to determine the material properties of two different types of epoxy adhesives [18]. Both products used in the tests were two-part cold cure epoxy adhesives in which one epoxy consisted of an unfilled resin and the other a heavily filled resin with a highly dispersed, amorphous, pure silicon filler.

For each material, the tensile strength, compressive strength, Young’s Modulus, shear strength, shear modulus, Poisson’s ratio, and elongation at break were determined. The material properties of the two adhesives are shown in Table 1. Several tests were conducted to determine the effects of curing time and temperature on the ultimate bond strength. It was observed that the bond strength of the adhesive was significantly reduced when subjected to moisture. For a hardened concrete to hardened concrete bond, the strength could be reduced by as much as 20 to 50 percent by the effects of moisture. From creep tests, it was concluded that cured epoxy adhesives have low creep strain values compared to other structural adhesives. However, the creep resistance is greatly reduced as the material approaches the heat deflection temperature.

Table 1. Material Properties of Hardened Epoxies

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Unfilled Epoxy Resin</th>
<th>Heavily Filled Resin with Reinforcing Filler</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength</td>
<td>4,950 psi (34.1 MPa)</td>
<td>3,090 psi (21.3 MPa)</td>
</tr>
<tr>
<td>Elongation at Break</td>
<td>4.82 %</td>
<td>4.69 %</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>11,200 psi (77.3 MPa)</td>
<td>10,100 psi (69.8 MPa)</td>
</tr>
<tr>
<td>Young’s Modulus</td>
<td>464 ksi (3.2 GPa)</td>
<td>609 ksi (4.2 GPa)</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>&gt; 5,800 psi (&gt; 40 MPa)</td>
<td>5,800 psi (40 MPa)</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>174 ksi (1.2 GPa)</td>
<td>218 ksi (1.5 GPa)</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.39</td>
<td>0.37</td>
</tr>
<tr>
<td>Heat Deflection Temperature</td>
<td>124 °F (51.0 °C)</td>
<td>127 °F (52.5 °C)</td>
</tr>
</tbody>
</table>
The material properties for Hilti, Adhesives Technologies, and Simpson epoxy systems were obtained from a review of the manufacturers’ specifications [19-23]. The material properties for several epoxy products are shown in Table 2. Summary tables of static tensile and shear capacities for various epoxy anchor products are shown in Appendix A [19-26].

Table 2. Material Properties Obtained from Epoxy Manufacturers

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Hilti HIT-RE 500</th>
<th>Adhesives Technology HS2000</th>
<th>Adhesives Technology Ultrabond 1</th>
<th>Adhesives Technology Ultrabond 3</th>
<th>Simpson ET</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bond Strength</td>
<td>1,800 psi (12.4 MPa)</td>
<td>2,400 psi (16.5 MPa)</td>
<td>1,640 psi (11.3 MPa)</td>
<td>1,960 psi (13.5 MPa)</td>
<td>2,030 psi (14.0 MPa)</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>12,000 psi (82.7 MPa)</td>
<td>15,260 psi (105 MPa)</td>
<td>10,990 psi (75.8 MPa)</td>
<td>10,110 psi (69.7 MPa)</td>
<td>13,390 psi (92.3 MPa)</td>
</tr>
<tr>
<td>Compressive Modulus</td>
<td>220 ksi (1.52 GPa)</td>
<td>322 ksi (2.22 GPa)</td>
<td>214 ksi (1.48 GPa)</td>
<td>201 ksi (1.39 GPa)</td>
<td>658 ksi (4.54 GPa)</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>6,310 psi (43.5 MPa)</td>
<td>7,080 psi (48.8 MPa)</td>
<td>6,790 psi (46.8 MPa)</td>
<td>7,840 psi (54.1 MPa)</td>
<td>-</td>
</tr>
<tr>
<td>Elongation at Break</td>
<td>2.00%</td>
<td>1.50%</td>
<td>1.90%</td>
<td>1.60%</td>
<td>-</td>
</tr>
<tr>
<td>Heat Deflection Temperature</td>
<td>146 °F (63.3 °C)</td>
<td>152 °F (66.7 °C)</td>
<td>134 °F (56.7 °C)</td>
<td>138 °F (58.9 °C)</td>
<td>168 °F (75.6 °C)</td>
</tr>
</tbody>
</table>

2.6 Effects of Protective Coatings on Steel Anchors

Galvanized or epoxy-coated reinforcement is commonly used on bridge projects to deter the effects of corrosion, and many Midwest Pooled Fund States require bridge rail reinforcement to be epoxy-coated. Particularly in northern states, where salt is used in the winter months to combat snow and ice, corrosion is a major concern. Unfortunately, very little information on the bond strength of epoxy-coated anchors bonded to concrete by the use of an epoxy adhesive was available. However, several sources of epoxy-coated
bars bonded to concrete were used to investigate the effects that epoxy coatings have on bond strength.

Yeomans investigated the performance of galvanized and epoxy-coated reinforcing bars embedded in concrete cylinders and exposed to an accelerated corrosion test program [27]. This consisted of two different methods: repetitive wetting and drying of the specimens in a salt bath, and exposing the specimens in a salt fog chamber. The results of the corrosion tests indicated that the galvanized finish significantly delayed the onset of corrosion compared to uncoated black steel, and the epoxy coating effectively eliminated corrosion. However, for both the galvanized and epoxy-coated finishes, the coatings needed to be repaired at points where damage to the coatings occurred or else premature corrosion would occur.

In 1976, Clifton and Mathey conducted several pullout tests of coated deformed reinforcing bars embedded in concrete [28]. A universal electromechanical testing machine was used to apply a tensile load to the bars with a bearing surface closely surrounding the bars which prevented a concrete cone failure. Failure was determined by one of the following: a slip of 0.01 in. (0.25 mm) at the loaded end, a slip of 0.002 in. (0.05 mm) at the embedded end, or yielding of the steel bar. The study involved testing several different types of epoxy and polyvinylchloride coatings.

The polyvinylchloride coating bond strengths were considerably less than uncoated bars and were not recommended for structural use. However, epoxy-coated bars with the coating less than 10 mils (0.25 mm) thick provided bond strengths of only six percent less than the bond strength for uncoated bars and were considered suitable to develop the yield strength of the reinforcement.
In 1989, Jirsa and Treece conducted several tests to determine the effects of using epoxy-coated reinforcing bars for strength development compared to uncoated bars [29]. The tests consisted of using a 4-point bending beam with steel reinforcement placed in the tensile region of the beam and reinforcement splices in the middle of the beam. Load was applied to the beam until tensile cracks formed at the constant moment section of the beam. The bond strength was calculated based on the stress developed in the steel at the time of failure.

After each test, the concrete cover of the reinforcing steel was removed to observe the bond at failure. The uncoated bars showed evidence of good adhesion as concrete particles were firmly attached to the bars. Concrete in contact with the bars had a dull, rough surface, and there was crushing of the concrete due to bearing against the bar lugs. Conversely, the epoxy-coated bars had a smooth glassy surface, and there were no signs that the concrete was crushed against the bar deformations.

The bond strength between the reinforcing bars and the concrete was reduced by 35 percent when the reinforcing bars were coated with epoxy. This reduction in bond strength did not vary with the concrete strength. Design recommendations were proposed which stated that the required development length should be multiplied by 1.5 for epoxy-coated bars with concrete cover less than 3 times the bar diameter or clear spacing less than 6 times the bar diameter. For all other cases of epoxy-coated bars, the required development length should be multiplied by 1.15; however the product of the combining factor for top reinforcement and the epoxy-coated reinforcement factor should never exceed 1.7. These coating factors were later adopted by ACI committee 318 in the *Building Code Requirements for Structural Concrete*. In a literature review, it was noted
that the 6 percent decrease in bond strength from the testing program by Clifton and Mathey did not represent the ultimate bond strength because of the criteria used to categorize failure.

A failure hypothesis explained that two forces, bearing and friction, act on the ribs of the bar. For epoxy-coated bars, the friction component was nearly lost resulting in a reduced bond strength. It was suggested that if the face of the rib formed a 90 degree angle with the axis of the bar, all the bond strength would be produced by direct bearing, and friction would be unnecessary.

The effects of protective coatings on the tensile capacities of reinforcing bars embedded in concrete were studied by Yeomans in 1991 [27]. The results of the pullout tests indicated that there was not a significant difference in the bond strength between black, galvanized, and epoxy-coated deformed reinforcing bars. However, for straight, non-deformed segments, there was a 17 percent decrease in the bond strength of epoxy-coated reinforcement and a 31 percent increase in the bond strength of galvanized reinforcement compared to plain black steel reinforcement.

Also in 1991, Cleary and Ramirez conducted 4-point bending slab tests (similar to Jirsa and Treece) to study the effects epoxy coating had on the bond strength to concrete [30]. Independent tests were performed for 4 different splice lengths for both coated and uncoated reinforcement. The epoxy coating contributed to reductions of 15 and 5 percent for specimens where the steel did not yield. For the two other test pairs, the steel in the uncoated specimens yielded and the test data was considered not as useful as the other tests, but indicated a strength reduction of at least 15 percent.
In 1992, Cusens and Yu published an article that summarized the findings of pullout tests for three different types of deformed steel reinforcing bars and studied how epoxy coatings affected the bond strength to concrete [31]. The critical bond stress was determined for each test, which corresponded to the lower bond stress value obtained from either the free or the loaded end of the reinforcing bar. The epoxy coating contributed to reductions of 56, 22, and 14 percent for the three different types and sizes of reinforcement. The 56 percent reduction corresponded to a reinforcing bar with significantly smaller deformation rib height and spacing compared to the other two samples. Therefore, a conclusion was made that larger and more closely spaced deformation patterns are required to provide satisfactory bond strengths with epoxy-coated reinforcing bars.

2.7 Creep Effects of Epoxy Anchors

Tests to examine the effect of sustained long-term loads are contained in several documents. The ICC-ES report AC58 was published in 1995 and was superseded by AC308 in 2007. These documents are used by manufacturers to qualify their adhesive anchor products. In AC58, creep testing of adhesive anchors was optional while in AC308, creep testing is mandatory [3]. Creep tests are conducted in uncracked concrete at standard and maximum temperature conditions. The anchor is loaded to 55 percent of its mean ultimate load multiplied by a factor based on concrete strength and the load is sustained for 42 days. Then a confined tension test to failure is conducted on the anchor following the sustained load test. The anchor must have at least 90 percent of its tension capacity after the sustained load test to pass creep test criteria.
The ACI 355.4-10 report (currently in draft form) will replace the AC308 report and contains only minor changes to the creep testing criteria. However, all anchors must be approved for creep in ACI 355.4-10. Therefore, all qualified products are required to pass creep test criteria. The strength of sustained tensile loaded adhesive anchors is also addressed in ACI 318-11. The nominal capacity of an adhesive anchor subject to sustained tensile loads can only be taken as 55 percent the nominal strength of the anchor [32].

2.8 Anchorage Used for Temporary Concrete Barriers

2.8.1 F-Shape Tie-Down with Drop-In Anchors

MwRSF developed a steel strap tie-down system for the Iowa F-shaped temporary concrete barrier in 2002 [33, 34]. The goal of the project was to develop a tie-down system that could constrain and limit barrier deflection and rotation during an impact event that did not utilize epoxied anchor studs or complete drilling through the bridge deck. The design consisted of a steel strap that attached to the connecting pin of adjacent barriers and utilized two 3/4 in. (19 mm) diameter by 3 3/8 in. (81 mm) long Red Head drop-in anchors [35]. The actual outside diameter of the sleeve of the drop in anchor was slightly larger than the nominal diameter of the 3/4 in. (19 mm) diameter by 1 3/4 in. (44 mm) long ISO class 8.8 bolts. The results of the crash test showed that a total of 4 anchor bolts (located near the impact location) were pulled completely out of the concrete, but all the remaining bolts were effectively anchored to the concrete decking. The addition of the tie-down strap limited the dynamic deflection to 37.80 in. (0.96 m) as compared to 45.28 in. (1.15 m) in previous testing of the F-shape portable concrete barrier in a free-
standing configuration [33, 34]. A picture of the steel strap tie-down design is shown in Figure 5.

![Figure 5. Temporary Concrete Barrier Steel Strap Tie-Down](image)

### 2.8.2 F-Shape Tie-Down with Screw-In Anchors

An alternative design for anchorage of the steel strap tie-down design was developed by MwRSF in 2007 [36]. Dynamic shear and tensile tests were conducted on the ¾ in. (19 mm) diameter Red Head drop-in anchor used for the original steel strap development in 2002. The average peak tensile load for this anchor was found to be 18.7 kips (83.2 kN) and the average peak shear load was found to be 25.6 kips (113.9 kN). It was desired to replace the drop-in anchor with a screw-in anchor that would be easier to install and remove. Several screw-in anchors were tested and two observations were made: (1) the screw-in anchors generally had higher tensile strengths due to their slightly
longer embedment depth compared to the drop-in anchor and (2) the screw-in anchors did not perform as well with regards to the shear testing due to smaller diameter anchors and lower grade steel. It was suggested that any alternative anchors needed to meet a peak tensile load of 18.7 kips (83.2 kN) and a peak shear load of 25.6 kips (113.9 kN) in order to be considered an acceptable retrofit for the 3/4 in. (19 mm) Red Head drop-in anchor. Two alternatives were identified: (1) the Red Head Large Diameter Tapcon (LDT) 3/4 in. (19 mm) diameter by 4 1/2 in. (114 mm) long which had ultimate tensile and shear strengths of 19.5 kips (86.7 kN) and 26.0 kips (115.7 kN), respectively, and (2) the Simpson Titen HD 3/4 in. (19 mm) diameter by 5 in. (127 mm) long anchor which had tensile and shear strengths of 19.0 kips (84.5 kN) and 34.3 kips (152.6 kN), respectively.

2.8.3 F-Shape Tie-Down with Three A307 Steel Anchors

In 2003, MwRSF developed a tie-down system for use on reinforced concrete bridge decks with a redesigned F-shape temporary concrete barrier [37]. This design consisted of bolting through the F-shape barrier at 3 locations along the impact side of the barrier. The threaded rods were made from ASTM A307 steel and had an anchor diameter of 1 1/8 in. (29 mm). The design was successfully crash tested with the anchors attached to the deck by Power Fasteners Power-Fast Epoxy with an embedment depth of 12 in. (305 mm). An alternate anchorage procedure that was considered acceptable was to run the bolt entirely through the bridge deck and use a nut and washer (bearing plate) on the bottom of the bridge deck. The anchorage was required to develop the full bolt capacity. A picture of the barrier and bolting pattern is shown in Figure 6. The Wisconsin Department of Transportation adopted this temporary concrete barrier anchorage design and required the epoxy anchors to develop the bolt capacity [38].
2.8.4 Steel H-Section Temporary Barrier

In 2003, MwRSF developed tie down system for a steel H-section temporary barrier that was originally developed in 1989 by MwRSF in order to limit the dynamic deflection of the barrier [33, 39]. The original design consisted of placing two ¾ in. (19 mm) Red Head drop-in anchors with ¾ in. (19 mm) ASTM A325 bolts that were 1 3/4 in. (44 mm) long at each end of the 20-ft (6.10-m) long barrier segments on the impact side. Upon failure of the initial crash test with a 4,478-lb (2,031 kg) pickup truck due to vehicle rollover, the anchor bolts were changed from ASTM A325 to ASTM A307 grade bolts to reduce the load capacity of the tie-down attachments and allow a slight increase in the deflection of the system. The modifications were also implemented to reduce vehicle snag. The modified system was successfully crash tested and four of the anchor
bolts failed by shear fracture while one anchor bolt failed by tensile pullout. A picture of the anchorage of the H-section temporary barrier is shown in Figure 7.

Figure 7. Steel H-Section Temporary Barrier Anchors

### 2.9 Anchors Used in Bridge Rail Retrofit Applications

#### 2.9.1 California Type 25 Concrete Barrier with Adhesive Anchors

In 1979, the California Department of Transportation (CALTRANS) conducted research on utilizing grouted deformed reinforcing bars to attach a new California Type 25 Concrete Barrier to an existing bridge deck [40]. A series of dynamic pullout tests and static barrier tests were performed to evaluate different types of cement and epoxy-mortar grouts. The preferred adhesive material was the Type II Portland Cement Grout because of its superior strength and low cost. However, the average dynamic pullout strength of specimens with an epoxy-mortar was found to be equivalent to the Type II Portland
Cement Grout. The anchors were tested with no. 5 and 6 (metric no. 16 and 19) reinforcing bars with embedment depths of 5 and 6 in. (127 and 152 mm).

The dynamic pullout test results of specimens with 5 in. (127 mm) embedment were somewhat erratic and inconsistent while specimens with 6 in. (152 mm) embedment yielded results that were more consistent and followed a more logical pattern. The capacity of specimens with 6 in. (152 mm) embedment depths had capacities approximately 40 percent higher than specimens with 5 in. (127 mm) embedment depths.

The conventional anchorage design for the barrier studied utilized cast-in-place no. 5 (metric no. 16) reinforcing bars with hooks on the embedded ends spaced 15 in. (381 mm) apart on the traffic side of the barrier. A no. 5 (metric no. 16) cast-in-place dowel bar spaced 30 in. (762 mm) apart was utilized on the back side of the railing. A 3-ft (0.91-m) section of the conventional design was constructed and tested by applying a static load to the top of the barrier. This section of the barrier was found capable of sustaining a load of 28.7 kips (128 kN) before failure.

Two retrofit designs were tested using Type II Portland Cement grouted anchors with embedment depths of 5 and 6 in. (127 and 152 mm). The 5-in. (128-mm) embedment design utilized no. 6 (metric no. 19) dowels spaced at 11 in. (279 mm) on the traffic side and 30 in. (762 mm) on the back side of the railing. The 6-in. (152-mm) embedment design utilized no. 6 (metric no. 19) dowels spaced at 15 in. (381 mm) on the traffic side and 30 in. (762 mm) on the back side of the railing. A 3-ft (0.91-m) section of each design was tested in a similar manner as the conventional design and the ultimate horizontal loads applied at the top of the barriers were 34.3 kips (153 kN) and 41.2 kips.
(183 kN) for the 5-in. (127-mm) and 6-in. (152-mm) embedment designs, respectively. Both of these developed designs had a higher capacity than the conventional design.

The final retrofit design consisted of using no. 6 (metric no. 19) grouted dowels with embedment depths of 6 in. (152 mm) spaced at 15 in. (381 mm) on the traffic side. The back side of the railing called for no. 6 (metric no. 19) grouted dowels with embedment depths of 5 in. (127 mm) spaced at 30 in. (762 mm). It was suggested that whenever possible, the anchors along the traffic side of the barrier should always have a 6 in. (152 mm) embedment depth. However, in special cases where embedment depths of 6 in. (152 mm) is not possible, slightly less embedment depths should be allowed. Embedment depths less than 5 in. (127 mm) should not be allowed in any case.

2.9.2 UT-Austin Impact Tests on New Jersey Bridge Rails

In 1985, the Center for Transportation Research at the University of Texas at Austin conducted static and dynamic tests that used ASTM A36 anchor bolts to attach cast-in-place and precast New Jersey bridge rails to a standard Texas bridge deck [41]. The goal of the research was to develop an anchorage design that exhibited a ductile failure mode. The original anchorage design utilized 1-in. (25-mm) diameter, ASTM A193 Grade B7 anchor bolts spaced at 50 in. (1,270 mm) that were attached with washers and nuts on the underside of the slab. It was believed that a ductile failure mode could be achieved by using 1-in. (25-mm) diameter, ASTM A36 anchor bolts spaced at 25 in. (635 mm) that were attached with nuts on the underside of the slab. The lower strength steel was used to lower the ultimate strength of the anchors and allow a much longer yield plateau to increase the amount of energy absorbed. Anchor spacings of 50 and 75 in. (1,270 and 1,905 mm) were also tested.
The testing program consisted three static tests and one impact test that used a hydraulic ram to apply a force to the top of the 12-ft 6-in. (3.81-m) long barriers. The impact test consisted of applying sets of three impulse loads which started at low magnitudes, but were gradually increased until failure.

Even though the barrier was heavily reinforced beyond the normal design to prevent a brittle failure, a brittle failure of the concrete still occurred before rupture of the steel anchors. It was noted that the anchors resisted a portion of the shear force at the barrier/deck interface, but for design purposes most of the shear was assumed to be resisted by the frictional force between the barrier and the slab. This assumption was confirmed to be correct from the tests as there was no evidence of shear distress in any of the anchor bolts.

For the impact tests, a series of three loads were applied at each load level. For the first impulse of each set, the barrier experienced additional damage. However, the additional two impulses at each load level did not cause additional degradation of the barrier or slab. The anchorage design was considered to lack the required ductility because the anchorage was too strong, which lead to brittle failures of the railing.

### 2.9.3 MwRSF Crash Tests with Adhesive Anchors

In 1991, MwRSF conducted three crash tests on a modified New Jersey bridge railing with a small car, a pickup truck, and a single unit truck [42]. The bridge railing was attached to a concrete slab-on-ground by two no. 5 (metric no. 16) reinforcing bars spaced at 12 in. (305 mm) that were embedded 8 in. (203 mm) into the concrete slab. An epoxy grout was used as the bonding agent. Reinforcing bars were placed near the traffic side and the back side of the barrier, and the distance between the bars was approximately
10 ½ in. (267 mm). Although the primary purpose of the study was not to design the anchorage for the bridge rail, it was observed that the anchorage design was adequate to sustain the loads applied by a 1,759 lb (798 kg) car travelling at a speed of 62.5 mph (100.6 km/h) and at an impact angle of 20 degrees, a 5,460 lb (2,477 kg) pickup truck travelling at a speed of 63.5 mph (102.2 km/h) and at an impact angle of 20 degrees, and an 18,111 lb (8,215 kg) single unit truck travelling at a speed of 52.5 mph (84.5 km/h) and at an impact angle of 16.1 degrees. No visible lateral movement of the bridge rail occurred in any of the crash three tests.

**2.9.4 MDOT Analysis of Railings with Adhesive Anchors**

In 2001, the Michigan Department of Transportation investigated the effectiveness of using adhesive anchors to retrofit concrete bridge railing attachments to bridge decks [43]. The overall barrier redirective strength was calculated using the AASHTO LRFD Bridge Design Specifications, and the maximum tensile strength of the anchorage was also calculated. The original anchorage design consisted of no. 5 (metric no. 16) grade 60 steel reinforcement spaced at 12 in. (305 mm) with an embedment depth equal to 7 ½ in. (191 mm). It was suggested to revise the design to no. 4 (metric no. 13) grade 60 steel reinforcing bars spaced at 8 in. (203 mm) with a shorter embedment depth of 6 in. (152 mm) to decrease the chance of cracking concrete on the bottom of the bridge deck when drilling. It was noted in the literature review that the bond stress at the concrete-epoxy interface for impact loading was found to be 150 percent greater than that of static loading, and the effect of winter temperatures had no effect on the dynamic bond strength of the anchors tested.
2.9.5 SUT (10000S) Vehicle Crash Test with New Jersey Barrier

In 2006, MwRSF conducted a crash test with a 10000S Single Unit Truck (SUT) vehicle in order to assess the effects of the proposed update the NCHRP Report No. 350 [44]. The permanent reinforced concrete New Jersey safety shape barrier was 32 in. (813 mm) tall and was attached to a concrete slab-on-ground by two no. 5 (metric no. 16) reinforcing bars spaced at 8 in. (203 mm) that were embedded 10 in. (254 mm) into the concrete slab. The Fast Set Formula Power-Fast High Strength Adhesive Epoxy was used as the bonding agent. The reinforcing bars were placed near the traffic side and back side of the barrier, and the distance between the bars was approximately 11 3/8 in. (289 mm). The 22,045 lb (9,999 kg) SUT impacted the barrier travelling at a speed of 56.5 mph (90.9 km/h) and at an angle of 16.2 degrees. There was no visible lateral movement of the bridge or the bridge rail anchorage due to the impact. Therefore, the anchorage size and spacing was adequate to withstand the impact, but the crash test failed the safety performance criteria found in the Update to NCHRP Report No. 350 due to the vehicle rolling over the top of the barrier. A cross section of the barrier and reinforcement in shown in Figure 8.
Figure 8. New Jersey Barrier Used in Crash Test with 10000S SUT

2.9.6 Texas T501 and T203 Railings Modified for use with Epoxy Anchors

The Texas Transportation Institute (TTI) conducted research on using epoxy anchors to attach two different types of bridge rails to a standard bridge deck in 2007 [45]. TTI first evaluated the Texas T501 New Jersey-shaped barrier and the Texas T203 open concrete railing with conventional cast-in-place anchoring. The New Jersey-shaped bridge rail design consisted of a 32-in. (813-mm) tall barrier that was continuously attached to the bridge deck with no. 5 (metric no. 16) U-shaped reinforcing bars spaced at 8 in. (203 mm). The open concrete bridge rail was a 27-in. (686-mm) tall railing that was attached to the bridge deck by 5-ft (1.5 m) wide posts that were spaced between 5-ft (1.5 m) long openings. The conventional anchoring for the open concrete bridge rail consisted of no. 4 (metric no. 13) U-shaped reinforcing bars spaced at 5 in. (127 mm).
Both static and dynamic tests were conducted for each bridge rail with strain gauges mounted on the reinforcing bars that experienced tensile forces. The static test utilized a hydraulic ram to apply a load to the top of the barriers over a bearing length of 3 ft – 6 in. (1.1 m). A rigid frame bogie with a 3-ft 6-in. (1.1-m) wide crushable nose was used for the dynamic tests.

After observing the tensile forces in the reinforcing bars with conventional anchoring, TTI developed a retrofit design for anchoring the bridge rails to the bridge deck using epoxy anchors. The bonding agent used for all designs in this report was the Hilti HIT-RE 500 epoxy adhesive. For the New Jersey bridge rail, a single no. 6 (metric no. 19) reinforcing bar with an embedment depth of 5 ¼ in. (133 mm) spaced at 16 in. (406 mm) at the mid-span and 8 in. (203 mm) near the end was used to develop anchorage to the bridge deck. The open concrete bridge rail utilized two rows of no. 5 (metric no. 16) reinforcing bars with embedment depths of 5 ¼ in. (133 mm) spaced at 8 in. (203 mm) on the traffic side and 14 in. (355 mm) on the back side of the bridge rail for the middle posts. The end post section utilized two rows of no. 5 (metric no. 16) reinforcing bars with embedment depths of 5 ¼ in. (133 mm) spaced at 6 ½ in. (165 mm) on the traffic side and 13 in. (330 mm) on the back side. The developed designs were tested with bogie crash tests which proved the design to be adequate. A detail of the modified Texas T501 bridge railing with epoxy anchors is shown in Figure 9 for the mid-span case.
2.10 Load Distributions for Vehicular Bridge Rails

In 2006, the Center for Transportation Research at the University of Texas at Austin analyzed the static and dynamic load distributions that occurred when a lateral load was applied to an open concrete bridge railing and a continuous bridge railing with conventional cast-in-place anchoring [46]. The open concrete rail analyzed was the Texas T203 concrete barrier that consisted of a 14-in. by 13 ½-in. (356-mm by 343-mm) concrete railing that was supported by 5-ft (1.52-m) wide by 7 ½-in. (191-mm) thick posts spaced 10 ft (3.1 m) apart. The continuous railing analyzed was the Texas T501 concrete barrier, which was a 32-in. (813-mm) tall New Jersey bridge rail. These railings have been crash tested to both TL-3 and TL-4 standards as defined by the National Highway Cooperative Research Program (NCHRP) Report No. 350.
The peak dynamic, 50-ms average dynamic, and static capacities were obtained from testing done by TTI in 2002 [46]. The static and dynamic structural analysis program SAP was used to determine the amount of the barrier capacity that was carried by the overturning capacity of the barrier (strength where the loads were transferred vertically to the bridge deck beneath the location of the applied load) and the continuity of the barrier (strength where the loads were transferred longitudinally along the length of the barrier). The findings of the barrier capacities and the proportion of the capacities carried by the stand alone strength of the barrier and anchorage versus the continuity of the barrier are summarized in Table 3.

For the open concrete rail design, approximately half the capacity was carried by both the overturning capacity of the posts, while the continuity of the barrier accounted for the other half of the capacity for static load conditions. As the loading rate increased, approximately 10 percent more of the capacity was carried by the continuity of the barrier rather than the overturning capacity and the anchorage beneath the applied load. It was suggested that this barrier needed to withstand a 50-ms average lateral dynamic load of 60 kips (267 kN) and a lateral static load of 54 kips (240 kN) to meet the design requirements of the crash tested barrier. The anchorage capacity of the barrier needs to resist the overturning force.

For the continuous New Jersey barrier, approximately 3 percent more of the capacity was carried by the overturning capacity than the continuity of the barrier under static loads relative to the 50-ms average dynamic load. Results from the dynamic analysis were conflicting as the stand alone capacity carried slightly more of the load when considering the 50-ms average, but slightly less of the capacity when considering
the peak dynamic capacity compared to the capacity of the barrier carried by the continuity of the barrier.

Based on the findings from the Center for Transportation Research, approximately 50 percent of the applied lateral loads to bridge barriers are transferred to the anchorage beneath the applied load while the other 50 percent is distributed throughout the longitudinal length of the barrier. Also based on the testing and analysis of the barrier sections, it was observed that the barriers and slab remained essentially elastic throughout the impact. The dynamic increase factor for bridge barriers was proposed to be between 1.2 and 1.6.

Table 3. Load Distributions for the Texas T203 and T501 Concrete Railings

<table>
<thead>
<tr>
<th></th>
<th>Capacity Carried by the Overturning Resistance of the Anchorage</th>
<th>Capacity Carried by the Continuity of the Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Open Concrete Railing (Texas T203)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peak Dynamic Capacity</td>
<td>160 kips (712 kN)</td>
<td>30% 50 kips (222 kN)</td>
</tr>
<tr>
<td>50-ms Average Dynamic Capacity</td>
<td>68 kips (302 kN)</td>
<td>40% 27 kips (120 kN)</td>
</tr>
<tr>
<td>Static Capacity</td>
<td>72 kips (320 kN)</td>
<td>50% 36 kips (160 kN)</td>
</tr>
<tr>
<td><strong>New Jersey Barrier (Texas T501)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peak Dynamic Capacity</td>
<td>155 kips (689 kN)</td>
<td>40% 60 kips (267 kN)</td>
</tr>
<tr>
<td>50-ms Average Dynamic Capacity</td>
<td>68 kips (302 kN)</td>
<td>54% 37 kips (165 kN)</td>
</tr>
<tr>
<td>Static Capacity</td>
<td>70 kips (311 kN)</td>
<td>57% 40 kips (178 kN)</td>
</tr>
</tbody>
</table>
2.11 Bridge Railing Design Load Background

2.11.1 AASHTO Standard Specifications for Highway Bridges

The AASHTO Standard Specifications for Highway Bridges has provided guidance for the design loads that bridge railings need to resist. Before 1965, bridge railings were required to resist a lateral horizontal force of 0.150 kips/ft (2.19 kN/m) and a vertical force of 0.100 kips/ft (1.46 kN/m) applied to the top of the railing [76]. The railing was required to have a minimum height of 27 in. (686 mm) and a maximum height of 42 in. (1,067 mm).

In 1962, because of poor accident history, the U.S. Department of Commerce, Bureau of Public Roads (BPR), which is now the Federal Highway Administration, proposed that bridge railings needed to resist a transverse load of 30 kips (133.5 kN) using plastic design procedures [76].

In 1965, AASHTO adopted the requirement that bridge railings needed to resist a transverse load of 10 kips (44.5 kN) using elastic, allowable stress design procedures. This load was to be applied as a concentrated load at the mid-span of railing panels and distributed over a longitudinal length of 5 ft (1.52 m) for parapet walls. The minimum height of the railing was required to be 27 in. (686 mm). It can be shown that the 10 kip (44.5 kN) load as determined by elastic analysis is approximately equal to a 30 kip (133.4 kN) load calculated by plastic analysis [76]. It is possible that the elastic design procedure was ultimately adopted because many of the AASHTO members were unfamiliar with plastic design procedures. This 10 kip (44.5 kN) load requirement essentially remained the same for the remaining releases of the AASHTO Standard Specifications for Highway Bridges.
2.11.2 AASHTO LRFD Bridge Design Specifications

In recent years, the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications has replaced the AASHTO Standard Specifications for Highway Bridges, and as of 2007, only the LRFD code has been allowed for new designs. Included in the AASHTO LRFD Bridge Design Specifications is an ultimate bridge rail design method based on yield line theory. Yield line design is an ultimate strength, plastic design procedure that is based on the principle that the internal energy absorbed by deformation equals the external work from the applied forces and deflections [47]. The ultimate capacity calculated from the yield line analysis must be greater than the load imparted to the railing from the vehicle to ensure the adequacy of the bridge railing.

One of the key steps in determining the ultimate capacity using yield line theory is correctly predicting the yield line pattern. Yield line patterns are estimated configurations of the plastic hinges that form in two dimensional members such as panels, walls, floors, and slabs. Often times in loaded concrete walls and slabs, yield lines are visible as crack patterns. Theoretically, several yield line patterns could occur in a structure, but one configuration will provide the lowest failure load, known as the yield line solution. However, an investigation of only a few simple and obvious patterns is needed because the solutions of these patterns are usually within a few percent of the correct solution [47].

In the AASHTO LRFD Bridge Design Specifications, the yield line patterns have already been derived as well as simple, user-friendly equations that coincide with the yield line pattern [48]. The bridge railing needs to be analyzed at both the middle of the railing and at the ends to account for interior and end vehicular impacts respectively. The
yield line patterns for an interior and end region of a continuous parapet are shown in Figure 10. As can be seen, the interior regions consist of three cracks, or yield lines, while the end region only contains only one yield line.

![Diagram of Interior Region and End Region](image)

**Figure 10. Yield Line Patterns for Continuous Bridge Railing [48]**

Based on the yield line patterns shown in Figure 10, the AASHTO LRFD Bridge Design Manual provides equations for the nominal railing resistance to transverse load applied at the top of the wall ($R_w$). This is a function of the critical length of the yield line pattern failure ($L_c$), the longitudinal length of distribution of impact forces ($L_t$), the flexural resistance of the cantilevered walls about an axis parallel to the longitudinal axis of the bridge ($M_c$), the flexural resistance of the wall about its vertical axis ($M_w$), the additional flexural resistance of a beam in addition to $M_w$ ($M_b$), and the height of the wall ($H$). The equations for the nominal railing resistance to transverse load for the interior and end regions of a railing are shown in Equations (17) and (19), respectively.
For Interior Regions:

\[ R_w = \left( \frac{2}{2L_c - L_t} \right) \left( 8M_b + 8M_w + \frac{M_c L_c^2}{H} \right) \]  

(17)

\[ L_c = \frac{L_t}{2} + \sqrt{\left( \frac{L_t}{2} \right)^2 + \frac{8H(M_b + M_w)}{M_c}} \]  

(18)

For End Regions:

\[ R_w = \left( \frac{2}{2L_c - L_t} \right) \left( M_b + M_w + \frac{M_c L_c^2}{H} \right) \]  

(19)

\[ L_c = \frac{L_t}{2} + \sqrt{\left( \frac{L_t}{2} \right)^2 + H \left( \frac{M_b + M_w}{M_c} \right)} \]  

(20)

Table A13.2-1 in the 2007 AASHTO LRFD Bridge Design Manual also provides required design forces and geometric parameters for the yield line analysis procedure. These values are shown in Table 4. The parameters correspond to test level conditions consistent with NCHRP Report No. 350. The load levels were determined from full-scale instrumented wall crash tests to measure the forces imparted to “rigid” barriers. These instrumented wall tests consisted of four relatively rigid concrete wall panels that were supported laterally by load cells to measure the impact force magnitude and location. The panels were instrumented with accelerometers to account for inertial effects. The force data was processed by averaging the data over 50 millisecond intervals [49]. Therefore, the transverse force \( F_t \) is the ultimate lateral 50 ms average dynamic load required to resist the impact.
Table 4. AASHTO Design Forces and Geometric Properties [48]

<table>
<thead>
<tr>
<th>Design Forces and Designations</th>
<th>Railing Test Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TL-1</td>
</tr>
<tr>
<td>$F_t$, Transverse, kips (kN)</td>
<td>13.5 (60)</td>
</tr>
<tr>
<td>$F_L$, Longitudinal, kips (kN)</td>
<td>4.5 (20)</td>
</tr>
<tr>
<td>$F_v$, Vertical Down, kips (kN)</td>
<td>4.5 (20)</td>
</tr>
<tr>
<td>$L_t$ and $L_L$, ft (m)</td>
<td>4 (1.22)</td>
</tr>
<tr>
<td>$L_v$, ft (m)</td>
<td>18 (5.49)</td>
</tr>
<tr>
<td>Min $H_e$, in. (mm)</td>
<td>18 (457)</td>
</tr>
<tr>
<td>Min $H$, in. (mm)</td>
<td>27 (686)</td>
</tr>
</tbody>
</table>

2.12 State Standard Bridge Rail Designs

In order to develop an anchorage design procedure that would enable the new and retrofit barriers to behave similarly to the barriers with conventional cast-in reinforcing bars, the standard bridge rail plans for several Midwest Pooled Fund States were reviewed. Tables 5 and 6 summarize the sizes, shapes, and anchor spacings for several state standard bridge railings [50-70]. The anchorage designs were similar for all continuously attached barriers. Vertical, New Jersey, and F-shaped barriers utilized stirrups that consisted of either no. 4, 5, or 6 (metric no. 13, 16, or 19) reinforcing bars spaced between 8 and 12 in. (203 and 305 mm) for barriers of heights between 20 and 51 in. (0.51 and 1.30 m). No. 5 (metric no. 16) bars were the most commonly used bar size, and the equivalent steel area ranged from 0.62 to 0.93 square inches per foot of barrier (1,312 to 1,969 square millimeters per meter of barrier). The open concrete rail designs consisted of posts with no. 7 (metric no. 22) bars on the inside face and no. 4 (metric no. 13) bars on the outside face of the barrier. There was no uniform spacing design for the
open concrete rail due to the differing widths of the posts, but most posts had an equivalent steel area close to 2 square inches per foot of post (4,233 square millimeters per meter of post). Most reinforcing bars required some type of protective coating that consisted of either a galvanized finish or, most commonly, an epoxy coating. Bent hooks at the ends of the embedded bars were commonly used for anchorage.
### Table 5. State Standard Bridge Rail Summary (English Units)

<table>
<thead>
<tr>
<th>State</th>
<th>Bridge Rail Type</th>
<th>Anchor Rebar Size/Spacing</th>
<th>Steel Area Per Foot (in.²)</th>
<th>Barrier Height</th>
<th>Other Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>FL</td>
<td>Traffic Railing ‘F Shape - 32”</td>
<td>2-#5 @ 8” O.C.</td>
<td>0.93</td>
<td>2'-8”</td>
<td>See details for bar shapes, 6” min embedment depth</td>
</tr>
<tr>
<td>FL</td>
<td>Traffic Railing ‘F Shape - 42”</td>
<td>2-#5 @ 8” O.C.</td>
<td>0.93</td>
<td>3'-6”</td>
<td>See details for bar shapes, 5” min embedment depth</td>
</tr>
<tr>
<td>FL</td>
<td>Traffic Railing ‘F Shape Median</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>2'-8”</td>
<td>See details for bar shapes, 6” min embedment depth</td>
</tr>
<tr>
<td>FL</td>
<td>Traffic Railing Vertical Shape - 32”</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>2'-8”</td>
<td>Upside down U-stirrups with tail, 6” min embedment depth</td>
</tr>
<tr>
<td>FL</td>
<td>Traffic Railing Vertical Shape - 42”</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>3'-6”</td>
<td>Upside down U-stirrups with tail, 6” min embedment depth</td>
</tr>
<tr>
<td>FL</td>
<td>Traffic Railing - Corral Shape</td>
<td>12-#4 @ Post, 12-#7 @ Post, Posts @ 10’ O.C.</td>
<td>1.92</td>
<td>2'-8”</td>
<td>#7 bar=upside down U-stirrups with hook, #4 bar= L-shaped, 5’ wide posts, 6” min embedment depth</td>
</tr>
<tr>
<td>IA</td>
<td>Barrier Rail 2'-10”</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>2'-10”</td>
<td>Epoxy-coated rebars, U-stirrups with hooks, 2” min clear cover, F-shaped</td>
</tr>
<tr>
<td>IA</td>
<td>Barrier Rail 3'-8”</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>3'-8”</td>
<td>Epoxy-coated rebars, U-stirrups with hooks, for the first 4’ from abutments of bridge: 2-#5 @ 6” O.C., required steel=1.24 in²/ft, 2” min clear cover, F-shaped</td>
</tr>
<tr>
<td>IL</td>
<td>F Shaped Parapet</td>
<td>2-#5 @ 11” O.C.</td>
<td>0.68</td>
<td>2'-10” or 3'-6”</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hooks</td>
</tr>
<tr>
<td>KS</td>
<td>F4 Barrier Curb</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>2'-8”</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hooks, F-shaped</td>
</tr>
<tr>
<td>KS</td>
<td>Corral</td>
<td>8-#7 @ Post, 8-#4 @ Post, Posts @ 10’ O.C.</td>
<td>2.13</td>
<td>2’-3” or 2’-8”</td>
<td>Epoxy-coated rebars, L-shaped bars @ slab/rail interface, 3’ wide posts</td>
</tr>
<tr>
<td>MO</td>
<td>CIP Barrier Curb</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>2'-8”</td>
<td>Galvanized rebars, see details for bar shapes, NJ-shaped</td>
</tr>
<tr>
<td>MO</td>
<td>CIP Barrier Curb (Type D)</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>3’-6”</td>
<td>Galvanized rebars, see details for bar shapes</td>
</tr>
<tr>
<td>OH</td>
<td>Bridge Railing Deflector Parapet</td>
<td>2-#6 @ 12” O.C.</td>
<td>0.88</td>
<td>3’-0” or 3’-6”</td>
<td>See details for bar shapes, NJ-shaped</td>
</tr>
<tr>
<td>WI</td>
<td>Sloped Face Parapet ‘LF’</td>
<td>2-#5 @ 8” O.C.</td>
<td>0.93</td>
<td>2’-7 7/8”</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, preferred on state and interstate highway bridges, 2” min clear cover, F-shaped</td>
</tr>
<tr>
<td>WI</td>
<td>Sloped Face Parapet ’HF’</td>
<td>2-#5 @ 8” O.C.</td>
<td>0.93</td>
<td>3’-6 1/8”</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, used where there is a high truck traffic and curved horizontal alignment, 2” min clear cover, F-shaped</td>
</tr>
<tr>
<td>WI</td>
<td>Sloped Face Parapet ’51F’</td>
<td>2-#5 @ 8” O.C.</td>
<td>0.93</td>
<td>4’-3”</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, used in median area of adjacent structures, 2” min clear cover, F-shaped</td>
</tr>
<tr>
<td>WI</td>
<td>Sloped Face Parapet ’B’</td>
<td>2-#4 @ 9” O.C.</td>
<td>0.53</td>
<td>2’-8”</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, 2” min clear cover, NJ-shaped</td>
</tr>
<tr>
<td>WI</td>
<td>Vertical Face Parapet ’TX’</td>
<td>2-#5 @ 9” O.C.</td>
<td>0.83</td>
<td>3’-6”</td>
<td>Epoxy-coated rebars, upside down U-stirrups with hook, decorative raling with windows, 2” min clear cover</td>
</tr>
<tr>
<td>WI</td>
<td>Vertical Face Parapet ’A’</td>
<td>2-#5 @ 12” O.C.</td>
<td>0.62</td>
<td>1’-8” or 2’-8”</td>
<td>Epoxy-coated rebars, upside down U-stirrups with hook, railing to be used alongside pedestrian walkway, 2” min clear cover</td>
</tr>
</tbody>
</table>
### Table 6. State Standard Bridge Rail Summary (Metric Units)

<table>
<thead>
<tr>
<th>State</th>
<th>Bridge Rail Type</th>
<th>Anchor Rebar Size/Spacing</th>
<th>Steel Area Per Meter (mm²)</th>
<th>Barrier Height</th>
<th>Other Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>FL</td>
<td>Traffic Railing 'F' Shape - 32'</td>
<td>2-#16 @ 203 mm O.C.</td>
<td>1,969</td>
<td>0.81 m</td>
<td>See details for bar shapes, 152 mm min embedment depth</td>
</tr>
<tr>
<td></td>
<td>Traffic Railing 'F' Shape - 42'</td>
<td>2-#16 @ 203 mm O.C.</td>
<td>1,969</td>
<td>1.07 m</td>
<td>See details for bar shapes, 203 mm min embedment depth</td>
</tr>
<tr>
<td></td>
<td>Traffic Railing 'F' Shape Median</td>
<td>2-#16 @ 305 mm O.C.</td>
<td>1,312</td>
<td>0.81 m</td>
<td>See details for bar shapes, 152 mm min embedment depth</td>
</tr>
<tr>
<td></td>
<td>Traffic Railing Vertical Shape - 32&quot;</td>
<td>2-#16 @ 305 mm O.C.</td>
<td>1,312</td>
<td>0.81 m</td>
<td>Upside down U-stirrups with tail, 152 mm min embedment depth</td>
</tr>
<tr>
<td></td>
<td>Traffic Railing Vertical Shape - 42&quot;</td>
<td>2-#16 @ 305 mm O.C.</td>
<td>1,312</td>
<td>1.07 m</td>
<td>Upside down U-stirrups with tail, 152 mm min embedment depth</td>
</tr>
<tr>
<td></td>
<td>Traffic Railing - Corral Shape</td>
<td>12-#13 @ Post, 12-#22 @ Post, Posts @ 10' O.C.</td>
<td>4,065</td>
<td>0.81 m</td>
<td>#22 bar=upside down U-stirrups with hook, #13 bar= L-shaped, 1.52 m wide posts, 152 mm min embedment depth</td>
</tr>
<tr>
<td>IA</td>
<td>Barrier Rail 2'-10&quot;</td>
<td>2-#16 @ 305 mm O.C.</td>
<td>1,312</td>
<td>0.86 m</td>
<td>Epoxy-coated rebars, U-stirrups with hooks, 51 mm min clear cover, F-shaped</td>
</tr>
<tr>
<td></td>
<td>Barrier Rail 3'-8&quot;</td>
<td>2-#16 @ 305 mm O.C.</td>
<td>1,312</td>
<td>1.12 m</td>
<td>Epoxy-coated rebars, U-stirrups with hooks, for the first 1.22 m from abutments of bridge: 2-#16 @152 mm O.C., required steel=2,265 mm²/m, 51 mm min clear cover, F-shaped</td>
</tr>
<tr>
<td>IL</td>
<td>F Shaped Parapet</td>
<td>2-#16 @ 279 mm O.C.</td>
<td>1,440</td>
<td>0.86 or 1.07 m</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hooks</td>
</tr>
<tr>
<td>KS</td>
<td>F4 Barrier Curb</td>
<td>2-#16 @305 mm O.C.</td>
<td>1,312</td>
<td>0.81 m</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hooks, F-shaped</td>
</tr>
<tr>
<td></td>
<td>Corral Rail</td>
<td>8-#22 @ Post, 8-#13 @ Post, Posts @ 10' O.C.</td>
<td>4,508</td>
<td>0.69 or 0.81 m</td>
<td>Epoxy-coated rebars, L-shaped bars @ slab/rail interface, 0.91 m wide posts</td>
</tr>
<tr>
<td>MO</td>
<td>CIP Barrier Curb (Type D)</td>
<td>2-#16 @ 305 mm O.C.</td>
<td>1,312</td>
<td>0.81 m</td>
<td>Galvanized rebars, see details for bar shapes, NJ-shaped</td>
</tr>
<tr>
<td></td>
<td>CIP Barrier Curb (Type D)</td>
<td>2-#16 @ 305 mm O.C.</td>
<td>1,312</td>
<td>1.07 m</td>
<td>Galvanized rebars, see details for bar shapes</td>
</tr>
<tr>
<td>OH</td>
<td>Bridge Railing Deflector Parapet</td>
<td>2-#19 @ 305 mm O.C.</td>
<td>1,864</td>
<td>0.91 or 1.07 m</td>
<td>See details for bar shapes, NJ-shaped</td>
</tr>
<tr>
<td>WI</td>
<td>Sloped Face Parapet 'LF'</td>
<td>2-#16 @ 203 mm O.C.</td>
<td>1,969</td>
<td>0.81 m</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, preferred on state and interstate highway bridges, 51 mm min clear cover, F-shaped</td>
</tr>
<tr>
<td></td>
<td>Sloped Face Parapet 'HF'</td>
<td>2-#16 @ 203 mm O.C.</td>
<td>1,969</td>
<td>1.07 m</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, used where there is a high truck traffic and curved horizontal alignment, 51 mm min clear cover, F-shaped</td>
</tr>
<tr>
<td></td>
<td>Sloped Face Parapet '51F'</td>
<td>2-#16 @ 203 mm O.C.</td>
<td>1,969</td>
<td>1.30 m</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, used in median area of adjacent structures, 51 mm min clear cover, F-shaped</td>
</tr>
<tr>
<td></td>
<td>Sloped Face Parapet 'B'</td>
<td>2-#13 @ 229 mm O.C.</td>
<td>1,122</td>
<td>0.81 m</td>
<td>Epoxy-coated rebars, upside down V-stirrups with hook, 51 mm min clear cover, NJ-shaped</td>
</tr>
<tr>
<td></td>
<td>Vertical Face Parapet 'TX'</td>
<td>2-#16 @ 229 mm O.C.</td>
<td>1,755</td>
<td>1.07 m</td>
<td>Epoxy-coated rebars, upside down U-stirrups with hook, decorative railing with windows, 51 mm min clear cover</td>
</tr>
<tr>
<td></td>
<td>Vertical Face Parapet 'A'</td>
<td>2-#16 @305 mm O.C.</td>
<td>1,312</td>
<td>0.51 or 0.81 m</td>
<td>Epoxy-coated rebars, upside down U-stirrups with hook, railing to be used alongside pedestrian walkway, 51 mm min clear cover</td>
</tr>
</tbody>
</table>
CHAPTER 3 - INITIAL MODEL DEVELOPMENT

Design procedures for cast-in-place and post-installed mechanical anchors have been established and accepted by various organizations. The mechanics involved with these types of anchors can be explained with relatively simple equations. However, the strength and mechanics involved with adhesive anchors is highly dependent on the particular adhesive product, which makes it difficult to develop a general design procedure that is applicable for all adhesive anchors. As a result, the design of adhesive anchors is highly dependent on test data obtained from the manufacturer or an independent testing organization. This test data is usually very discrete and does not provide for much flexibility for scenarios not explicitly tested.

Several manufacturers have adopted the design procedure contained in ICC-ES AC308 which allows for much more flexibility of the physical aspects of the anchorage design (i.e. anchor size, embedment depth, spacing, etc). But due to the complicated mechanics involved with adhesive anchors, extensive testing is required to determine the large amount of input parameters for the design equations. In addition, the parameters developed for this procedure are based on static load conditions and do not take into consideration the dynamic effects of impact loading conditions.

3.1 Conventional Anchorage Design Strength

Concrete barriers that utilize epoxy anchorages need to develop either the strength of the conventional cast-in-place anchorage design or that required by the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specification. Section 13 of the AASHTO LRFD Bridge Design Specification states that minor details in approved crash tested designs for bridge rails can be changed...
provided that the proposed installation does not detract from the performance of the crash
tested rail system [71]. Therefore, in order to retrofit epoxy anchors into an existing cast-
in-place barrier, analytical calculations are needed first to determine the strength of the
conventional design.

As illustrated in Chapter 2, the standard anchorage design with the most strength
for the state of Wisconsin and the other Pooled Fund States utilized two no. 5 (metric no.
16) epoxy-coated reinforcing bars spaced 8 in. apart on center. Both shear and moment
strengths are necessary to redirect a vehicle. Subsequently, both the shear and the
overturning moment capacities of the original barrier must be achieved in an epoxy
anchorage design. The Wisconsin Standard Sloped Face Parapet ‘LF’ bridge rail was
selected as the baseline design because it is preferred on most state and interstate
highway bridges [70] and it consisted of upside down U-shaped, epoxy coated no. 5
(metric no. 16) stirrups with a hooked end spaced at 8 in. (203 mm) on center. Figure 11
shows a cross-section of the barrier while detailed drawings and static calculations of the
strength of this bridge rail are shown in Appendix B.

The two force requirements that have to be met are the shear and moment
capacities of the barrier rail. When a vehicle impacts the barrier rail, the shear force at the
bottom of the rail restricts the barrier from moving laterally. An overturning moment
force is also present due to the eccentricity of the applied impact load relative to the
anchorage. The overturning moment capacity restricts the barrier from rotating in the
direction of the applied impact force.

The moment strength of the Wisconsin Sloped Face Parapet ‘LF’ bridge rail is
carried by the tensile force in the hooked end of the stirrup and the bearing force of the
barrier on the edge of the concrete slab. Therefore, the tensile force of the hooked end of the anchor essentially carries the full moment capacity. Most of the shear capacity of the anchor is also carried by the hooked end because the straight embedded end is close to the edge of the concrete and the shear strength is greatly reduced because of the concrete breakout in shear. The shear and moment capacities of a single no. 5 (metric no. 16) stirrup were calculated and then normalized by dividing by the anchor spacing to determine the capacities as forces per length of barrier. Utilizing the calculations shown in Appendix B, the shear and moment strengths of the barrier were found to be 19.13 k/ft and 13.85 k-ft/ft (279.2 kN/m and 61.6 kN-m/m) respectively. Alternatively, a yield line analysis could be completed to verify the strength of a barrier.

Figure 11. Wisconsin Sloped Face Parapet ‘LF’ Cross-Sectional Drawing [64]
3.2 Tensile Failure Modes

Epoxy bonded anchors have three main modes of failure in tension that consist of steel rupture, full concrete cone breakout, and pullout of the adhesive core accompanied by a partial cone breakout. Within the pullout of the adhesive core failure mode, bond failure can occur at the epoxy-concrete interface, the epoxy-anchor interface, or both the epoxy-concrete and the epoxy-anchor interfaces. Since most states prefer to have a protective epoxy coating on the reinforcing bars, the epoxy-anchor interface failure is actually a failure between the protective epoxy coating and the epoxy adhesive. A summary of the failure modes is shown in Figure 12.

![Figure 12. Failure Modes for Epoxy Anchors Loaded in Tension](image_url)

3.3 Tensile Design Models

Three common models that can be used to predict the tensile capacity of adhesive anchors are a uniform bond stress distribution, an elastic bond stress distribution, and a concrete cone failure. Many of the proposed procedures (as outlined in Chapter 2) use a combination of these failure models to better describe the failure mechanisms present and
attempt to improve the accuracy of the solution. Several of the models discussed in the literature review, as well as additional procedures were proposed and compared to test data obtained from the epoxy manufacturer, Hilti.

### 3.3.1 Steel Rupture Model

The steel rupture model is a function of the cross-sectional area of the anchor ($A_s$) and the ultimate strength of the anchor steel ($f_u$). Shown in Equation (21), this is the commonly used equation to calculate the rupture strength of steel materials. This failure controls when there is significant embedment depth to preclude concrete breakout or bond failure.

$$N_n = A_s f_u \quad (21)$$

### 3.3.2 Concrete Cone Model

The concrete cone model is generally only valid for adhesive anchors with shallow embedment depths because the concrete breakout capacity is lower than the bond pullout capacity only at short embedment depths. However, most of the failure modes observed in previous testing had a shallow concrete cone that formed near the surface of the concrete. For deeper embedment depths, the fracture area of the concrete cone increases significantly, eventually reaching a transition from a cone failure to a simultaneous cone and bond failure.

The concrete cone model for cast-in-place and post-installed mechanical anchors vary significantly from the concrete cone model for adhesive anchors. This is because of the inherent differences between the load transfers for the different systems. A cast-in-place anchor generally has a stud or a bend at the embedded end of the anchor which causes most of the load to be transferred at the bottom of the anchor. There is not enough
room for the end of the anchor to slip out so a full concrete cone is pulled out with a
height equal to the embedment depth of the anchor. Similarly, a post-installed mechanical
anchor transfers the load by a bearing force near the bottom of the anchor that is obtained
by the expansion of the anchor.

Conversely, with adhesive anchors, the load is distributed along the bonded area
and there is little stress concentration at the bottom of the anchor. Because the diameter
of the anchor is relatively uniform along the entire embedment depth, there is a relatively
low mechanical interlock between the anchor and the concrete compared to that of cast-
in-place or post-installed mechanical anchors. This allows adhesive anchors to slip out of
the hole before a full concrete cone can develop and usually only a shallow concrete cone
forms near the top.

The concrete model assumed that the strength of the pullout capacity of all tests
would be controlled by the formation of a concrete cone and that the concrete cone was
the only component of the system that contributed to the pullout capacity. The calibration
coefficient in Equation (4) was modified so that English units could be utilized which
resulted in Equation (22) shown below.

\[ N_n = 11.08h_{ef}^2 \sqrt{f'_c} \]  \hspace{1cm} (22)

The procedure used to convert Equation (4) to English units is shown in Appendix
C. Note that in Equation (22), \( h_{ef} \) should use units of inches and \( f'_c \) should use units of
pounds per square inch.

3.3.3 Full Uniform Bond Stress Model

The uniform bond stress model assumes that the stress is transferred evenly across
the entire bonded area by an average uniform bond stress. The average uniform bond
stress value is calculated based on previous test data for the particular adhesive and anchor size, and can be calculated as the failure load divided by the bonded area. The mechanics of this model are very basic as the only required parameters are the average uniform bond stress and the bonded area. This model has been used as the basis for many adhesive anchor design procedures including ICC-ES AC308 and ACI 318-11. Studies have shown that this model accurately predicts the tensile capacities of adhesive anchors for short to medium embedment depths. This model generally yields unconservative values for anchors with deep embedment depths. Short depths include anchors with less than 4 in. (102 mm) embedment, medium depths include embedments between 4 and 8 in. (102 and 203 mm), and deep depths include embedments greater than 8 in. (203 mm).

This model calculated the pullout strength by multiplying the average uniform bond stress by the bond area obtained from the full embedment depth of the anchor, and the equation used to calculate the pullout capacity for the full uniform bond model is shown in Equation (23) below. This model did not take into account the effect of a concrete cone formation, and Equation (23) was used to predict the pullout strength for every test in the database.

\[ N_n = \tau_0 \pi d_0 h_{ef} \]  

(23)

3.3.4 Cone or Full Uniform Bond Model

The height of the concrete cone was first estimated by a modified version of Equation (3), where English units of inches and pounds were utilized. The resulting equation is shown below.

\[ h_{cone} = \frac{\tau_0 \pi d_0}{22.13 \sqrt{f'_c}} \]  

(24)
Two limit states of either a concrete cone failure or a full uniform bond failure were implemented based on which failure mode was likely to govern. If the estimated height of the concrete cone calculated by Equation (24) was greater than or equal to the total embedment depth, Equation (22) was utilized. Otherwise, the uniform bond stress Equation (23) was used to calculate the pullout capacity.

### 3.3.5 Cone or Partial Uniform Bond with Calculated Cone Height

It was suggested that the pullout capacity of an adhesive bonded anchor could be accurately predicted by the strength predicted by a partially bonded anchor neglecting the concrete cone [6, 15]. This model utilized Equation (22) when the cone height predicted by Equation (24) was greater than the embedment depth. Otherwise a partial uniform bond stress model was used. Recall that the capacity of a partially bonded anchor is given by the following equation.

\[ N_n = \tau_0 \pi d_0 (h_{ef} - h_{cone}) \]

### 3.3.6 Cone or Partial Uniform Bond with Assumed Cone Height

Equation (24) did not agree with the observations made by Collins, Klingner, and Polyzois [7] that the height of the concrete cone decreased with an increase in embedment depth, and the calculated heights of the cone were greater than the observed heights noted in previous studies [6, 7]. Therefore, the accuracy of Equations (3) and (24) were questioned. In lieu of a better equation to predict the height of the concrete cone, the cone height was assumed to be equal to 2 in. (51 mm) in all cases as recommended by Doerr and Klingner [6]. The procedure in section 3.3.5 was modified with \( h_{cone} \) equal to 2 in. (51 mm) for all cases.
3.3.7 Cone or Cone Plus Partial Uniform Bond Model with Calculated Cone Height

This model was similar to the one proposed by Cook [8] except that the equations were converted to English units and the elastic bond stress equation was not utilized because of the lack of available data for the maximum bond stress and the adhesive stiffness parameter. When the height of the concrete cone predicted by Equation (24) was less than or equal to the embedment depth of the anchor, the concrete cone Equation (22) was used. Otherwise, a modified version of Equation (5), which allows the input of English units of inches and pounds, was used. This equation is shown below.

\[ N_n = \tau_0 \pi d_0 (h_{ef} - h_{cone}) + 11.08 h_{ef}^2 \sqrt{f_c} \left[ \frac{7.93 \sqrt{d_0} - (h_{ef} - h_{cone})}{7.93 \sqrt{d_0}} \right] \]  (26)

3.3.8 Cone or Cone Plus Partial Uniform Bond Model with Assumed Cone Height

Again, due to the questionable accuracy of Equations (3) and (24), the height of the concrete cone was assumed to be 2 in. (51 mm) to test this model with a potentially more accurate concrete cone height. The procedure presented in section 3.3.7 was repeated with \( h_{cone} \) taken to be 2 in (51 mm).

3.3.9 Modified Cone or Cone Plus Partial Uniform Bond Model with Assumed Cone Height

The procedure presented in section 3.3.8 was repeated with the last bracketed term in Equation (26) dropped out of the equation. Occasionally, the elastic bond stress model is not considered due to the lack of the required parameters. Therefore, this term is not needed. The resulting equation is shown below.
The elastic bond stress model theoretically better describes the mechanics of an adhesive anchor than the uniform bond stress model. It also satisfies both the compatibility of equilibrium and displacements at the anchor-adhesive interface while the uniform bond stress model only satisfies equilibrium [9]. The derivation of this model is obtained by setting the net energy of the adhesive anchor system equal to the total internal strain energy minus the external energy. A drawing of the adhesive anchor with the geometric variables is shown in Figure 13.

\[
N_h = \tau_0 \pi d_0 (h_{ef} - h_{cone}) + 11.08 h_{ef}^2 \sqrt{f_c}
\]

(27)

3.3.10 Elastic Bond Stress Model

Figure 13. Adhesive Anchor Used to Develop the Elastic Model
The internal energy in the steel \((U_s)\) is given by Equation (28) where \(\sigma\) is the axial stress in the steel, \(\varepsilon\) is the axial strain in the steel, and \(h_{ef}\) is the embedment depth of the anchor.

\[
U_s = \frac{1}{2} \int_0^{h_{ef}} \int_A \sigma \varepsilon \, dA \, dy
\]  

(28)

If the axial displacement of the anchor is given by the function \(u(y)\), then the strain is calculated as the first derivative of the displacement function.

\[
\varepsilon = \frac{du(y)}{dy} = u'
\]  

(29)

By assuming a linear relationship between the stress and strain, the axial stress can be determined by Hook’s law as the modulus of elasticity of the steel \((E_s)\) times the strain.

\[
\sigma = E_s \varepsilon
\]  

(30)

By combining Equations (29) and (30), the stress can be expressed by Equation (31).

\[
\sigma = E_s u'
\]  

(31)

The area of the anchor can be assumed to be constant throughout the embedment depth, so the integral over the area can be reduced to the cross-sectional area of the anchor steel \((A_s)\).

\[
\int_A dA = A_s
\]  

(32)

By substituting Equations (29), (31), and (32) into Equation (28), the internal energy of the steel is given by Equation (33).
\[ U_s = \frac{1}{2} \int_0^{h_{ef}} E_s A_s (u')^2 \, dy \]  

(33)

The internal energy of the adhesive \((U_a)\) is given by Equation (34) where \(\tau\) is the shear stress in the adhesive and \(\gamma\) is the shear strain in the adhesive.

\[ U_a = \frac{1}{2} \int_0^{h_{ef}} \int_A \tau \gamma \, dA \, dy \]  

(34)

By assuming an elastic response by the adhesive, the shear stress can be determined by Hook’s law in shear as the shear modulus of elasticity of the adhesive \((G_a)\) times the shear strain.

\[ \tau = G_a \gamma \]  

(35)

The shear strain is defined as the axial displacement divided by the thickness of the adhesive layer \((t)\).

\[ \gamma = \frac{u}{t} \]  

(36)

By combining Equations (35) and (36), the shear stress can be calculated by Equation (37).

\[ \tau = \frac{G_a u}{t} \]  

(37)

The integral over the area in Equation (34) can be approximated by Equation (38) where \(d_0\) is the diameter of the hole.

\[ \int_A dA \approx \pi d_0 t \]  

(38)

By substituting Equations (36), (37), and (38) into Equation (34), the internal energy due to the adhesive is given by Equation (39).
The external energy applied to the system \( (U_e) \) is simply the work applied by the load at the top of the anchor. This is given by Equation (40) where \( N \) is the applied tensile load at the top of the anchor, and \( u(h_{ef}) \) is the deflection of the anchor at the concrete surface (the function \( u \) evaluated at \( h_{ef} \)).

\[
U_e = Nu(h_{ef})
\]  

(40)

By combining Equations (33), (39), and (40), the net energy of the system \( (U_{net}) \) is shown below.

\[
U_{net} = U_s + U_a - U_e
\]

(41)

\[
U_{net} = \frac{1}{2} \int_0^{h_{ef}} E_s A_s (u')^2 dy + \frac{1}{2} \int_0^{h_{ef}} \frac{G_a d_0 \pi u^2}{t} dy - Nu(h_{ef})
\]

Based on the principle of minimum total potential energy, the internal energy will approach a minimum value at equilibrium [72]. The resulting second order homogeneous differential equation obtained by minimizing the net energy of the system with respect to the displacement is shown in Equation (42).

\[
u'' - \lambda^2 u = 0
\]  

(42)

The \( \lambda^2 \) variable is an elastic property of the anchor system and is given in terms of the dimensions and material properties of the components, as shown in Equation (43).

\[
\lambda^2 = \frac{G_a d_0 \pi}{t E_s A_s}
\]  

(43)

The second order homogeneous differential equation can be put into a more general form as shown in Equation (44) where \( a = 1, b = 0, \) and \( c = -\lambda^2 \).
Equation (44) can be solved by finding the roots of the following equation.

\[ ar^2 + br + c = 0 \]
\[ = 1r^2 + 0r + (-\lambda^2) = 0 \]
\[ = (r - \lambda)(r + \lambda) = 0 \]
\[ \therefore r = \pm \lambda \]

Therefore, the general solution to Equation (44) is shown below.

\[ u = c_1 e^{\lambda y} + c_2 e^{-\lambda y} \] (46)

The initial conditions of \( u(0) = 1 \) and \( u'(0) = 0 \) are satisfied if \( c_1 = c_2 = 1/2 \). Therefore, Equation (46) can be expressed as the following equation.

\[ u_1 = \frac{1}{2} e^{\lambda y} + \frac{1}{2} e^{-\lambda y} = \cosh(\lambda y) \] (47)

Similarly, the initial conditions of \( u(0) = 0 \) and \( u'(0) = 1 \) are satisfied if \( c_1 = 1/2 \) and \( c_2 = -1/2 \). Therefore, Equation (46) can be expressed as the following equation.

\[ u_2 = \frac{1}{2} e^{\lambda y} - \frac{1}{2} e^{-\lambda y} = \sinh(\lambda y) \] (48)

Equations (47) and (48) form a fundamental set of solutions, and the general solution to Equation (42) is shown below [73].

\[ u(y) = k_1 \cosh(\lambda y) + k_2 \sinh(\lambda y) \] (49)

The first derivative of Equation (49) with respect to \( y \) is shown below.

\[ u'(y) = k_1 \lambda \sinh(\lambda y) + k_2 \lambda \cosh(\lambda y) \] (50)
By assuming that the epoxy below the bottom of the anchor carries no load, the strain at the bottom of the anchor is equal to zero.

\[ \varepsilon(0) = u'(0) = 0 \] (51)

By applying the above boundary condition to Equation (50), the constant \( k_2 \) must be equation to zero.

\[ k_2 = 0 \] (52)

The second boundary condition is derived from the strain in the anchor steel at the concrete surface. The value of the strain in the anchor at the concrete surface is shown in Equation (53).

\[ \varepsilon(h_{ef}) = u'(h_{ef}) = \frac{N}{A_s E_s} \] (53)

By applying Equations (52) and (53) to Equation (50), the constant \( k_1 \) can be solved for and is given by Equation (54).

\[ k_1 = \frac{N}{A_s E_s \lambda \sinh(\lambda h_{ef})} \] (54)

By substituting the constants \( k_1 \) and \( k_2 \) into Equation (49), the displacement function is given by Equation (55).

\[ u(y) = \frac{N \cosh(\lambda y)}{E_s A_s \lambda \sinh(\lambda h_{ef})} \] (55)

Rearranging the above equation yields the following equation for the applied tensile load.

\[ N = u(y) E_s A_s \lambda \frac{\sinh(\lambda h_{ef})}{\cosh(\lambda y)} \] (56)
Since the maximum shear stress \((\tau_{\text{max}})\) will occur at the top of the anchor, the shear strain at the top of the anchor can be determined based on the maximum shear stress.

\[
\gamma(h_{ef}) = \frac{\tau_{\text{max}}}{G_a} = \frac{u(h_{ef})}{t}
\]  

(57)

Rearranging these terms to solve for the axial displacement at the top of the anchor yields the following equation.

\[
u(h_{ef}) = \frac{t\tau_{\text{max}}}{G_a}
\]  

(58)

Combining Equations (56) and (58), the maximum force at the top of the anchor \((y = h_{ef})\) can be calculated by Equation (59).

\[
N_{\text{max}} = \frac{t\tau_{\text{max}}}{G_a}E_sA_s\lambda \tanh(\lambda h_{ef})
\]  

(59)

Equation (43) can be rearranged to the following equation.

\[
\frac{tE_sA_s}{G_a} = \frac{d_0\pi}{\lambda^2}
\]  

(60)

By substituting Equation (60) in Equation (59), the maximum tensile load of an adhesive anchor can be expressed as follows.

\[
N_{\text{max}} = \frac{\tau_{\text{max}}d_0\pi}{\lambda} \tanh(\lambda h_{ef})
\]  

(61)

However, the \(\lambda\) term is dependent on the diameter of the hole. If the area of the steel is approximated by the area of the hole, then the adhesive stiffness parameter \((\lambda')\) can be derived by the following procedure.

\[
A_s \approx \frac{\pi d_0^2}{4}
\]  

(62)
Finally, substituting Equation (64) into Equation (61) yields the equation for the elastic bond stress model that is shown below [6].

\[
N_{\text{max}} = \frac{\pi t_{\text{max}} d_0^{1.5}}{\lambda'} \tanh \left( \frac{\lambda' h_{ef}}{\sqrt{d_0}} \right)
\]  

(66)

This equation is limited by that fact that it only accounts for load transfer through the bonded interface. As explained in the literature review, many of the failures observed from prior testing had a concrete cone failure near the concrete surface. The maximum shear stress in Equation (66) will not necessarily be controlled by the maximum shear stress that the adhesive can hold before failure in shear, but could be controlled by the shearing stress at the adhesive-concrete or adhesive-anchor interfaces. This is difficult to determine and could be highly sensitive to installation conditions and the particular adhesive used.

3.4 Shear Design Models

Due to the similar behavior between adhesive, cast-in-place, and post-installed mechanical anchors in shear, the provisions presented in Appendix D of ACI 318-08 appear to be applicable to adhesive anchors. The only significant addition that ICC-ES AC308 provides to ACI 318-08 as a specification for the design of adhesive anchors in
shear is a section that computes the nominal pryout strength in shear. This capacity is based on the pryout and breakout strengths of the anchor in tension. The results of Bickel and Shaikh’s study indicated that the CCD method was adequate to predict the capacities of adhesive anchors loaded in shear [14]. Since that study, a more recent revision of the code, ACI 318-11 was released, that is very similar to the previous methods. Therefore, the specification presented in ACI 318-11 was adopted in the design guide developed in this report.

3.5 Pullout Model Comparisons to Manufacturer Test Data

In order to evaluate the various models to predict the dynamic pullout capacities of epoxy-bonded anchors, several models were compared to test capacities from single anchor pullout tests. The products chosen to use in this study was the Hilti HIT-RE 500 and the Hilti HIT-RE 500-SD adhesive epoxies. The “SD” indicates the epoxy can be used for strength design. The Hilti products showed high anchorage capacities and is available from many suppliers around the country as well as direct sales from Hilti. Comparison of epoxy manufacturers’ specified load is shown in Appendix A. Further, extensive testing was conducted by ICC-ES to determine the bond stress properties of the Hilti HIT-RE 500-SD epoxy that are needed to implement into the uniform bond stress model contained in ICC-ES AC308. The results from this testing program are contained in the ICC-ES report ESR-2322 [74].

The models described in sections 3.3.2 through 3.3.9 were evaluated and compared to a database of test data from the manufacturer. For each model investigated, the pullout capacity was calculated based on the adhesive parameters and the physical dimensions of the test specimen. A test-to-predicted capacity ratio was calculated for
each data point by dividing the actual capacity obtained from the test data by the calculated capacity determined by the model. For each model, the mean of the test-to-predicted ratios for all data points was used to examine the accuracy. The standard deviation and coefficient of variation (COV) of the mean test-to-predicted ratios were calculated for each model to analyze the precision. Due to the complexities involved with determining the parameters for the elastic bond stress model, only variations of the concrete cone and/or uniform bond stress models were analyzed during this part of the research.

The 2008 Hilti North American Product Technical Guide specifies a single bond stress value according to ASTM C882-91 for the Hilti HIT-RE 500 epoxy [19]. However, ICC-ES ESR-2322 specifies bond strengths that decreased with an increase in anchor diameter for the Hilti HIT-RE 500-SD epoxy [74]. The bond strength value stated in the Hilti technical guide for the HIT-RE 500 epoxy was lower than the lowest value for the HIT-RE 500-SD listed in ICC-ES ESR-2322.

3.5.1 Comparison of Proposed Models with Test Data

All of the models were calculated with the bond stress specified by the Hilti Technical Guide and the bond stress values obtained from ICC-ES ESR-2322. The calculated pullout capacities and the corresponding test-to-predicted ratios for the various models are shown in Appendix D. For each model, a mean test-to-predicted ratio was calculated as well as the standard deviation and coefficient of variation of the mean values. A summary of the mean test-to-predicted ratios and coefficients of the variations for all models are shown in Table 7. Due to the lack of detailed test data, these values are
based solely on the ultimate strengths and do not consider whether the proper failure
mechanism that was predicted matched that observed from testing.

Most of the models had a slightly better relation to the actual test capacities with
the bond stress values contained in ICC-ES ESR-2322. This suggested that the average
uniform bond stress is not constant for all anchor sizes and embedment depths. However,
the specified bond stress from the Hilti product documentation provided good results that
were slightly more conservative than the more detailed bond stress values obtained from
ICC-ES ESR-2322.

Table 7. Summary of Model Comparison with Hilti Test Data

<table>
<thead>
<tr>
<th>Model Type</th>
<th>Bond Stress Specified in Hilti Technical Guide</th>
<th>Bond Stress Specified In ICC-ES ESR-2322</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Test-to-Predicted Ratio</td>
<td>COV of Mean Test-to-Predicted Ratio</td>
</tr>
<tr>
<td>Full Uniform Bond Model</td>
<td>0.87</td>
<td>0.27</td>
</tr>
<tr>
<td>Concrete Cone Model</td>
<td>1.00</td>
<td>0.19</td>
</tr>
<tr>
<td>Cone or Full Uniform Bond Model</td>
<td>1.04</td>
<td>0.15</td>
</tr>
<tr>
<td>Cone or Cone Plus Partial Uniform Bond Model</td>
<td>1.42</td>
<td>0.18</td>
</tr>
<tr>
<td>Cone or Cone Plus Partial Uniform Bond Model with Calculated Cone Height</td>
<td>1.26</td>
<td>0.20</td>
</tr>
<tr>
<td>Cone or Cone Plus Partial Uniform Bond Model with Assumed Cone Height</td>
<td>1.17</td>
<td>0.19</td>
</tr>
<tr>
<td>Modified Cone or Cone Plus Partial Uniform Bond Model with Assumed Cone Height</td>
<td>3.63</td>
<td>2.46</td>
</tr>
<tr>
<td>Cone or Partial Uniform Bond with Calculated Cone Height</td>
<td>1.62</td>
<td>0.62</td>
</tr>
<tr>
<td>Cone or Partial Uniform Bond with Assumed Cone Height</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The full uniform bond model was the most unconservative of all models analyzed. It was extremely unconservative for the shortest embedment depths in which the calculated capacity was usually close to twice the test capacity. This was expected since for short embedment depths, a small cone failure with little or no bond failure is likely to occur. Therefore, the full bond strength is not developed because the concrete fails before the adhesive reaches its maximum limit. For longer embedment depths, this model showed good results.

The concrete cone model was very accurate in estimating the capacities for all anchors as its mean test-to-predicted value was equal to 1.0 with a relatively small coefficient of variation. This model, however, does a poor job of describing the mechanics of the actual failure modes that would be expected to occur.

The cone or full uniform bond model also did an excellent job of predicting the pullout capacities for all anchors. The mean test-to-predicted values were 1.04 and 0.98 for bond stress values specified by the Hilti product documentation and ICC-ES ESR-2322, respectively. This model also had the smallest coefficient of variation of all the models compared.

The cone or cone plus partial uniform bond with the calculated cone height was conservative for every data point. For shorter embedment depths, the calculated values corresponded to the concrete cone model values which were only slightly below the actual test capacities. For medium to deep embedment depths this model was very conservative. Theoretically, this model does an adequate job of explaining the failure mechanisms that are expected to be present. However, it does not work with the particular equation used to estimate the concrete breakout strength and the bond stress
values input into this model. The error in this model is believed to be attributed to the inaccuracy of Equation (24). This equation was believed to overestimate the height of the concrete cone compared to previous testing observed from the literature review.

When the cone or cone plus partial uniform bond model was used with an assumed cone height of 2 in. (51 mm), slightly more accurate results were obtained. Perhaps this is because the cone height equation predicts the cone height to be too large, resulting in an overestimation of the strength contributed by the concrete breakout. With a smaller cone height the influence of the bond stress has a greater contribution to the overall capacity, especially for deeper embedment depths.

The modified cone or cone plus partial uniform bond model showed good results that were slightly conservative. The average test-to-predicted ratio was 1.05 when the bond stress values from ICC-ES ESR-2322 were used. The results were slightly less accurate than the cone or full uniform bond model, but the modified cone or cone plus partial uniform bond model better describes the actual failure modes that would be expected to be present.

Both the cone or partial bond models calculated capacities that were quite conservative. These models also became very unstable if the calculated or assumed height of the concrete cone was slightly less than the embedment depth. This is because when the embedment depth was slightly more that the cone height, only a very small bond area was considered to develop the capacity of the anchor.

3.6 Creep Consideration

As discussed in the literature review, all qualified products approved by ICC-ES AC308 or ACI 355. Y are required to be tested and meet creep criteria. Since epoxy is a
visco-plastic material, creep of the anchors would only occur because of long-term sustained tensile loading. Bridge railings and barriers are supported vertically by the bridge deck so there is not any long-term sustained tensile loading in the anchors. An impact of a crash on the barrier would not allow a long enough duration load to induce creep behavior of the anchors. Therefore, creep of the anchors does not need to be a design consideration when using epoxy anchors in bridge rail and temporary barrier anchorages.

3.7 Discussion

The cone or full uniform bond model showed a high correlation to test data obtained from the manufacturer for static loading conditions. This model proved to be the most accurate and stable for all embedment depths while providing a reasonable prediction of the expected failure mode. Therefore, a limit state design of either a concrete cone breakout or a full uniform bond failure was selected for further development. For this method, two failure strengths would be calculated and the lower of the two failure modes would be the governing strength. However, in bridge rail applications, the cone model is not likely to be the governing design consideration due the fact that very short embedment depths will not be utilized and a bond failure mode will most likely govern in most cases. Further, the manufacturers’ specifications provide the bond stress values for the epoxies that can be easily and quickly implemented into the full uniform bond model. The elastic model solution appeared to show validity based on the energy method of analysis. However, the complex parameters required would not be readily available without additional testing for each product.
4.1 Purpose

Dynamic bogie tests were conducted on epoxy anchors to determine their capacities under dynamic loads in both shear and tension. Both epoxy-coated and plain black steel reinforcing bars were tested to investigate how protective coatings affect the strength of the anchor. Duel anchor tensile tests were also conducted to determine if closely spaced anchors experienced a reduction in tensile capacity.

4.2 Scope

Custom designed test jigs, as explained in section 4.4.2, were used to transfer the momentum of the bogie vehicle into dynamic forces on the anchors. The target impact conditions were 10 mph (16.09 km/h) for single anchor tests and 15 mph (24.14 km/h) for double anchor tests. All tests were conducted in an unreinforced concrete slab with an unconfined compressive strength of 6,454 psi (44.50 MPa) according to concrete cylinder testing. The anchor holes were constructed using a carbide-tipped concrete bit and a rotary hammer drill. The holes were clean by repeated brushing and blowing compressed air into the hole according to the manufacturer’s specifications. Material strength specifications sheets are shown in Appendix H. Material specifications were not available for the ASTM A307 threaded rods. The test setup drawings for test nos. WEAB-1 through WEAB-8 are shown in Figures 14 and 15. Detailed test setup drawings for all tests are shown in Appendix G. The test matrix is shown in Table 8.

4.3 Test Facility

The testing facility is located at the Lincoln Air Park on the northwest side of the Lincoln Municipal Airport and is approximately 5 miles (8.0 km) northwest of the University of Nebraska-Lincoln.
Table 8. Dynamic Bogie Test Matrix

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Bar Size, US (Metric)</th>
<th>Bar Coating</th>
<th>Epoxy Type</th>
<th>Spacing</th>
<th>Target Bogie Speed, mph (km/h)</th>
<th>Bogie Weight, lb (kg)</th>
<th>Steel Type</th>
<th>Ultimate Steel Strength, ksi (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-1</td>
<td>Tensile</td>
<td>#5 (#16)</td>
<td>None</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,485 (674)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-2</td>
<td>Tensile</td>
<td>#5 (#16)</td>
<td>None</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,485 (674)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-3</td>
<td>Tensile</td>
<td>#5 (#16)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,485 (674)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-4</td>
<td>Tensile</td>
<td>#5 (#16)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,485 (674)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-5</td>
<td>Shear</td>
<td>#5 (#16)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,735 (787)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-6</td>
<td>Shear</td>
<td>#5 (#16)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,735 (787)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-7</td>
<td>Tensile</td>
<td>#5 (#16)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,735 (787)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-8</td>
<td>Tensile</td>
<td>#5 (#16)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,735 (787)</td>
<td>ASTM A615, Grade 60</td>
<td>103,937 (717)</td>
</tr>
<tr>
<td>WEAB-9</td>
<td>Tensile</td>
<td>#6 (#19)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>15.00 (24.14)</td>
<td>1,727 (783)</td>
<td>ASTM A615, Grade 60</td>
<td>100,400 (692)</td>
</tr>
<tr>
<td>WEAB-10</td>
<td>Tensile</td>
<td>#6 (#19)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>15.00 (24.14)</td>
<td>1,727 (783)</td>
<td>ASTM A615, Grade 60</td>
<td>100,400 (692)</td>
</tr>
<tr>
<td>WEAB-11</td>
<td>Tensile</td>
<td>#6 (#19)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>15.00 (24.14)</td>
<td>1,727 (783)</td>
<td>ASTM A615, Grade 60</td>
<td>100,400 (692)</td>
</tr>
<tr>
<td>WEAB-12</td>
<td>Tensile</td>
<td>#6 (#19)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>15.00 (24.14)</td>
<td>1,727 (783)</td>
<td>ASTM A615, Grade 60</td>
<td>100,400 (692)</td>
</tr>
<tr>
<td>WEAB-13</td>
<td>Shear</td>
<td>#6 (#19)</td>
<td>Epoxy</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,736 (787)</td>
<td>ASTM A615, Grade 60</td>
<td>100,400 (692)</td>
</tr>
<tr>
<td>WEAB-14</td>
<td>Tensile</td>
<td>1 1/8 in. (29 mm)</td>
<td>None</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>15.00 (24.14)</td>
<td>1,505 (682)</td>
<td>ASTM A307</td>
<td>60,000+ (414)*</td>
</tr>
<tr>
<td>WEAB-15</td>
<td>Shear</td>
<td>1 1/8 in. (29 mm)</td>
<td>None</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>10.00 (16.09)</td>
<td>1,741 (790)</td>
<td>ASTM A307</td>
<td>60,000+ (414)*</td>
</tr>
<tr>
<td>WEAB-16</td>
<td>Tensile</td>
<td>#6 (#19)</td>
<td>None</td>
<td>Hilit HIT-RE 500-SD</td>
<td>Single</td>
<td>15.90 (25.58)</td>
<td>1,723 (782)</td>
<td>ASTM A615, Grade 60</td>
<td>100,400 (692)</td>
</tr>
</tbody>
</table>

*Based on rated material capacities, not actual capacities

4.4 Equipment and Instrumentation

4.4.1 Accelerometers

Two environmental shock and vibration sensor/recorder systems were used to measure the accelerations in the longitudinal direction for test nos. WEAB-1 through WEAB-16. All of the accelerometers were mounted near the center of gravity of the bogie.

The first accelerometer system was a two-arm piezoresistive accelerometer system manufactured by Endevco of San Juan Capistrano, California. The accelerometer
Figure 14. Tension Test Setup, Test Nos. WEAB-1 Though WEAB-5, WEAB-7 and WEAB-8.
Figure 15. Shear Test Setup, Test Nos. WEAB-5 Though WEAB-6
was used to measure the longitudinal accelerations at a sample rate of 10,000 Hz. The accelerometer was configured and controlled using a system developed and manufactured by Diversified Technical Systems, Inc. (DTS) of Seal Beach, California. More specifically, data was collected using a DTS Sensor Input Module (SIM), Model TDAS3-SIM-16M. The SIM was configured with 16 MB SRAM and 8 sensor input channels to 250 kB SRAM/channel. The SIM was mounted on a TDAS3-R4 module rack. The module rack was configured with isolated power/event/communications, 10BaseT Ethernet and RS232 communication, and an internal backup battery. Both the SIM and module rack were crashworthy. The “DTS TDAS Control” computer software program and a customized Microsoft Excel worksheet were used to analyze and plot the accelerometer data. For test nos. WEAB-10 through WEAB-15, two DTS accelerometers were utilized.

The second system, Model EDR-3, was a triaxial piezoresistive accelerometer system manufactured by IST of Okemos, Michigan. The EDR-3 was configured with 256 kB of RAM, a range of ±200 g’s, a sample rate of 3,200 Hz, and a 1,120 Hz low-pass filter. The “DynaMax 1 (DM-1)” computer software program and a customized Microsoft Excel worksheet were used to analyze and plot the accelerometer data.

4.4.2 Test Jigs

Two test jigs were utilized in the bogie tests to apply either shear or tensile loads to the anchors. The tensile jig design consisted of a 28-in. (711-mm) long W6x25 (W150x37.1) I-beam welded to a 1-in. (25-mm) thick base plate. The reinforcing bar anchors were held by Erico Lenton LOCK or Dayton Bar Lock mechanical reinforcing bar splices that were installed on the reinforcing bars above the base plate. The center
connecting pin was removed to allow the use of more bolts to grip the reinforcing bar. Hex nuts were used for tests that involved threaded rod. A kick plate was attached to the concrete slab on the non-impact side of the test jig to provide shear resistance and allow the jig to rotate putting a tensile load on the anchors. A drawing of the tensile jig is shown in Figure 16.

![Tensile Jig Diagram]

Figure 16. Tensile Jig

The shear jig consisted of two 3-ft (0.91-m) long, C6x8.2 (C150x12.2) channels welded to a metal sled plate on the front end and an impact plate on the rear end. The anchors were held by Erico Lenton LOCK or Dayton Bar Lock mechanical reinforcing bar splices that were installed on top of the sled plate on the front end of the test jig. A metal strap was wrapped around the reinforcing bar splices to attach the anchors to the jig and prevent rotation of the anchors upon impact. The center connection pin was removed to allow the use of more bolts to grip the reinforcing bar. Hex nuts were used for tests that involved threaded rod. The impact plate was welded to the channels with ½-in. (12.7-
mm) stiffener plates and a long metal guidance plate was welded to the rear end of the channels. A metal plate was screwed to the concrete above the guidance plate to prevent the jig from yawing or lifting off the concrete surface. Calculations that estimated the maximum loads that would be applied to the test jigs are shown in Appendix E. Analytical design calculations and detailed drawings of the test jigs are shown in Appendix F. The test jigs were modified accordingly to accommodate larger anchors for test nos. WEAB-9 through WEAB-16. A drawing of the tensile jig is shown in Figure 17.

![Figure 17. Shear Jig](image)

### 4.4.3 Bogie

The dynamic bogie tests were conducted using a corrugated beam guardrail to guide the tires of the bogie vehicle. A pickup was used to push the bogie vehicle to the required impact velocity. After reaching the target velocity, the push vehicle braked allowing the bogie to be free rolling as it came off the track. For the tension tests, the bogie head impacted the test jig at an impact height of approximately 21 5/8 in. (549 mm). This caused the tension jig to rotate applying an upward force to the anchors. For the shear tests, the bogie head impacted the test jig at an impact height of approximately
7 3/8 in. (187 mm). The bogie impact load was transferred to the anchor as the shear jig translated along the concrete surface.

A rigid frame bogie was used to impact the test jigs. For the tensile tests, the bogie head was constructed of an 8-in. (203-mm) diameter, ½-in. (13-mm) thick standard steel pipe, with ¾-in. (19-mm) neoprene belting wrapped around the pipe to prevent local damage to the post from the impact. The height of impact for the tensile test was 21.66 in. (550 mm). A variable height, detachable steel impact head was used in the shear tests. The shear impact head had an impact height of approximately 7 5/16 in. (186 mm) from the ground surface. A ¾ in. (19-mm) neoprene pad was attached to the impact plate of the shear jig. Pictures of the bogie and test setups are shown in Figures 18 and 19.

Figure 18. Tensile Test Setup
4.4.4 Pressure Tape Switches

Three pressure tape switches were placed near the end of the bogie track and were used to determine the speed of the bogie before impact. The switches were spaced at approximately 18 in. (457 mm) for test nos. WEAB-1 through WEAB-8 and 39.37 in. (1 m) for test nos. WEAB-9 through WEAB-16. As the right-front tire of the bogie passed over each tape switch, a strobe light was fired sending an electronic timing signal to the data acquisition system. The system recorded the signals and the time each occurred. The speed was then calculated using the spacing between the sensors and the time between the signals. Strobe lights and high-speed video analysis are used only as a backup in the event that vehicle speeds cannot be determined from the electronic data.
4.4.5 Digital Cameras

Two AOS VITcam high-speed digital video cameras and one JVC digital video camera were used to document each test. The AOS high-speed camera had a frame rate of 500 frames per second and the JVC digital video camera had a frame rate of 29.97 frames per second. All the cameras were placed laterally from the test jig, with a view perpendicular to the bogie’s direction of travel. A Nikon D50 digital still camera was also used to document pre- and post-test conditions for all tests.

4.4.6 Data Processing

The electronic accelerometer data obtained in dynamic testing was filtered using the SAE Class 60 Butterworth filter conforming to the SAE J211/1 specifications [75]. The pertinent acceleration signal was extracted from the bulk of the data signals. The processed acceleration data was then multiplied by the mass of the bogie to get the impact force using Newton’s Second Law. Next, the acceleration trace was integrated to find the change in velocity versus time. Initial velocity of the bogie, calculated from the pressure tape switch data, was then used to determine the bogie velocity, and the calculated velocity trace was integrated to find the bogie’s displacement. This displacement is also the displacement of the test jig at the impact location. Combining the previous results, a force vs. deflection curve was plotted for each test. Finally, integration of the force vs. deflection curve provided the energy vs. deflection curve for each test.

The anchor force for the tensile tests was determined by summing the moments about the reaction point of the test jig and solving for the anchor force. The reaction point was estimated to be the end of the base plate on the non-impact side. This was selected because after a slight rotation of the test jig, only that point would be in contact with the
concrete and only a point force would be applied to the jig at this point. Therefore, the anchor force was calculated as the bogie force multiplied by the ratio of the vertical distance to the horizontal distance from the reaction point to the bogie force. A free body diagram of the forces associated with the tension test jig is shown in Figure 20.

The anchor force for the shear jig was calculated as a sum of the forces in the horizontal direction. The frictional forces of the jig sliding on the concrete were neglected so the anchor force was simplified to the bogie force from the accelerometer data.

Figure 20. Free Body Diagram of the Tension Test Jig
CHAPTER 5 - DYNAMIC TESTING RESULTS AND DISCUSSION

5.1 Dynamic Testing Results

A series of 16 dynamic bogie tests were conducted on various epoxied anchors to determine the shear and tensile capacities. Test nos. WEAB-1 through WEAB-8 utilized no. 5 (metric no. 16) deformed reinforcing bars and test nos. WEAB-9 through WEAB-13 and WEAB-16 utilized no. 6 (metric no. 19) deformed reinforcing bars. Test nos. WEAB-14 and WEAB-15 utilized 1 1/8 in. (29 mm) diameter threaded rod. Both tension and shear tests were conducted for each type of anchor. Duel anchor tension tests were also conducted for the reinforcing bar anchors to determine the effects of closely spaced anchors. The results for test nos. WEAB-1 through WEAB-16 are described in the following sections.
5.1.1 Test No. WEAB-1

For test no. WEAB-1, a single, uncoated no. 5 (metric no. 16) deformed reinforcing bar was loaded in tension. The bogie impacted the test jig at a speed of 9.78 mph (15.74 km/h). The anchor experienced necking and fractured approximately 1 ¼ in. (32 mm) above the concrete surface. A concrete cone of approximately 4 to 5 in. (102 to 127 mm) in diameter by 1 in. (25 mm) deep spalled off from the concrete surface. The concrete cone was split into several small pieces that were disengaged from the anchor. The maximum tensile load observed was 38.8 kips (172.6 kN) according to the EDR-3 data and 37.9 kips (168.6 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 21. A plot of the force versus time history is shown in Figure 22. Sequential photographs are shown in Figure 23.

Figure 21. Pre- and Post-Test Photographs, Test No. WEAB-1
Figure 22. Force vs. Time, Test No. WEAB-1

Figure 23. Sequential Photographs, Test No. WEAB-1
5.1.2 Test No. WEAB-2

For test no. WEAB-2, a single, uncoated no. 5 (metric no. 16) deformed reinforcing bar was loaded in tension. The bogie impacted the test jig at a speed of 10.40 mph (16.74 km/h). The anchor experienced necking and fractured approximately 1¼ in. (32 mm) above the concrete surface. A concrete cone of approximately 3 in. (76 mm) in diameter by ¾ in. (19 mm) deep spalled off from the concrete surface. The concrete cone was split into several small pieces that were disengaged from the anchor. The maximum tensile load observed was 39.8 kips (177.2 kN) according to the EDR-3 data and 38.9 kips (173.2 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 24. A plot of the force versus time history is shown in Figure 25. Sequential photographs are shown in Figure 26.

Figure 24. Pre- and Post-Test Photographs, Test No. WEAB-2
Figure 25. Force vs. Time, Test No. WEAB-2

Figure 26. Sequential Photographs, Test No. WEAB-2
5.1.3 Test No. WEAB-3

For test no. WEAB-3, a single, epoxy-coated no. 5 (metric no. 16) deformed reinforcing bar was loaded in tension. The bogie impacted the test jig at a speed of 9.47 mph (15.24 km/h). The anchor experienced necking and fractured approximately 3 in. (76 mm) above the concrete surface. A concrete cone of approximately 3 ½ in. (89 mm) in diameter by ½ in. (13 mm) deep spalled off from the concrete surface. The concrete cone was split into several small pieces that were bonded to the anchor. The maximum tensile load observed was 35.1 kips (156.2 kN) according to the EDR-3 data and 34.9 kips (155.1 kN) according to the DTS data. The fracture occurred at a localized minimum cross-sectional area that was created from one of the coupler screws. Therefore, the maximum force was governed by an area less than that of a no. 5 (metric no. 16) reinforcing bar. Pre- and post-test photographs are shown in Figure 27. A plot of the force versus time history is shown in Figure 28. Sequential photographs are shown in Figure 29.

![Pre-Test](image1.png) ![Post-Test](image2.png)

Figure 27. Pre- and Post-Test Photographs, Test No. WEAB-3
Figure 28. Force vs. Time, Test No. WEAB-3

Figure 29. Sequential Photographs, Test No. WEAB-3
5.1.4 Test No. WEAB-4

For test no. WEAB-4, a single, epoxy-coated no. 5 (metric no. 16) deformed reinforcing bar was loaded in tension. The bogie impacted the test jig at a speed of 8.86 mph (14.26 km/h). The anchor experienced necking and fractured approximately 2 in. (51 mm) below the concrete surface. A concrete cone of approximately 3 ¾ in. (95 mm) in diameter by 7/8 in. (22 mm) deep spalled off from the concrete surface. The concrete cone was split into several small pieces that were disengaged from the anchor. The maximum tensile load observed was 36.8 kips (163.8 kN) according to the EDR-3 data and 35.1 kips (156.0 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 30. A plot of the force versus time history is shown in Figure 31. Sequential photographs are shown in Figure 32.

Figure 30. Pre- and Post-Test Photographs, Test No. WEAB-4
Figure 31. Force vs. Time, Test No. WEAB-4

Figure 32. Sequential Photographs, Test No. WEAB-4
5.1.5 Test No. WEAB-5

For test no. WEAB-5, a single, epoxy-coated no. 5 (metric no. 16) deformed reinforcing bar was loaded in shear. The bogie impacted the test jig at a speed of 9.64 mph (15.51 km/h). The anchor sheared off at the concrete surface. The maximum shear load observed was 25.7 kips (114.4 kN) according to the EDR-3 data and 32.4 kips (144.0 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 33. A plot of the force versus time history is shown in Figure 34. Sequential photographs are shown in Figure 35.

Figure 33. Pre- and Post-Test Photographs, Test No. WEAB-5
Figure 34. Force vs. Time, Test No. WEAB-5

Figure 35. Sequential Photographs, Test No. WEAB-5
5.1.6 Test No. WEAB-6

For test no. WEAB-6, a single, epoxy-coated no. 5 (metric no. 16) deformed reinforcing bar was loaded in shear. The bogie impacted the test jig at a speed of 9.71 mph (15.62 km/h). The anchor sheared off at the concrete surface. There was a 1/8 in. (8 mm) gap between the epoxy-coated anchor and the edge of the concrete hole on the impact side. The maximum shear load observed was 23.7 kips (105.6 kN) according to the EDR-3 data and 28.4 kips (126.4 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 36. A plot of the force versus time history is shown in Figure 37. Sequential photographs are shown in Figure 38.

![Pre-Test](image1.png) ![Post-Test](image2.png)

Figure 36. Pre- and Post-Test Photographs, Test No. WEAB-6
Figure 37. Force vs. Time, Test No. WEAB-6

Figure 38. Sequential Photographs, Test No. WEAB-6
5.1.7 Test No. WEAB-7

For test no. WEAB-7, two epoxy-coated no. 5 (metric no. 16) deformed reinforcing bars spaced 8 in. (203 mm) apart were loaded in tension. The bogie impacted the test jig at a speed of 16.64 mph (26.78 km/h). One of the anchors fractured 2 1/8 in. (54 mm) below the concrete surface and was accompanied by a concrete cone breakout of 5 in. (127 mm) in diameter by 1 ¼ in. (32 mm) deep. Flaking of the epoxy coating on this anchor was observed at locations that were bonded to the concrete. The other anchor pulled out of the concrete and was accompanied by a concrete cone breakout of 6 in. (152 mm) diameter by 1 ¼ in. (32 mm) deep. Slight flaking of the epoxy coating was observed on the anchor that pulled out. Both reinforcing bars were slightly bent. The maximum tensile load observed was 73.8 kips (328.3 kN) according to the EDR-3 data and 73.8 kips (328.3 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 39. Pictures of the anchors are shown in Figure 40. A plot of the force versus time history is shown in Figure 41. Sequential photographs are shown in Figure 42.

![Pre-Test](image1.png) ![Post-Test](image2.png)

Figure 39. Pre- and Post-Test Photographs, Test No. WEAB-7
Figure 40. Post-Test Anchor Photographs, Test No. WEAB-7
Figure 41. Force vs. Time, Test No. WEAB-7

Figure 42. Sequential Photographs, Test No. WEAB-7
5.1.8 Test No. WEAB-8

For test no. WEAB-8, two epoxy-coated no. 5 (metric no. 16) deformed reinforcing bars spaced 8 in. (203 mm) apart were loaded in tension. The bogie impacted the test jig at a speed of 14.05 mph (22.61 km/h). Both anchors pulled out of the concrete. The concrete breakout area was approximately 29 in. (737 mm) long by 24 in. (610 mm) wide by 4 in. (102 mm) deep at the anchor hole locations. It was suspected that the failure area was much larger than expected because of existing damage to the aged concrete. Bond failures were present on both the epoxy-anchor and the epoxy-concrete interfaces. The epoxy adhesive was attached to the anchor for the bottom 4 in. (102 mm) of both reinforcing bars. Both reinforcing bars were slightly bent. The maximum tensile load observed was 72.6 kips (323.1 kN) according to the EDR-3 data and 72.4 kips (322.1 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 43. Pictures of the pulled-out anchors are shown in Figure 44. A plot of the force versus time history is shown in Figure 45. Sequential photographs are shown in Figure 46.
Figure 44. Post-Test Anchor Photographs, Test No. WEAB-8
Figure 45. Force vs. Time, Test No. WEAB-8

Figure 46. Sequential Photographs, Test No. WEAB-8
5.1.9 Test No. WEAB-9

For test no. WEAB-9, a single, epoxy-coated no. 6 (metric no. 19) deformed reinforcing bar was loaded in tension. The bogie impacted the test jig at a speed of 14.23 mph (22.91 km/h). The anchor pulled out of the concrete hole and still had some epoxy adhesive attached on the bottom half of the embedded anchor length. There was not any flaking of the protective epoxy coating of the anchor. A concrete cone of approximately 3 ½ in. (89 mm) in diameter by ½ in. (13 mm) deep broke out and small concrete chucks were scattered around the anchor area. The maximum tensile load observed was 41.0 kips (182.3 kN) according to the EDR-3 data and 41.6 kips (185.2 kN) according to the DTS data. Pre- and post-test photographs are shown in Figure 47. A picture of the pulled out anchor is shown in Figure 48. A plot of the force versus time history is shown in Figure 49. Sequential photographs are shown in Figure 50.

Figure 47. Pre- and Post-Test Photographs, Test No. WEAB-9
Figure 48. Post-Test Anchor Photograph, Test No. WEAB-9
Figure 49. Force vs. Time, Test No. WEAB-9

Figure 50. Sequential Photographs, Test No. WEAB-9
5.1.10 Test No. WEAB-10

For test no. WEAB-10, a single, epoxy-coated no. 6 (metric no. 19) deformed reinforcing bar was loaded in tension. The bogie impacted the test jig at a speed of 15.73 mph (25.31 km/h). The anchor pulled out of the concrete hole and very little epoxy adhesive was still bonded to the anchor. There was a significant amount of the protective epoxy coating removed from the anchor at the middle 1/3 of the embedded portion. A concrete cone of approximately 4 ½ in. (114 mm) diameter by 1 in. (25 mm) deep broke out and small concrete chunks were scattered around the anchor area. The maximum tensile load observed was 42.7 kips (189.9 kN) according to the EDR-3 data, 44.2 kips (196.5 kN) according to the DTS no. 1 data, and 44.4 kips (197.3 kN) according to the DTS no. 2 data. Pre- and post-test photographs are shown in Figure 51. A picture of the pulled out anchor is shown in Figure 52. A plot of the force versus time history is shown in Figure 53. Sequential photographs are shown in Figure 54.

![Pre-Test](image1.png) ![Post-Test](image2.png)

Figure 51. Pre- and Post-Test Photographs, Test No. WEAB-10
Figure 52. Post-Test Anchor Photograph, Test No. WEAB-10
Figure 53. Force vs. Time, Test No. WEAB-10

Figure 54. Sequential Photographs, Test No. WEAB-10
5.1.11 Test No. WEAB-11

For test no. WEAB-11, two epoxy-coated no. 6 (metric no. 19) deformed reinforcing bars spaced 8 in. (203 mm) apart were loaded in tension. The bogie impacted the test jig at a speed of 15.11 mph (24.32 km/h). The anchors pulled out of the concrete holes and still had most of the epoxy adhesive still bonded to the anchors. The concrete breakout surface was approximately 14 in. (356 mm) by long by 16 in. (406 mm) wide. The maximum depths of the concrete breakout surface were 2 ¾ in. (70 mm) and 3 in. (76 mm), respectively, at the locations of the two anchor holes. The maximum tensile load observed was 60.9 kips (270.8 kN) according to the EDR-3 data, 60.5 kips (269.1 kN) according to the DTS no. 1 data, and 60.6 kips (269.5 kN) according to the DTS no. 2 data. Pre- and post-test photographs are shown in Figure 55. Pictures of the pulled out anchors are shown in Figure 56. A plot of the force versus time history is shown in Figure 57. Sequential photographs are shown in Figure 58.

![Pre-Test](image1.png) ![Post-Test](image2.png)

Figure 55. Pre- and Post-Test Photographs, Test No. WEAB-11
Figure 56. Post-Test Anchor Photograph, Test No. WEAB-11
Figure 57. Force vs. Time, Test No. WEAB-11

Figure 58. Sequential Photographs, Test No. WEAB-11
5.1.12 Test No. WEAB-12

For test no. WEAB-12, two epoxy-coated no. 6 (metric no. 19) deformed reinforcing bars spaced 8 in. (203 mm) apart were loaded in tension. The bogie impacted the test jig at a speed of 15.08 mph (24.26 km/h). The anchors pulled out of the concrete. Anchor no. 1 had some epoxy adhesive still attached to the middle 1/3 of the embedment length and a significant amount of the epoxy coating had flaked off the bottom 1/3 of the embedded length. Anchor no. 2 had some epoxy adhesive still attached on the top 1/3 of the embedded length and most of the protective epoxy coating was flaked away for the bottom ½ of the embedded length. Two separate concrete cone breakouts occurred. The cone size for anchor no. 1 was approximately 6 ½ in. (165 mm) in diameter by 2 in. (51 mm) deep while the cone size for anchor no. 2 was approximately 4 in. (102) in diameter by 1 ½ in. (38 mm) deep. The maximum tensile load observed was 75.7 kips (336.6 kN) according to the EDR-3 data, 75.7 kips (336.0 kN) according to the DTS no. 1 data, and 75.5 kips (335.7 kN) according to the DTS no. 2 data. Pre- and post-test photographs are shown in Figure 59. Pictures of the pulled out anchors are shown in Figure 60. A plot of the force versus time history is shown in Figure 61. Sequential photographs are shown in Figure 62.
Figure 59. Pre- and Post-Test Photographs, Test No. WEAB-12
Figure 60. Post-Test Anchor Photograph, Test No. WEAB-12
Figure 61. Force vs. Time, Test No. WEAB-12

Figure 62. Sequential Photographs, Test No. WEAB-12
5.1.13 Test No. WEAB-13

For test no. WEAB-13, a single, epoxy-coated no. 6 (metric no. 19) deformed reinforcing bar was loaded in shear. The bogie impacted the test jig at a speed of 9.98 mph (16.07 km/h). The anchor sheared off at the concrete surface. There was a 3/8 in. (10 mm) gap between the epoxy-coated anchor and the edge of the concrete hole on the impact side. A small amount of concrete dust and particles were loose on the non-impact side of the anchor. The maximum shear load observed was 32.1 kips (142.9 kN) according to the EDR-3 data, 29.6 kips (131.9 kN) according to the DTS no.1 data, and 28.4 kips (126.4 kN) according to the DTS no. 2 data. Pre- and post-test photographs are shown in Figure 63. A plot of the force versus time history is shown in Figure 64. Sequential photographs are shown in Figure 65.

Figure 63. Pre- and Post-Test Photographs, Test No. WEAB-13
Figure 64. Force vs. Time, Test No. WEAB-13

Figure 65. Sequential Photographs, Test No. WEAB-13
5.1.14 Test No. WEAB-14

For test no. WEAB-14, a single, 1 1/8 in. (29 mm) diameter threaded rod was loaded in tension. The bogie impacted the test jig at a speed of 15.19 mph (24.45 km/h). The anchor pulled out of the concrete hole and had most of the epoxy adhesive still attached on the bottom 2/3 of the embedded length. A concrete cone of approximately 15 in. (381 mm) in diameter by 2 ¾ in. (70 mm) deep broke out and concrete chucks were scattered around the anchor area. The maximum tensile load observed was 43.7 kips (194.5 kN) according to the EDR-3 data, 46.7 kips (207.8 kN) according to the DTS no. 1 data, and 45.5 kips (202.3 kN) according to the DTS no. 2 data. Pre- and post-test photographs are shown in Figure 66. A picture of the pulled out anchor is shown in Figure 67. A plot of the force versus time history is shown in Figure 68. Sequential photographs are shown in Figure 69.

![Pre-Test Photograph](image1)
![Post-Test Photograph](image2)

Figure 66. Pre- and Post-Test Photographs, Test No. WEAB-14
Figure 67. Post-Test Anchor Photograph, Test No. WEAB-14
Figure 68. Force vs. Time, Test No. WEAB-14

Figure 69. Sequential Photographs, Test No. WEAB-14
5.1.15 Test No. WEAB-15

For test no. WEAB-14, a single, 1 1/8 in. (29 mm) diameter threaded rod was loaded in shear. The bogie impacted the test jig at a speed of 9.28 mph (14.93 km/h). The welds on the bogie head fractured before the anchor failed. The anchor experienced plastic deformation and bent to an angle of 6 degrees from the vertical direction. A slight shear fracture surface started to form on the impact side of the anchor. The maximum shear load observed was 43.7 kips (194.2 kN) according to the EDR-3 data, 39.1 kips (173.8 kN) according to the DTS no. 1 data, and 39.2 kips (174.3 kN) according to the DTS no. 2 data. Pre- and post-test photographs are shown in Figure 70. A plot of the force versus time history is shown in Figure 71. Sequential photographs are shown in Figure 72.

![Pre-Test and Post-Test Photographs, Test No. WEAB-15](image)

Figure 70. Pre- and Post-Test Photographs, Test No. WEAB-15
Figure 71. Force vs. Time, Test No. WEAB-15

Figure 72. Sequential Photographs, Test No. WEAB-15
5.1.16 Test No. WEAB-16

For test no. WEAB-16, a single, uncoated no. 6 (metric no. 19) deformed reinforcing bar was loaded in tension. The bogie impacted the test jig at a speed of 15.90 mph (25.58 km/h). The anchor pulled out of the concrete hole and had a small amount of epoxy still attached on the bottom 3 in. (76 mm) of the embedded length. A concrete cone of approximately 6 in. (152 mm) in diameter by 1 ¼ in. (32 mm) deep broke out and small concrete chunks were scattered around the anchor area. The maximum tensile load observed was 49.6 kips (220.4 kN) according to EDR-3 data, 47.0 kips (209.2 kN) according to DTS no. 1 data, and 45.2 kips (200.9 kN) according to DTS no. 2 data. Pre- and post-test photographs are shown in Figure 73. A picture of the pulled out anchor is shown in Figure 74. A plot of the force versus time history is shown in Figure 75. Sequential photographs are shown in Figure 76.
Figure 74. Post-Test Anchor Photograph, Test No. WEAB-16
Figure 75. Force vs. Time, Test No. WEAB-16

Figure 76. Sequential Photographs, Test No. WEAB-16
5.2 Discussion of Results

The dynamic bogie tests were used to establish criteria for the increase in load capacity based on dynamic loading, the effects of using epoxy-coated reinforcing bars, and the effects of groups of anchors located in close proximity to each other. The anchors were embedded 5 ¼ in. (133 mm) for all tests to allow for use in an 8-in. (203-mm) thick bridge deck without damage to the underside during installation. Most of the tests were conducted with either no. 5 or 6 (metric no. 16 or 19) ASTM A615, grade 60 reinforcing bars because those were the most commonly used designs discovered in the review of state standard bridge railings (see Tables 5 and 6). Tension and shear tests were also conducted on 1 1/8 in. (29 mm) ASTM A307 threaded rods, the anchorage used in the tie-down system for the F-shape temporary concrete barrier developed by MwRSF [37]. The results for test nos. WEAB-1 through WEAB-16 are shown in Table 9.

For the no. 5 (metric no. 16) epoxy-coated reinforcing bar tension tests, the average pullout loads were 35.60 kips (159.9 kN) and 73.2 kips (325.5 kN), respectively, for single and double anchors spaced 8 in. (203 mm) apart. Test no. WEAB-3 was not considered since the anchor failed at a localized minimum area of the coupler screw. Therefore, the failure load was not based on the cross-sectional area of a no. 5 (metric no. 16) reinforcing bar. For the single anchor tests, the primary failure mode was steel rupture, but it was evident that the bond failure mode had begun. The true strength of the bond failure mode would have been higher than the failure load observed. However, there was not a reduction in the average force per anchor when comparing single no. 5 (metric no. 16) and double no. 5 (metric no. 16) reinforcing bars spaced 8 in. (203 mm) apart.
For the no. 6 (metric no. 19) epoxy-coated reinforcing bar tension tests, the average pullout loads were 42.8 kips (190.2 kN) and 68.1 kips (303.1 kN), respectively for single and duel anchors spaced 8 in. (203 mm) apart. This suggests that there is a 20 percent decrease in capacity for groups of anchors spaced 8 in. (203 mm) apart. Unlike the test with the no. 5 (metric no. 16) reinforcing bars, the failure mode for no. 6 (metric no. 19) reinforcing bars consisted of the pullout of the adhesive core accompanied by a small concrete cone breakout. The steel failure mode is more desirable than the bond failure mode in bridge rail applications because it will limit the damage to the bridge deck. Also, fracturing of the steel is more ductile and will allow for increased energy absorption of the barrier upon impact.

The shear reinforcing bar tests confirmed that the steel failure mode would control with no. 5 and 6 (metric no. 16 and 19) bars with at least 5 ¼ in. (133 mm) embedment and located sufficiently far away from an edge to prevent concrete breakout.

It was also evident that the protective epoxy coating of the anchors affected the ultimate bond strength. The average dynamic pullout load from uncoated no. 6 (metric no. 19) reinforcing bars was 47.2 kips (210.1 kN) while the average dynamic pullout load from ASTM A775 epoxy coated no. 6 (metric no. 19) reinforcing bars was 42.8 kips (190.2). Therefore, approximately a 9 percent decrease in bond strength was observed when the reinforcing bars had a protective epoxy coating according to ASTM A775 standards.

In order to allow for an alternative anchorage design for tie-down F-shape temporary concrete barrier developed by MwRSF, the epoxy anchorage needed to be able to develop the nominal ultimate strength of the 1 1/8 in. (29 mm) diameter ASTM A307
threaded rod anchors. The ultimate strengths of the A307 rods were determined from simple principles of mechanics of materials. The ultimate stress ($\sigma_u$) of the A307 rods was specified to be 60 ksi (414 MPa) and the cross-sectional area ($A$) for a 1 1/8 in. (29 mm) is 0.763 in.$^2$ (492 mm$^2$). The equations used to calculate the ultimate tension ($P_u$) and shear capacities ($V_u$) are shown in Equations (67) and (68). Note that the shear capacity was calculated using Von Mises criteria.

$$P_u = \sigma_u A$$

(67)

$$V_u = \frac{\sigma_u A}{\sqrt{3}}$$

(68)

The ultimate tension and shear capacities were calculated to be 45.9 kips (203.6 kN) and 26.4 kips (117.6 kN), respectively. The average ultimate tension and shear loads observed from the dynamic testing program of the 1 1/8 in. (29 mm) A307 rods were 45.3 kips (201.5 kN) and a minimum of 40.6 kips (180.8 kN), respectively. The failure mode in tension consisted of a pullout of the adhesive core accompanied by 2 ¾ in. (70 mm) deep concrete cone breakout. The ultimate shear value is an estimated minimum value because the anchor did not fail in the test, the load was governed by the equipment. Nonetheless, the ultimate shear capacity was determined to be far greater that the nominal shear capacity of the anchor and the ultimate tension capacity was within one percent of the nominal tension capacity. Therefore, the anchorage design with 5 ¼ in. (133 mm) embedment depth utilizing the Hilti HIT-RE 500-SD epoxy adhesive was considered an adequate alternative anchorage design for the 1 1/8 in. (29 mm) A307 rods used in the tie-down temporary concrete barrier developed by MwRSF because the tested capacities
met the nominal capacities of the anchorages used in the full-scale crash test. However, the failure in the tension test created significant damage to the deck.

Table 9. Dynamic Bogie Testing Summary

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Bar Coating</th>
<th>Bar Size, US (Metric)</th>
<th>Spacing</th>
<th>Bogie Speed, mph (km/h)</th>
<th>Max Anchor Load EDR-3, kips (kN)</th>
<th>Max Anchor Load DTS 1, kips (kN)</th>
<th>Max Anchor Load DTS 2, kips (kN)</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-1</td>
<td>Tensile</td>
<td>None</td>
<td>#5 (#16)</td>
<td>Single</td>
<td>9.78 (15.74)</td>
<td>38.80 (172.60)</td>
<td>37.91 (168.62)</td>
<td>Anchor fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-2</td>
<td>Tensile</td>
<td>None</td>
<td>#5 (#16)</td>
<td>Single</td>
<td>10.40 (16.74)</td>
<td>39.83 (177.19)</td>
<td>38.94 (173.20)</td>
<td>Anchor fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-3</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#5 (#16)</td>
<td>Single</td>
<td>9.47 (15.24)</td>
<td>35.12 (156.23)</td>
<td>34.86 (155.07)</td>
<td>Anchor fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-4</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#5 (#16)</td>
<td>Single</td>
<td>8.86 (14.26)</td>
<td>36.83 (163.83)</td>
<td>35.07 (155.98)</td>
<td>Anchor fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-5</td>
<td>Shear</td>
<td>Epoxy</td>
<td>#5 (#16)</td>
<td>Single</td>
<td>9.64 (15.51)</td>
<td>25.72 (114.43)</td>
<td>32.38 (144.02)</td>
<td>Anchor fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-6</td>
<td>Shear</td>
<td>Epoxy</td>
<td>#5 (#16)</td>
<td>Single</td>
<td>9.71 (15.62)</td>
<td>23.73 (105.57)</td>
<td>28.41 (126.39)</td>
<td>Anchor fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-7</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#5 (#16)</td>
<td>2 @ 8 in. (2 @ 203 mm)</td>
<td>16.64 (26.78)</td>
<td>73.80 (328.30)</td>
<td>73.80 (328.28)</td>
<td>Anchor fracture/ core pullout</td>
<td></td>
</tr>
<tr>
<td>WEAB-8</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#5 (#16)</td>
<td>2 @ 8 in. (2 @ 203 mm)</td>
<td>14.05 (22.61)</td>
<td>72.64 (323.14)</td>
<td>72.41 (322.10)</td>
<td>Core pullout</td>
<td></td>
</tr>
<tr>
<td>WEAB-9</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#6 (#19)</td>
<td>Single</td>
<td>14.23 (22.91)</td>
<td>40.99 (182.34)</td>
<td>41.63 (185.18)</td>
<td>Core pullout</td>
<td></td>
</tr>
<tr>
<td>WEAB-10</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#6 (#19)</td>
<td>Single</td>
<td>15.73 (25.31)</td>
<td>42.69 (189.90)</td>
<td>44.16 (196.45)</td>
<td>Core pullout</td>
<td></td>
</tr>
<tr>
<td>WEAB-11</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#6 (#19)</td>
<td>2 @ 8 in. (2 @ 203 mm)</td>
<td>15.11 (24.32)</td>
<td>60.88 (270.80)</td>
<td>60.48 (269.05)</td>
<td>Core pullout</td>
<td></td>
</tr>
<tr>
<td>WEAB-12</td>
<td>Tensile</td>
<td>Epoxy</td>
<td>#6 (#19)</td>
<td>2 @ 8 in. (2 @ 203 mm)</td>
<td>15.08 (24.26)</td>
<td>75.66 (336.55)</td>
<td>75.74 (336.90)</td>
<td>Core pullout</td>
<td></td>
</tr>
<tr>
<td>WEAB-13</td>
<td>Shear</td>
<td>Epoxy</td>
<td>#6 (#19)</td>
<td>Single</td>
<td>9.98 (16.07)</td>
<td>32.13 (142.90)</td>
<td>29.64 (131.87)</td>
<td>Anchor fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-14</td>
<td>Tensile</td>
<td>None</td>
<td>1 1/8 in. (29 mm)</td>
<td>Single</td>
<td>15.19 (24.45)</td>
<td>43.73 (194.51)</td>
<td>46.71 (207.77)</td>
<td>45.47 (202.28)</td>
<td>Anchor fracture</td>
</tr>
<tr>
<td>WEAB-15</td>
<td>Shear</td>
<td>None</td>
<td>1 1/8 in. (29 mm)</td>
<td>Single</td>
<td>9.28 (14.93)</td>
<td>43.65 (194.15)</td>
<td>39.06 (173.76)</td>
<td>Bogie head fracture</td>
<td></td>
</tr>
<tr>
<td>WEAB-16</td>
<td>Tensile</td>
<td>None</td>
<td>#6 (#19)</td>
<td>Single</td>
<td>15.90 (25.58)</td>
<td>49.56 (220.43)</td>
<td>47.02 (209.15)</td>
<td>Core pullout</td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 6 - EPOXY ANCHOR STATIC TESTING

6.1 Purpose

The purpose of the static tension test was to determine the relationship of the static pullout capacity to the dynamic pullout capacity. Static bond strength data was available from the manufacturer’s published specifications. However, by conducting a static test in the same concrete slab and utilizing similar testing methods, a more accurate comparison could be obtained. Also, data from the manufacturer’s published specifications was not the average true strength values, but were based on the 5 percent fractile strengths as required by ICC-ES AC308 [4]. Further, the epoxy manufacturer could also impose safety factors to ensure an increase in reliability.

6.2 Scope

The conditions for the static testing (i.e. concrete slab, epoxy adhesive, bar size, test jig) for the static test were similar to the dynamic bogie test no. WEAB-16 in order to minimize the effects of other variables affecting the test results and to get an accurate comparison of load capacity based on loading rate.

6.3 Test Setup

The static test utilized an uncoated, deformed no. 6 (metric no. 19), ASTM A615 grade 60 steel reinforcing bar that was embedded 5 ¼ in. (133 mm) into an unreinforced concrete slab and bonded by the Hilti HIT-RE 500-SD epoxy adhesive. The anchor hole was constructed using a carbide-tipped concrete bit and a rotary hammer drill. The concrete slab had an average unconfined compressive strength of 6,454 psi (44.50 MPa), as determined from concrete cylinder testing. Material strength specification sheets are shown in Appendix H.
The tensile jig used in the dynamic bogie testing was modified by cutting a hole in the web of the W-beam, which allowed a chain to be attached to the jig. Two load cells assembled in series were then connected to the chain that was attached to the test jig on one end and a hydraulic ram on the other. The hydraulic ram was supported by wood blocking at approximately the chain mounting height to ensure a perpendicular connection to the test jig. The rear end of the hydraulic ram was then secured to a rigid anchor which was bolted to the concrete slab. The test setup drawing for the static tensile test is shown in Figure 77. Detailed drawings are shown in Appendix G.

6.4 Test Facility

The testing facility is located at the Lincoln Air Park on the northwest side of the Lincoln Municipal Airport and is approximately 5 miles (8.0 km) northwest of the University of Nebraska-Lincoln.

6.5 Equipment and Instrumentation

6.5.1 Load Cells

Two load cells were placed in series with the test apparatus to measure the force exerted on the test jig until failure of the anchor. The load cells were placed in tension between the test jig and the hydraulic ram and had a maximum capacity of 50 kips (222 kN).
Epoxy Anchor Rebar Static Testing—Tensile Test Setup
1. Hydraulic Ram
2. One high-speed digital camera perpendicular
3. One high-speed digital camera perpendicular zoomed in on the tensile test jig and rebar.
4. JVC digital video
5. Two Load Cells
6. Note any installation issues or problems in fieldbook.
7. Note failure mode in fieldbook.
8. Each test must be at least 2" from any previous test anchor holes to prevent anchor spacing and edge spacing issues from affecting results.
9. Kick plate anchors must be replaced each test.
10. Before installation of anchors, measure the bar deformation rib height.
11. Anchors must be installed while the concrete temperature remains above 45° during curing time for the epoxy. If required, heat the concrete to the desired temperature.
12. Anchors are to be installed in same concrete slabs as used in test nos. WEAB 1–15.
13. Epoxy used for fourth round of testing will be Hilti HIT-RE 500-SD.

---

Figure 77. Tension Test Setup, Test No. WEAB-17
6.5.2 Hydraulic Ram

The hydraulic ram model used was the SAE-9436 manufactured by Prince Manufacturing Corporation of North Sioux City, South Dakota. It had a 36 in. (914 mm) stroke and a 4 in. (102 mm) diameter bore. An external pump was used to push hydraulic fluid into the hydraulic cylinder.

6.5.3 Test Jig

The tensile test jig used in the dynamic bogie testing was modified by cutting a hole in the web of the W-beam to allow a chain to be attached. The center of the hole had a mounting height of approximately 24 ½ in. (622 mm) from the concrete slab surface. For more details on the design of the tensile test jig refer to Section 4.4.2 or Appendix F. A picture of the test setup is shown in Figure 78.

![Figure 78. Static Test Setup](image)

6.5.4 Digital Cameras

Two AOS VITcam high-speed digital video cameras and one JVC digital video camera were used to document the test. The AOS high-speed camera had a frame rate of 120 frames per second and the JVC digital video camera had a frame rate of 29.97 frames per second. All the cameras were placed laterally from the test jig, with a view
perpendicular to the hydraulic ram’s direction of travel. A Nikon D50 digital still camera was also used to document pre- and post-test conditions for all tests.

6.5.5 Data Processing

As computed in the dynamic bogie tests, the anchor force for the tensile test was determined by summing the moments about the reaction point of the test jig and solving for the anchor force. However, the applied load height for the static testing was 24 ½ in. (622 mm). For details about the calculation of the anchor force from the applied force to the test jig, refer to Section 4.4.6.
CHAPTER 7 - STATIC TESTING RESULTS AND DISCUSSION

7.1 Results

7.1.1 Test No. WEAB-17

For test no. WEAB-17, the hydraulic ram applied an increasing load for approximately 18 seconds until failure of the anchor. The anchor pulled out of the concrete hole and most of the epoxy adhesive was still attached to the reinforcing bar. A concrete cone of approximately 10 to 12 in. (254 to 305 mm) in diameter by 2 in. (51 mm) deep broke out and was still attached to the reinforcing bar. The maximum tensile load observed was 45.2 kips (201.1 kN) according to load cell no. 1 data and 43.7 kips (194.3 kN) according to load cell no. 2. Pre- and post-test photographs are shown in Figure 79. A plot of the force versus time history is shown in Figure 80. Sequential photographs are shown in Figure 81. A picture of the pulled out anchor is shown in Figure 82.

![Pre-Test and Post-Test Photographs](image_url)

Figure 79. Pre- and Post-Test Photographs, Test No. WEAB-17
Figure 80. Force vs. Time, Test No. WEAB-17

![Graph showing force vs. time for Test No. WEAB-17.](image)

Figure 81. Sequential Photographs, Test No. WEAB-17

0.000 sec

17.950 sec

17.983 sec

18.100 sec
Figure 82. Post-Test Anchor Photograph, Test No. WEAB-17
7.2 Discussion

The average static pullout strength from the two load cell recordings from test no. WEAB-17 was 44.5 kips (198 kN), which corresponded to a static bond strength value of 3.59 ksi (24.77 MPa) according to the uniform bond stress model. The average dynamic pullout strength from processed accelerometer data from test no. WEAB-16 was 47.2 kips (210 kN), which corresponded to a dynamic bond strength value of 3.82 ksi (26.33 MPa). The dynamic increase factor for the bond strength failure mode was found to be 1.06, as calculated by the dynamic capacity divided by the static capacity.

It should be noted that this dynamic increase factor was far lower than values recommended by existing literature. Berra and Solomos reported that the dynamic capacity of post-installed anchors to range from 1.59 to 2.39 times as high as those predicted from static loading conditions and that a dynamic increase factor of 1.25, as permitted in ACI 349-97, was reasonable for chemical adhesive anchors [16]. However, the static values used in that study were based on predictive equations from ACI and the CCD method and not true analysis or actual test data. A dynamic increase factor of 1.2 was suggested by Braimah, Concontestabile and Guilbeault [17].

The bond strength published by Hilti for uncracked concrete was listed as 2.07 ksi (14.3 MPa) for no. 6 (metric no. 19) reinforcing bars [19]. Therefore, from the test data available, the bond strength determined from the static test program was 74 percent higher than the published static bond strength listed in the manufacturer’s product specification.
CHAPTER 8 - FINAL MODEL DEVELOPMENT

8.1 Tension Model

8.1.1 Steel Model

The equation to predict the steel strength of an anchor in tension provided accurate results by adding a dynamic increase factor. From tests WEAB-1 through WEAB-4, it was found that a dynamic increase factor of 1.15 for the steel failure mode provided a reasonable correlation from the dynamic test data to the static strength obtained from material certifications. Equation (69) is suggested to be used to estimate the strength of the anchor for the steel failure mode, where $\phi$ is the strength reduction factor, $A_{se,N}$ is the effective cross-sectional area of the anchor in tension, $f_{uta}$ is the ultimate tensile stress of the anchor, and $\psi_{sd,N}$ is the dynamic increase factor for the steel failure mode in tension. The strength reduction factors for the steel failure mode are given in Appendix D of ACI 318-11.

$$N_{sd} = \phi A_{se,N} f_{uta} \psi_{sd,N}$$

(69)

Where:

$$\psi_{sd,N} = \begin{cases} 
1.00 & \text{for static loading} \\
1.15 & \text{for dynamic loading} 
\end{cases}$$

8.1.2 Bond Model

A limit states design method of a full concrete cone or full uniform bond model accurately predicted the failure strengths of adhesive anchors loaded in tension from previous analysis in this study. This model was expanded upon after static and dynamic testing from this study. From testing, the true dynamic increase factor, epoxy coating reduction factor and limited data for the effects of anchor spacing were determined. For
the bond failure mode, the dynamic increase factor was found to be 1.06, as a ratio of the
dynamic to static pullout bond strengths. The effect of epoxy coating on an anchor
relative to plain black steel anchors was found to result in a reduction of 9 percent. To be
conservative, a 10 percent reduction was used in the suggested model. Test data for no. 5
(metric no. 16) reinforcing bars showed that there was not a reduction when the anchors
were spaced 8 in. (203 mm) apart. However, a reduction of approximately 20 percent was
found when no. 6 (metric no. 16) reinforcing bars were spaced 8 in. (203 mm) apart. With
these factors in mind, the equation for estimating the dynamic bond strength of an
adhesive anchor in tension \((N_{a,d})\) is shown in Equation (70), where \(\phi\) is the strength
reduction factor, \(\tau\) is the static bond stress, \(d_a\) is the diameter of the anchor, \(h_{ef}\) is the
embedment depth of the anchor, \(\psi_{ad,N}\) is the dynamic increase factor in tension for the
adhesive bond failure mode, \(\psi_{c,N}\) is the anchor coating factor, and \(\psi_{s,N}\) is the spacing
factor. The strength reduction factors for pullout are given in Appendix D of ACI 318-11.
Alternatively, the dynamic bond strength can be used with the dynamic increase factor set
equal to unity.

\[
N_{a,d} = \phi \tau \pi d_a h_{ef} \psi_{ad,N} \psi_{c,N} \psi_{s,N}
\]  

(70)

Where:

\[
\psi_{ad,N} = \begin{cases} 
1.00 & \text{for static loading} \\
1.06 & \text{for dynamic loading} 
\end{cases}
\]

\[
\psi_{c,N} = \begin{cases} 
1.00 & \text{for black steel rods} \\
0.90 & \text{for epoxy coated rods} 
\end{cases}
\]

\[
\psi_{s,N} = \begin{cases} 
1.00 & \text{for single anchors} \\
1.00 & \text{for no. 5 (metric no. 16) anchors spaced at 8 in. (203 mm)} \\
0.80 & \text{for no. 6 (metric no. 19) anchors spaced at 8 in. (203 mm)} 
\end{cases}
\]
8.1.3 Concrete Model

Due to the limitations of this study, the concrete breakout model was not able to be studied. Therefore, the static concrete breakout model from Section D.5.2 of ACI 318-11 is suggested. The equation for the basic concrete breakout strength of a single anchor in tension is shown in Equation (71), where $\phi$ is the strength reduction factor, $k_c$ is a coefficient that is equal to 17 for post-installed anchors, $\lambda_a$ is a modification factor for lightweight concrete, $f'_c$ is the unconfined compressive strength of the concrete, and $h_{ef}$ is the embedment depth of the anchor. For anchor groups, a ratio of the projected area for an anchor group to the projected area for a single anchor is used to compensate for the close proximity of anchors. The strength reduction factors for concrete breakout are given in Appendix D of ACI 318-11.

$$N_b = \phi k_c \lambda_a \sqrt{f'_c h_{ef}^{1.5}}$$

However, it is believed that the concrete breakout model from ACI is quite conservative based on testing from this study. A more detailed study of this failure mode is recommended in further research.

8.2 Shear Models

8.2.1 Steel Model

The equation to predict the steel strength of an anchor in shear provided accurate results by adding a dynamic increase factor. From tests WEAB-5, WEAB-6, and WEAB-13, it was found that a dynamic increase factor of 1.15 for the steel failure mode provided a reasonable correlation from the dynamic test data to the static strength obtained from material certifications. Equation (72) is suggested to be used to estimate the strength of the anchor for the steel failure mode, $\phi$ is the strength reduction factor, $A_{se,v}$ is the
effective cross-sectional area of the anchor in shear, \( f_{uta} \) is the ultimate tensile stress of the anchor, and \( \psi_{sd,y} \) is the dynamic increase factor for the steel failure mode in shear. The strength reduction factors for the steel failure mode are given in Appendix D of ACI 318-11.

\[
N_{sd} = \phi(0.6)A_{se,y}f_{uta}\psi_{sd,y}
\]  

(72)

Where:

\[
\psi_{sd,y} = \begin{cases} 
1.00 & \text{for static loading} \\
1.15 & \text{for dynamic loading} 
\end{cases}
\]

**8.2.2 Concrete Model**

A concrete failure mode was not able to be forced for any of the anchorages tested in this study for the embedment depths of 5 1/4 in. (133 mm). Therefore, no firm conclusions could be made about estimating the capacity of this failure mode. However, it is believed that this mode will not control as long as sufficient edge distance is provided. Until further research can be conducted on this model, the current procedure in Section D.6.2 is recommended. However, previous testing has shown that epoxy anchors installed into a reinforced concrete slab with an embedment depth of 5 1/4 in. (133 mm) and an edge distance of 11 in. (279 mm) did not break out of the concrete slab [45].

**8.3 Model Comparison to Test Data**

A generalized summary of the anchor strengths tested in this study is shown in Table 10. Bond strengths calculated from the testing program in this study are shown in Table 11. The models presented in Sections 8.1 and 8.2 were compared to the actual observed loads from the dynamic testing program in this study. However, the concrete failure modes were not considered because the dynamic information of these failure
modes could not be determined since a concrete failure was not observed in testing. A
test-to-predicted ratio was calculated as the average actual test load divided by the
predicted load. For the bond models, the bond stress was taken from ICC-ES test data
[74]. A correction factor of 1.74 was applied to compensate for the variation of the static
bond stress calculated in this study to the bond stress from ICC-ES. This was done for
comparison purposes of this study only. The suggested model does not allow for this
because of uncertainty that this increase can always be applied. A summary of the model
comparison to test data is shown in Table 12. The suggested dynamic increase factors,
epoxy coating factor, and spacing factor allowed the predicted loads from the model to
correspond well with loads from the dynamic testing program. Test no. WEAB-15 was
not included in Table 12 because it was governed by equipment failure.

Table 10. Anchor Load Summary

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Anchor Spacing</th>
<th>Loading Type</th>
<th>Anchor Coating</th>
<th>Average Tensile Load Per Anchor, kips (kN)</th>
<th>Average Shear Load Per Anchor, kips (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#5</td>
<td>Single</td>
<td>Dynamic</td>
<td>None</td>
<td>38.87 (172.91)</td>
<td>-</td>
</tr>
<tr>
<td>#5</td>
<td>Single</td>
<td>Dynamic</td>
<td>Epoxy</td>
<td>35.47 (157.78)</td>
<td>27.56 (122.60)</td>
</tr>
<tr>
<td>#5</td>
<td>2 @ 8 in.</td>
<td>Dynamic</td>
<td>Epoxy</td>
<td>36.58 (162.73)</td>
<td>-</td>
</tr>
<tr>
<td>#6</td>
<td>Single</td>
<td>Dynamic</td>
<td>Epoxy</td>
<td>42.61 (189.55)</td>
<td>30.58 (136.03)</td>
</tr>
<tr>
<td>#6</td>
<td>Single</td>
<td>Dynamic</td>
<td>None</td>
<td>47.24 (210.14)</td>
<td>-</td>
</tr>
<tr>
<td>#6</td>
<td>Single</td>
<td>Static</td>
<td>None</td>
<td>44.45 (197.71)</td>
<td>-</td>
</tr>
<tr>
<td>#6</td>
<td>2 @ 8 in.</td>
<td>Dynamic</td>
<td>Epoxy</td>
<td>34.08 (151.62)</td>
<td>-</td>
</tr>
<tr>
<td>1 1/8 in.</td>
<td>Single</td>
<td>Dynamic</td>
<td>None</td>
<td>45.30 (201.50)</td>
<td>40.63* (180.75)*</td>
</tr>
</tbody>
</table>

*Load governed by equipment failure, actual capacity is estimated to be higher
Table 11. Tensile Bond Strengths

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Anchor Spacing</th>
<th>Loading Type</th>
<th>Anchor Coating</th>
<th>Bond Stress, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#6</td>
<td>Single</td>
<td>Dynamic</td>
<td>Epoxy</td>
<td>3.44 (23.75)</td>
</tr>
<tr>
<td>#6</td>
<td>Single</td>
<td>Dynamic</td>
<td>None</td>
<td>3.82 (26.33)</td>
</tr>
<tr>
<td>#6</td>
<td>Single</td>
<td>Static</td>
<td>None</td>
<td>3.59 (24.77)</td>
</tr>
<tr>
<td>#6 2 @ 8 in.</td>
<td>Dynamic</td>
<td>Epoxy</td>
<td>2.76 (19.00)</td>
<td></td>
</tr>
<tr>
<td>1 1/8 in.</td>
<td>Single</td>
<td>Dynamic</td>
<td>None</td>
<td>2.44 (16.83)</td>
</tr>
</tbody>
</table>

8.4 Critical Embedment Depth to Ensure Steel Failure Mode

Deck damage from concrete breakout of the anchors is not desired because of the time and cost of repair. Further, in order to utilize the yield line analysis procedure in AASHTO, the ultimate strength of the anchorage needs to be developed to prevent a yield line at the railing/deck interface. Therefore, it is required to provide enough embedment of the anchors to ensure a steel failure.
Table 12. Model Comparison to Test Data

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Description</th>
<th>Predicted Steel Load, kips</th>
<th>Predicted Bond Load, kips</th>
<th>Actual Failure Mode</th>
<th>Actual Load, kips</th>
<th>Test-to-Predicted Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-1</td>
<td>#5, uncoated, single tension</td>
<td>$0.31 \times 1.15 \times 103.937$</td>
<td>$37.05$</td>
<td>$=2.145 \times P(1) \times (5/8) \times 5.25 \times 1.06 \times 1.74$</td>
<td>$40.78$</td>
<td>Steel rupture</td>
</tr>
<tr>
<td>WEAB-2</td>
<td>#5, uncoated, single tension</td>
<td>$0.31 \times 1.15 \times 103.937$</td>
<td>$37.05$</td>
<td>$=2.145 \times P(1) \times (5/8) \times 5.25 \times 1.06 \times 1.74$</td>
<td>$40.78$</td>
<td>Steel rupture</td>
</tr>
<tr>
<td>WEAB-3</td>
<td>#5, epoxy coated, single tension</td>
<td>$0.31 \times 1.15 \times 103.937$</td>
<td>$37.05$</td>
<td>$=2.145 \times P(1) \times (5/8) \times 5.25 \times 1.06 \times 0.9$</td>
<td>$36.70$</td>
<td>Steel rupture</td>
</tr>
<tr>
<td>WEAB-4</td>
<td>#5, epoxy coated, single tension</td>
<td>$0.31 \times 1.15 \times 103.937$</td>
<td>$37.05$</td>
<td>$=2.145 \times P(1) \times (5/8) \times 5.25 \times 1.06 \times 1.74 \times 0.9$</td>
<td>$36.70$</td>
<td>Steel rupture</td>
</tr>
<tr>
<td>WEAB-5</td>
<td>#5, epoxy coated, single shear</td>
<td>$0.31 \times 0.6 \times 103.937 \times 1.15$</td>
<td>$22.23$</td>
<td></td>
<td></td>
<td>Shear fracture</td>
</tr>
<tr>
<td>WEAB-6</td>
<td>#5, epoxy coated, single shear</td>
<td>$0.31 \times 0.6 \times 103.937 \times 1.15$</td>
<td>$22.23$</td>
<td></td>
<td></td>
<td>Shear fracture</td>
</tr>
<tr>
<td>WEAB-7</td>
<td>#5, epoxy coated, @ 8 in. tension</td>
<td>$2 \times 0.31 \times 1.15 \times 103.937$</td>
<td>$74.11$</td>
<td>$=2 \times 2.145 \times P(1) \times (5/8) \times 5.25 \times 1.06 \times 1.74 \times 0.9$</td>
<td>$73.41$</td>
<td>Steel rupture/anchor pullout</td>
</tr>
<tr>
<td>WEAB-8</td>
<td>#5, epoxy coated, @ 8 in. tension</td>
<td>$2 \times 0.31 \times 1.15 \times 103.937$</td>
<td>$74.11$</td>
<td>$=2 \times 2.145 \times P(1) \times (5/8) \times 5.25 \times 1.06 \times 1.74 \times 0.9$</td>
<td>$73.41$</td>
<td>Anchor pullout</td>
</tr>
<tr>
<td>WEAB-9</td>
<td>#6, epoxy coated, single tension</td>
<td>$0.44 \times 1.15 \times 100.4$</td>
<td>$50.80$</td>
<td>$=2.067 \times P(1) \times 0.75 \times 5.25 \times 1.06 \times 1.74 \times 0.9$</td>
<td>$42.44$</td>
<td>Anchor pullout</td>
</tr>
<tr>
<td>WEAB-10</td>
<td>#6, epoxy coated, single tension</td>
<td>$0.44 \times 1.15 \times 100.4$</td>
<td>$50.80$</td>
<td>$=2.067 \times P(1) \times 0.75 \times 5.25 \times 1.06 \times 1.74 \times 0.9$</td>
<td>$42.44$</td>
<td>Anchor pullout</td>
</tr>
<tr>
<td>WEAB-11</td>
<td>#6, epoxy coated, @ 8 in. tension</td>
<td>$2 \times 0.44 \times 1.15 \times 100.4$</td>
<td>$101.60$</td>
<td>$=2 \times 2.067 \times P(1) \times 0.75 \times 5.25 \times 1.06 \times 1.74 \times 0.8 \times 0.9$</td>
<td>$67.91$</td>
<td>Anchor pullout</td>
</tr>
<tr>
<td>WEAB-12</td>
<td>#6, epoxy coated, @ 8 in. tension</td>
<td>$2 \times 0.44 \times 1.15 \times 100.4$</td>
<td>$101.60$</td>
<td>$=2 \times 2.067 \times P(1) \times 0.75 \times 5.25 \times 1.06 \times 1.74 \times 0.8 \times 0.9$</td>
<td>$67.91$</td>
<td>Anchor pullout</td>
</tr>
<tr>
<td>WEAB-13</td>
<td>#6, epoxy coated, single shear</td>
<td>$1.15 \times 0.6 \times 0.44 \times 100.4$</td>
<td>$30.48$</td>
<td></td>
<td></td>
<td>Shear fracture</td>
</tr>
<tr>
<td>WEAB-14</td>
<td>1 1/8 in., uncoated rod, single tension</td>
<td>$0.763 \times 1.15 \times 60$</td>
<td>$52.65$</td>
<td>$=1.904 \times P(1) \times 1.125 \times 5.25 \times 1.06 \times 1.74$</td>
<td>$65.16$</td>
<td>Anchor pullout/concrete breakout</td>
</tr>
<tr>
<td>WEAB-15</td>
<td>#6, uncoated, single tension</td>
<td>$0.44 \times 1.15 \times 100.4$</td>
<td>$50.80$</td>
<td>$=2.067 \times P(1) \times 0.75 \times 5.25 \times 1.06 \times 1.74$</td>
<td>$47.16$</td>
<td>Anchor pullout/concrete breakout</td>
</tr>
<tr>
<td>WEAB-16</td>
<td>#6, uncoated, single tension (static)</td>
<td>$0.44 \times 100.4$</td>
<td>$44.18$</td>
<td>$=2.067 \times P(1) \times 0.75 \times 5.25 \times 1.74$</td>
<td>$44.49$</td>
<td>Anchor pullout/concrete breakout</td>
</tr>
</tbody>
</table>
CHAPTER 9 - BRIDGE RAIL ANALYSIS

9.1 Purpose

In order to determine the required load needed to be resisted by the anchors of a concrete bridge railing, the impact load for the crash is needed. Currently, the required impact loads are specified in AASHTO’s LRFD Bridge Design Specifications, as described in the literature review. However, these loads are based on NCHRP Report No. 350 crash conditions. With the adoption of MASH, heavier and higher center of gravity vehicles are used that will increase the impact loads on traffic barriers [77]. Therefore, a need exists for predicting the impact loads for MASH vehicles. The required anchorage strength for concrete bridge railings can be determined from AASHTO’s yield line analysis procedure.

9.2 Applied Load Determination – Actual Crash Test Data

9.2.1 Selection of Crash Tests to Analyze

Data from full-scale crash tests was used to estimate the forces impacted to bridge railings for eight concrete barriers and two pinned-down temporary concrete barriers. These crash tests were selected from previous testing to investigate the impact loads of the crash test. A summary of the analyzed crash tests are shown in Table 13.

9.2.2 Calculation Methodology Using Accelerometers

The lateral force imparted to several concrete barriers was estimated using data from the on-board accelerometers, the angular rate transducers, and the overhead videos. The yaw angle was used in conjunction with the initial impact angle of the vehicle to calculate the vehicle angle to the barrier for each time step. The onboard accelerometers recorded accelerations in the longitudinal, lateral, and vertical directions relative to the
vehicles and were mounted near the center of gravity of the vehicles. The longitudinal accelerometer force \( F_{Long} \) and lateral accelerometer forces \( F_{Lat} \) were determined by multiplying the mass of the vehicle by the accelerometer traces from the longitudinal and lateral directions. The vehicle forces were then transformed from the local coordinate system of the vehicle to the global coordinate system of the barrier by using Equations (73) and (74). The free body diagrams of the forces are shown in Figures 83 and 84. Therefore, the barrier force in the longitudinal and lateral directions \( F'_{Long} \) and \( F'_{Lat} \) could be determined for every data point in the accelerometer trace using the angle between the longitudinal axis of the vehicle and the longitudinal axis of the barrier \( \theta \).

\[
F'_{Long} = F_{Long} \cos(\theta) + F_{Lat} \cos(90^\circ - \theta) \tag{73}
\]

\[
F'_{Lat} = F_{Long} \sin(\theta) - F_{Lat} \sin(90^\circ - \theta) \tag{74}
\]
9.2.3 Actual Crash Test Maximum Loads

Using the procedure described in section 9.2.2, a dynamic forcing function could be determined. The general shapes of these functions were discovered to take the shape...
of a “saw tooth” for the initial and tail-slap forces. A graph of a dynamic forcing function, as determined by processing the on-board accelerometer data, of an impact with a 4,442 lb (2,015 kg) pickup truck impacting a vertical concrete parapet at 64.8 mph (104.3 km/h) and 25.5 degrees is shown in Figure 85. The first initial spike in force was due to the initial front-end impact, and the second spike was caused by tail-slap of the rear of the vehicle. A summary of the full-scale crash tests that were processed is shown in Table 14. The maximum 50 ms average forces listed are based on processed accelerometer data.

Figure 85. Forcing Function from Vertical Parapet Crash Test
Table 14. Estimated Lateral Forces from Full-Scale Crash Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Designation</th>
<th>NCHRP 350 or MASH Vehicle</th>
<th>Vehicle Weight/Mass, lb (kg)</th>
<th>Actual Speed, mph (km/h)</th>
<th>Actual Angle, deg</th>
<th>Barrier Type</th>
<th>Barrier Height, in. (mm)</th>
<th>Impact Severity, kip-ft (kJ)</th>
<th>Max 50 ms Ave. Force, kips (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NIT-1</td>
<td>3-11</td>
<td>NCHRP 350 2000P</td>
<td>4,445 (2,016)</td>
<td>62.0 (99.8)</td>
<td>24.8</td>
<td>Open Concrete Rail</td>
<td>29 (737)</td>
<td>100.4 (136.1)</td>
<td>68.6 (305.0)</td>
</tr>
<tr>
<td>CBPP-1</td>
<td>3-11</td>
<td>NCHRP 350 2000P</td>
<td>4,442 (2,015)</td>
<td>64.8 (104.3)</td>
<td>25.5</td>
<td>Vertical Parapet</td>
<td>32 (813)</td>
<td>115.5 (156.5)</td>
<td>68.8 (306.3)</td>
</tr>
<tr>
<td>MOBR-1</td>
<td>3-11</td>
<td>NCHRP 350 2000P</td>
<td>4,443 (2,015)</td>
<td>63.0 (101.4)</td>
<td>25.5</td>
<td>Single Slope Combination Rail</td>
<td>32 (813)</td>
<td>109.3 (148.2)</td>
<td>69.9 (311.1)</td>
</tr>
<tr>
<td>MOBR-2</td>
<td>3-11</td>
<td>NCHRP 350 2000P</td>
<td>4,473 (2,029)</td>
<td>63.8 (102.7)</td>
<td>27.1</td>
<td>Single Slope Combination Rail</td>
<td>32 (813)</td>
<td>125.9 (170.7)</td>
<td>78.6 (349.4)</td>
</tr>
<tr>
<td>ZOI-1</td>
<td>4-12</td>
<td>NCHRP 350 8000S</td>
<td>17,605 (7,985)</td>
<td>48.7 (78.4)</td>
<td>15.8</td>
<td>Single Slope with Luminaire</td>
<td>32 (813)</td>
<td>104.0 (141.0)</td>
<td>69.9 (310.9)</td>
</tr>
<tr>
<td>ZOI-3</td>
<td>4-12</td>
<td>NCHRP 350 8000S</td>
<td>17,637 (8,000)</td>
<td>50.2 (80.8)</td>
<td>16.4</td>
<td>Single Slope with Luminaire</td>
<td>32 (813)</td>
<td>118.5 (160.7)</td>
<td>78.4 (348.6)</td>
</tr>
<tr>
<td>CYRO-1</td>
<td>4-12</td>
<td>NCHRP 350 8000S</td>
<td>17,840 (8,092)</td>
<td>51.2 (82.4)</td>
<td>17.7</td>
<td>Single Slope with 19.08 ft (5.82 m) Tall Sound Wall</td>
<td>32 (813)</td>
<td>144.4 (195.8)</td>
<td>72.9* (324.4)</td>
</tr>
<tr>
<td>2214NJ-2</td>
<td>4-12</td>
<td>MASH 10000S</td>
<td>22,045 (9,999)</td>
<td>56.5 (90.9)</td>
<td>15.7</td>
<td>New Jersey Rail</td>
<td>32 (813)</td>
<td>171.2 (232.1)</td>
<td>117.6 (523.2)</td>
</tr>
<tr>
<td>KTB-1</td>
<td>3-11</td>
<td>NCHRP 350 2000P</td>
<td>4,448 (2,018)</td>
<td>62.0 (99.8)</td>
<td>25.3</td>
<td>Temporary F-Shape Bolted to Deck</td>
<td>32 (813)</td>
<td>104.3 (141.4)</td>
<td>62.7 (278.8)</td>
</tr>
<tr>
<td>NYTNCB-4</td>
<td>3-11</td>
<td>MASH 2270P</td>
<td>5,002 (2,269)</td>
<td>62.3 (100.2)</td>
<td>25.7</td>
<td>Temporary NJ-Shape Pinned to Deck</td>
<td>32 (813)</td>
<td>121.6 (164.9)</td>
<td>61.1 (271.6)</td>
</tr>
</tbody>
</table>

*Peak force due to impact with sound wall was ignored
9.2.4 Estimation of Inertial Forces

A process for estimating the inertial load of a crash test was developed using the barrier deflection versus time data that was extracted from the overhead view high-speed videos. Therefore, the initial maximum deflection and the time at which that deflection occurred could be determined. Utilizing Newton’s Second Law, the inertial force could be estimated by multiplying the mass of the accelerated portion of the barrier by the acceleration of the barrier.

A linear function of the form \( a(t) = kt \) was estimated to represent the acceleration of the barrier up to the maximum deflection, with \( k \) equal to an acceleration constant determined from the barrier deflection versus time data and \( t \) representing time. In order to obtain the relationship of the acceleration function to the deflection versus time data, the acceleration function needed to be integrated twice. The velocity function, \( v(t) \), was determined by integration of the acceleration function with respect to time, and the deflection function, \( s(t) \), was determined by integration of the velocity function with respect to time. The derivation of the deflection versus time function, \( s(t) \), is shown in Equations (75) through (82). Variables with zero subscripts are the initial conditions.

\[
v(t) = \int a(t) dt = \int ktdt
\]  
\[
v(t) = \frac{1}{2} kt^2 + c_1
\]  
\[
v(0) = v_0, \therefore c_1 = v_0
\]  
\[
v(t) = \frac{1}{2} kt^2 + v_0
\]
\[ s(t) = \int v(t) dt = \int \frac{1}{2} kt^2 + v_0 dt \]  
\[ s(t) = \frac{1}{6} kt^3 + v_0 t + c_2 \]  
\[ s(0) = s_0, \therefore c_2 = s_0 \]  
\[ s(t) = \frac{1}{6} kt^3 + v_0 t + s_0 \]  

The initial velocity of the barrier \((v_0)\) and the initial displacement of the barrier \((s_0)\) were both zero before impact. From the video analysis, the initial maximum deflection of the barrier and the time at which that occurred could be determined. The acceleration constant \((k)\) could be determined by solving Equation (82) with the initial maximum deflection and the time at which that occurred. The acceleration function could be reflected at the point of maximum acceleration to obtain a “saw tooth” shape.

The acceleration function could be multiplied by the estimated mass of the portion of the barrier that was accelerated to obtain a force versus time plot. This data could then be processed to a 50 ms average force by averaging the data points on 50 ms intervals.

A great uncertainty existed in estimating the mass of the portion of the barrier that was accelerated. Further, the video analysis only measured the lateral deflection of the barrier and didn’t measure the rotation. Due to the very high uncertainty of unknown parameters in this model, an accurate estimation for the inertial forces could not be determined. However, it was considered conservative to estimate the load imparted to bridge railings as the full load from processed from accelerometer data.
9.3 Applied Load Determination – Predictive Crash Loads

There are several different ways to analyze the lateral loads that bridge railings need to resist. In the past, loads were recorded from full-scale crash tests into an instrumented wall. In recent years, the design vehicle fleet has changed with the adoption of MASH. There has been little research conducted to determine the load requirements for these bigger, higher center of gravity vehicles. A few load calculation methods were explored to determine an accurate model for estimating the lateral loads on bridge railing and compared to data obtained from full-scale crash tests.

9.3.1 Current AASHTO Loads

The design forces according the AASHTO LRFD code were based on the nominal impact conditions as there was no method for deriving an impact force based on the actual crash conditions. The discrete AASHTO model was converted to a continuous load versus impact severity graph by determining the impact severity of each test level and the specified lateral load. An entire NCHRP Report No. 350 test matrix was constructed and the impact severities for each test were calculated using Equation (88), as described in the subsequent sections. For each test level, the maximum impact severity was matched with the lateral load obtained from AASHTO (Table 4). A summary of the impact severity calculations and the AASHTO lateral forces is shown in Table 15. Each test level was considered a data point and the data points were connected to form a continuous, piece-wise linear function.
9.3.2 Impulse-Momentum Principles

The basic principles of impulse-momentum (IM) can also be utilized to estimate the lateral loads on bridge railings. The actual force of an impact will be somewhere between the force from a perfectly plastic impact (coefficient of restitution \((e) = 0\)) to a perfectly elastic impact (coefficient of restitution \((e) = 1\)) in the lateral direction. Theoretically, for a perfectly plastic impact, the lateral velocity of the vehicle relative to the longitudinal...
axis of the barrier will be equal to zero after the impact. Conversely, for a perfectly elastic impact, the lateral velocity of the vehicle relative to the longitudinal axis of the barrier will be equal and opposite to the initial lateral impact velocity. Therefore, a conservative estimate of the maximum lateral force can be determined by a perfectly elastic impact.

The impulse of the impact \( I \) is shown by Equation (83), assuming the x-direction is perpendicular to the longitudinal direction of the barrier. The integration of the force with respect to time is equal to the change in momentum, where \( m \) is the mass of the vehicle, \( V_{l,x} \) is the initial lateral velocity, and \( V_{F,x} \) is the final lateral velocity. The average lateral force \( (F_{lat,ave}) \) can be solved for by estimating the time for the duration of the impact (Equation (84)). This force is greatly dependent on the time interval of the impact \( (\Delta t) \), which greatly affects the accuracy of this model. In lieu of a better analysis, the time interval of the impact can be estimated by Equation (85), as described in the next section.

\[
I = \int_0^t F_x dt = m(V_{l,x} - V_{F,x}) \tag{83}
\]

\[
F_{lat,ave} = \frac{m(V_{l,x} - V_{F,x})}{\Delta t} \tag{84}
\]

9.3.3 NCHRP Report No. 86

The NCHRP Report No. 86 contains an approximate, mathematical method for estimating the forces on longitudinal barriers and is essentially a more specific form of the impulse-momentum model. This method requires simple input parameters of the type of barrier (flexible or rigid), the dimensions of the vehicle, the location of the center of
mass of the vehicle, the impact speed, the impact angle, and the coefficient of friction between the vehicle and the barrier [85]. Several pickup trucks and SUTs were reviewed based on measurements from previous crash tests to determine proper input values for these equations. The average of the values found were taken as input values for the model. A diagram of the parameters involved in this model are shown in Figure 86. The vehicle dimensions for NCHRP Report No. 350 and MASH are shown in Table 16 [44, 78, 79, 80, 81, 82, 86, 87, 88, 89].

![Diagram of Input Values for NCHRP Report No. 86 Model](image)

Figure 86. Diagram of Input Values for NCHRP Report No. 86 Model [85]

The time interval of the impact ($\Delta t$) can be calculated by Equation (85) and is a function of the distance from the front of the vehicle to the center of mass ($AL$), half the width of the vehicle ($B$), the lateral displacement of the barrier ($D$), the impact velocity ($V_I$), and the impact angle ($\theta$). The lateral deceleration in g’s ($G_{lat}$) can then be calculated by Equation (86), where $g$ is the acceleration due to gravity. Finally, the average lateral
impact force \( F_{lat,ave} \) can be calculated by multiplying the weight of the vehicle \( W \) by the lateral deceleration in \( g \)’s (Equation (87)). The dynamic magnification factor \((DMF)\) accounts for impact loading and usually ranges from \( \pi/2 \) to 2. For the analysis in this study, the dynamic magnification factor was conservatively estimated to be equal to 2.

\[
\Delta t = \frac{AL \sin(\theta) - B[1 - \cos(\theta)] + D}{\frac{1}{2}[V_f \sin(\theta)]}
\]

\[
G_{lat} = \frac{V_f \sin(\theta)}{\Delta t \cdot g}
\]

\[
F_{lat,ave} = W G_{lat}(DMF)
\]

### 9.3.4 Load Estimation Based on Impact Severity

The lateral force required by a longitudinal barrier to resist an impact is directly related to the impact severity \((IS)\) of the event. The impact severity is calculated by Equation (88), where \( m \) is the test inertial mass of the vehicle, \( V_f \) is the impact speed, and \( \theta \) is the impact angle.

\[
IS = \frac{1}{2} m(V_f \sin(\theta))^2
\]

The data from several full-scale crash tests was processed as described in Section 9.2.2 to estimate the maximum forces imparted to bridge railings. The inertial effects were considered negligible for reasons discussed in Section 9.2.4. For each analysis, the corresponding impact severity was calculated using Equation (88). The data was then plotted as an impact force versus impact severity scatter. The temporary concrete barrier tests were not included in this analysis because the goal of this model was to estimate the lateral forces of rigid barriers only. Then, a linear regression was conducted, with forcing
the y-intercept to be zero, to obtain a direct relationship between the impact forces and the impact severities. This procedure is illustrated in Figure 87. From this analysis, the function for the 50 ms average force ($F_{lat,50ms,ave}$) is shown in Equation (89).

Table 16. Vehicle Dimensions for NCHRP Report No. 86

<table>
<thead>
<tr>
<th>Test Vehicle</th>
<th>NCHRP 350 or MASH</th>
<th>Test No.</th>
<th>$A_l$, ft (m)</th>
<th>$2B$, ft (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000P</td>
<td>NCHRP 350</td>
<td>NIT-1</td>
<td>7.34 (2.24)</td>
<td>6.17 (1.88)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CBPP-1</td>
<td>7.47 (2.28)</td>
<td>6.19 (1.89)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MOBR-1</td>
<td>7.51 (2.29)</td>
<td>6.17 (1.88)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MOBR-2</td>
<td>7.51 (2.29)</td>
<td>6.19 (1.89)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td>7.46 (2.27)</td>
<td>6.18 (1.88)</td>
</tr>
<tr>
<td>2270P</td>
<td>MASH</td>
<td>MGSBR-1</td>
<td>8.69 (2.65)</td>
<td>6.48 (1.97)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WVBR-1</td>
<td>8.38 (2.55)</td>
<td>6.48 (1.97)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4CMB-1</td>
<td>8.46 (2.58)</td>
<td>6.50 (1.98)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4CMB-5</td>
<td>8.48 (2.58)</td>
<td>6.48 (1.97)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td>8.50 (2.59)</td>
<td>6.48 (1.98)</td>
</tr>
<tr>
<td>8000S</td>
<td>NCHRP 350</td>
<td>ZOI-1</td>
<td>15.49 (4.72)</td>
<td>7.75 (2.36)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ZOI-3</td>
<td>13.06 (3.98)</td>
<td>7.71 (2.35)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CYRO-1</td>
<td>15.72 (4.79)</td>
<td>7.75 (2.36)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td>14.76 (4.50)</td>
<td>7.74 (2.36)</td>
</tr>
<tr>
<td>10000S</td>
<td>MASH</td>
<td>2214NJ-2</td>
<td>16.41 (5.00)</td>
<td>7.79 (2.37)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td>16.41 (5.00)</td>
<td>7.79 (2.37)</td>
</tr>
</tbody>
</table>
Figure 87. Linear Regression of Lateral Impact Force vs. Impact Severity

\[ F_{lat,50ms,ave} = 0.6554(IS) \]  

(89)

9.3.5 Comparison of Force Calculation Methods

The force calculation methods presented, as well as the design values listed in the AASHTO LRFD Bridge Manual, were compared to forces calculated from full-scale crash tests. The inertial effects were considered negligible and the barrier forces were calculated based on the onboard accelerometers in the vehicles. Three different categories of crash tests were used for comparison with the proposed force calculation methods: (1) NCHRP Report No. 350 3-11/4-11, (2) NCHRP Report No. 350 4-12, and (3) MASH 4-12. There was not any data available from a MASH 3-11/4-11 crash test with a permanent concrete barrier. For each crash test category, the averages of the actual
impact conditions were used to predict the barrier loads instead of the nominal conditions. This allowed for a more accurate comparison between the analytical models and the actual crash test data. The actual crash conditions were generally more severe than the nominal impact conditions defined in NCHRP Report no. 350 and MASH. A comparison of the different models to the processed MwRSF accelerometer data is shown in Table 17. Plots of the force histories relative to the predicted model design loads are shown in Figures 88 through 90.

As can be seen from the data, the current AASHTO LRFD design loads slightly under-predicted the actual loads obtained from processing accelerometer data from full-scale crash tests for NCHRP Report No. 350. The model appears that it would not be suitable for a MASH 4-12 test either due to the increase severity of MASH criteria. It should be noted however, that the AASHTO data in this comparison is based on the nominal impact conditions, as opposed to the actual impact conditions of the test data that were slightly more severe than the nominal conditions.
Table 17. Model Comparison to Actual Crash Data

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Average Vehicle Weight/Mass, lb (kg)</th>
<th>Average Actual Speed, mph (km/h)</th>
<th>Average Actual Angle, deg</th>
<th>Average Actual IS, k-ft (kJ)</th>
<th>Average Actual Lateral Load, kips (kN)</th>
<th>Test-to-Predicted Ratio, AASHTO LRFD</th>
<th>NCHRP Report No. 86/ Impulse-Momentum (e=1) Model Load, kips (kN)</th>
<th>Test-to-Predicted Ratio, NCHRP Report No. 86/ Impulse-Momentum (e=1)</th>
<th>Impact Severity Model Load, kips (kN)</th>
<th>Test-to-Predicted Ratio, Impact Severity Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCHRP Report No. 350, 3-11/4-11</td>
<td>4,451 (2,019)</td>
<td>63.4 (102.1)</td>
<td>25.71</td>
<td>112.7 (152.9)</td>
<td>71.5 (317.9)</td>
<td>54.0 (240.2)</td>
<td>1.32</td>
<td>76.1 (338.4)</td>
<td>0.94</td>
<td>73.9 (328.6)</td>
</tr>
<tr>
<td>NCHRP Report No. 350, 4-12</td>
<td>17,694 (8,026)</td>
<td>50.0 (80.5)</td>
<td>16.65</td>
<td>122.3 (165.8)</td>
<td>73.7 (328.0)</td>
<td>54.0 (240.2)</td>
<td>1.37</td>
<td>59.5 (264.6)</td>
<td>1.24</td>
<td>80.2 (356.5)</td>
</tr>
<tr>
<td>MASH, 4-12</td>
<td>22,045 (9,999)</td>
<td>56.5 (90.9)</td>
<td>15.67</td>
<td>171.2 (232.1)</td>
<td>117.6 (523.2)</td>
<td>N/A</td>
<td>N/A</td>
<td>80.2 (356.5)</td>
<td>1.47</td>
<td>112.2 (499.1)</td>
</tr>
</tbody>
</table>
Figure 88. Accelerometer Force Comparison for an NCHRP Report No. 350 3-11 Test

Figure 89. Accelerometer Force Comparison for an NCHRP Report No. 350 4-12 Test
The relationship of the predicted values from the NCHRP Report No. 86 coincided fairly well for NCHRP Report No. 350 criteria. However, for a MASH 4-12 test, the test-to-predicted ratio was 1.47. This increase in error is attributed to the vehicle center of gravity location. Equation (85) is highly dependent on the variable $AL$. For MASH pickup and SUT vehicles, the center of gravity is located farther back from the front of vehicle than for NCHRP Report No. 350 vehicles. The time interval of the impact was therefore increased, which allowed for a smaller force. This caused the increase in error for the MASH 4-12 test. Therefore, it was considered that this procedure was not as accurate for MASH vehicles due to its possible instabilities with vehicles that have large dimensions from the front of the vehicle to the center of gravity.
When the coefficient of restitution was set equal to 1 and Equation (85) was used to estimate the time interval of the impact, the results of the impulse-momentum model were equal to the results from the NCHRP Report No. 86 model. The lower bound of the impulse-momentum model corresponded to values significantly lower than the observed loads determined from the accelerometer analysis. This indicates that the impact is more closely related to an elastic impact rather than a perfectly plastic impact. However, this model was also highly dependent on the estimated time interval of impact, which was difficult to accurately determine. Therefore, this model was also considered to have instabilities due to the fact that small changes in the estimation of the time interval of impact could have significant effects on the lateral load estimation.

The impact severity model showed the most accurate, precise, and stable behavior of all the models for all three test configurations explored. It is basically an average of all test data that provides a method of interpolation for any impact within the range of the data that was analyzed. Due to the accuracy and simplicity of this model, it was considered the best method for estimating the lateral loads imparted to rigid bridge railings during vehicular impacts.

In addition, more crash test data obtained from previous studies was considered. This addition crash test data could not be included in some of the previous models due to the lack of known input parameters. The crash test load data was obtained from a table compiled by Hirsh and published in the Transportation Research Record (TRR) [85]. Impacts of vehicles with concrete walls and parapets were filtered from the database but data from tractor-trailers was not considered due to their articulating nature. The data contained the maximum 50 ms average lateral loads as determined from accelerometer
data and, when applicable, load cell data from instrumented wall tests. For the instrumented wall tests, an estimated height of the resultant force was also listed. The data from Hirsh is summarized in Table 18. The loads obtained from vehicle accelerometer analysis were within 20 percent of the loads obtained from the instrumented walls, further confirming the hypothesis that accelerometer analysis can be used to accurately determine loads from crash tests.

A comparison of the crash test data compared to the impact severity and AASHTO models is shown in Figure 91. The test data contains force data obtained from Hirsh, as well as the processed MwRSF data. It can be seen that the impact severity model compares very well to data obtained from full-scale crash tests. Also, it appears that the current AASHTO model underestimates the lateral forces as determined from this analysis. Therefore, it is suggested that impact severity model can be used to estimate the lateral impact forces of concrete bridge railings for impacts with an impact severity up to 300 kip-ft (407 kJ). Design load models for heavy articulated vehicles are available in Reference 90.
Table 18. Full-Scale Crash Test Forces Compiled by Hirsch [85]

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Vehicle Type</th>
<th>Vehicle Weight/Mass, lb (kg)</th>
<th>Vehicle Speed, mph (km/h)</th>
<th>Impact Angle, deg</th>
<th>Barrier Type</th>
<th>Barrier Height, in. (mm)</th>
<th>Height of Resultant Force, in. (mm)</th>
<th>IS, k-ft (kJ)</th>
<th>Lateral Load from Load Cells, kips (kN)</th>
<th>Lateral Load from Accelerometer, kips (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3451-32</td>
<td>Plymouth</td>
<td>4,680 (2,123)</td>
<td>52.9 (85.1)</td>
<td>15</td>
<td>Instrumented Concrete Wall</td>
<td>42 (1,067)</td>
<td>21.2 (538)</td>
<td>29.3 (39.7)</td>
<td>52.10 (231.8)</td>
<td>43.60 (193.9)</td>
</tr>
<tr>
<td>3451-36</td>
<td>Plymouth</td>
<td>4,740 (2,150)</td>
<td>59.9 (96.4)</td>
<td>24</td>
<td>Instrumented Concrete Wall</td>
<td>42 (1,067)</td>
<td>21.9 (556)</td>
<td>94.0 (127.4)</td>
<td>59.90 (266.4)</td>
<td>69.60 (309.6)</td>
</tr>
<tr>
<td>230-3</td>
<td>School Bus</td>
<td>19,690 (8,931)</td>
<td>54.4 (87.5)</td>
<td>15</td>
<td>Concrete Parapet with Metal Rail</td>
<td>42 (1,067)</td>
<td>-</td>
<td>130.4 (176.8)</td>
<td>-</td>
<td>96.50 (429.3)</td>
</tr>
<tr>
<td>3825-17</td>
<td>Ford SUT</td>
<td>18,240 (8,274)</td>
<td>60.3 (97.1)</td>
<td>15</td>
<td>CMB Concrete</td>
<td>32 (813)</td>
<td>-</td>
<td>148.4 (201.3)</td>
<td>-</td>
<td>153.00 (680.6)</td>
</tr>
<tr>
<td>3451-34</td>
<td>School Bus</td>
<td>20,030 (9,085)</td>
<td>57.6 (92.7)</td>
<td>15</td>
<td>Instrumented Concrete Wall</td>
<td>42 (1,067)</td>
<td>32.7 (831)</td>
<td>148.7 (201.6)</td>
<td>73.80 (328.3)</td>
<td>82.20 (365.6)</td>
</tr>
<tr>
<td>3825-8</td>
<td>School Bus</td>
<td>20,000 (9,072)</td>
<td>57.7 (92.9)</td>
<td>15</td>
<td>CMB Concrete Parapet</td>
<td>32 (813)</td>
<td>-</td>
<td>149.0 (202.0)</td>
<td>-</td>
<td>106.00 (471.5)</td>
</tr>
<tr>
<td>3080-1</td>
<td>School Bus</td>
<td>20,270 (9,194)</td>
<td>61.6 (99.1)</td>
<td>15</td>
<td>CMB Concrete</td>
<td>32 (813)</td>
<td>-</td>
<td>172.1 (233.3)</td>
<td>-</td>
<td>120.00 (533.8)</td>
</tr>
<tr>
<td>3115-1</td>
<td>School Bus</td>
<td>19,990 (9,067)</td>
<td>60.9 (98.0)</td>
<td>16</td>
<td>CMB Concrete</td>
<td>32 (813)</td>
<td>-</td>
<td>188.1 (255.1)</td>
<td>-</td>
<td>120.00 (533.8)</td>
</tr>
<tr>
<td>8307-3</td>
<td>Scenicruiser Bus</td>
<td>40,030 (18,157)</td>
<td>54.0 (86.9)</td>
<td>14</td>
<td>CMB Concrete</td>
<td>32 (813)</td>
<td>-</td>
<td>228.2 (309.4)</td>
<td>-</td>
<td>170.00 (756.2)</td>
</tr>
<tr>
<td>3451-35</td>
<td>Intercity Bus</td>
<td>32,020 (14,524)</td>
<td>56.9 (91.6)</td>
<td>15.7</td>
<td>Instrumented Concrete Wall</td>
<td>42 (1,067)</td>
<td>28.4 (721)</td>
<td>253.6 (343.8)</td>
<td>211.20 (939.5)</td>
<td>220.00 (978.6)</td>
</tr>
<tr>
<td>8307-1</td>
<td>Scenicruiser Bus</td>
<td>40,020 (18,153)</td>
<td>54.0 (86.9)</td>
<td>16.2</td>
<td>CMB Concrete</td>
<td>32 (813)</td>
<td>-</td>
<td>303.4 (411.4)</td>
<td>-</td>
<td>150.00 (667.2)</td>
</tr>
</tbody>
</table>
The yield line analysis method can be utilized to determine the adequacy of the bridge railing. However, AASHTO states that the yield line analysis procedure is only valid when the yield line pattern does not extend into the deck [71]. Therefore, the ultimate strength of the anchorage is required to be developed when using this method. The flexural resistance about its vertical axis can be calculated using conventional principles of reinforce concrete design. Finally, the nominal railing resistance can be compared to the design impact force calculated by Equation (89) to determine the adequacy of the barrier design.

Figure 91. Lateral Force vs. Impact Severity Test Data Comparison

9.4 Anchor Loads from Yield Line Analysis

The yield line analysis method can be utilized to determine the adequacy of the bridge railing. However, AASHTO states that the yield line analysis procedure is only valid when the yield line pattern does not extend into the deck [71]. Therefore, the ultimate strength of the anchorage is required to be developed when using this method. The flexural resistance about its vertical axis can be calculated using conventional principles of reinforce concrete design. Finally, the nominal railing resistance can be compared to the design impact force calculated by Equation (89) to determine the adequacy of the barrier design.
CHAPTER 10 - SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

10.1 Summary

The objective of this project was to determine if epoxy anchors can be utilized to anchor crash barriers to bridge decks and to create design procedures for implementing epoxy anchorages into concrete bridge railings. These procedures should allow for more flexibility from a design and construction perspective as the use of epoxy anchors should simplify construction by eliminating the need for cast-in-place anchors. This research was intended to allow for the installation of precast aesthetic concrete traffic barriers, cast-in-place barriers, and temporary concrete barriers. Also, this technique should be applicable barriers, as well as retrofit solutions.

An extensive literature review was conducted to review the common design methodologies used to design epoxy concrete anchors. Most of these studies focused only on static loading conditions. The anchorages used in bridge rail applications required a protective coating against corrosion of either galvanization or more commonly epoxy. None of the reviewed anchorage studies were conducted with epoxy-coated anchor bars. Several models were analyzed and it was determined that the full uniform bond model was the most accurate and stable for the medium embedment depths associated with bridge rail applications.

A series of 16 dynamic bogie tests and one static test was conducted to investigate the behavior of bridge railing anchors under dynamic load. Most of the anchors tested were no. 5 (metric no. 16) or no. 6 (metric no. 19) deformed reinforcing bars, which were the most commonly used anchorages according to a review of the Midwest States Pooled Fund standard plans. Additional dynamic tests were conducted on 1 1/8 in. (29 mm)
diameter A307 threaded rods, which was the anchorage required for the F-shape temporary concrete barrier developed by MwRSF.

An analytical and experimental study on the design loads imparted to bridge railings was also conducted in this study. A new model based on the impact severity of a crash was developed to predict the required capacity of a rigid barrier for non-articulating vehicles with an impact severity of up to 300 kip-ft (407 kJ).

10.2 Conclusions

By comparing the bond strengths from static and dynamic tests for the uncoated no. 6 (no. 16 metric) reinforcing bars, the dynamic increase factor for the bond strength failure mode was calculated to be 1.06. This value was lower than values obtained from the literature search. It was observed that anchors coated with epoxy according to ASTM A775 standards lead to a decrease in bond strength by approximately 9 percent. According to the test data and the specifications for the Hilti HIT-RE 500-SD, the bond strength values from the manufacturer were specified to be 74 percent lower than the static bond strength from testing in this study.

Anchor spacing has been observed to affect the strength of a group of anchors placed in close proximity to one another. Test results indicated that an 8 in. (203 mm) spacing was sufficient to all no. 5 (metric no. 16) epoxy-coated, deformed reinforcing bars to reach their maximum capacity for 5 ¼ in. (133 mm) embedments. However, no. 6 (metric no. 19) reinforcing bars installed in the same manner produced only 80 percent of the measured strength of a single bar. These values were applied as calibration factors in the final model described in Chapter 8.
From the analysis of 8 full-scale crash tests involving pickup trucks and SUTs, it was concluded that the design impact force on rigid barriers could be accurately and conservatively determined from the processed accelerometer data and yaw angles. A regression analysis was performed to develop a model to predict the lateral load imparted to rigid bridge railings for non-articulated vehicles with an impact severity up to 300 kip-ft (407 kJ). Equation (90) shows the relationship between the 50 ms average lateral force in kips and the impact severity in kip-ft and was shown to provide accurate results for the tests analyzed in this study.

\[ F_{lat,50ms,ave} = 0.6554(IS) \]  

(90)

10.3 Recommendations

The procedures contained in Chapter 8 are recommended for use in designing epoxy anchorages for bridge railings. The steel and bond failure modes have been investigated using static and dynamic experiments in this study. Further, dynamic bond stresses have been developed for the Hilti HIT-RE 500-SD epoxy adhesive, as shown in Table 11. However, the limit state of concrete failure was not able to be explored due to the constraints of this project. As a conservative approach, conventional design procedures from ACI are recommended as checks for concrete breakout strength. It is desired to develop the ultimate strength of the anchorage steel when designing bridge railings.

The model for estimating the ultimate load on bridge railings was presented in Section 9.3.4. An empirical equation, as a function of the impact severity, was recommended for estimating the ultimate dynamic load a barrier is required to resist. A
yield line analysis technique is recommended for determining the ultimate resistance of the barrier and required anchorages.

**10.4 Recommendations for Future Work**

Due to the limitations of this study, a proper investigation of the concrete breakout strength was unable to be conducted. It is believed that for epoxy anchors with an embedment depth of at least 5 ¼ in. (133 mm) a full concrete cone breakout will not control, as testing in this study showed. However, the tests in this investigation utilized concrete with a compressive strength much higher than some bridge deck designs commonly used. Therefore, no explicit conclusions could be made from this study about the concrete breakout strength in lower strength concrete. It is believed that the concrete cone breakout equation in ACI 318-11 is quite conservative, it is recommended that this be investigated further to confirm this estimation. Further, the concrete breakout of an adhesive anchor in shear is recommended for further study to determine the critical edge distance that an anchor can be placed from a free edge to preclude side fracturing of the slab.
CHAPTER 11 - REFERENCES

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2. Breen, J.E., Eligehausen, R., Fuchs, Werner., Concrete Capacity Design (CCD) Approach for Fastening to Concrete, American Concrete Institute Structural Journal, January-February 1995.


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CHAPTER 12 - APPENDICIES
Appendix A. Comparison of Epoxy Manufacturers’ Test Data
Table A-1. Epoxy Manufacturers’ Test Data with Threaded Rod

<table>
<thead>
<tr>
<th>Anchor Diameter (in)</th>
<th>Embedment Depth (in)</th>
<th>Product</th>
<th>Tensile Strength (k)</th>
<th>Shear Strength (k)</th>
<th>f', (ksi)</th>
<th>Ave Tensile Strength (k)</th>
<th>Ave Shear Strength (k)</th>
<th>Max Tensile Strength (k)</th>
<th>Max Shear Strength (k)</th>
<th>Max Tensile Product</th>
<th>Max Shear Product</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.5</td>
<td>Power Fasteners Power Fast +</td>
<td>4.4</td>
<td>4.6</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Power Fasteners T308+</td>
<td>4.06</td>
<td>N/A</td>
<td>4</td>
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<td></td>
<td></td>
<td></td>
<td>Adhesives Technology HS200</td>
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<tr>
<td></td>
<td>1.875</td>
<td>USP Structural Connectors CIA-GEL 7000</td>
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<td>3.375</td>
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</tr>
<tr>
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<td>3.375</td>
<td>Power Fasteners T308+</td>
<td>9.58</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>3.375</td>
<td>Adhesives Technology HS200</td>
<td>8.214</td>
<td>7.072</td>
<td>2</td>
<td>10.9</td>
<td>7.312</td>
<td>10.9</td>
<td>7.312</td>
<td>Adhesives Technology HS200</td>
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<td>3.375</td>
<td>Adhesives Technology Ultrabond 1</td>
<td>9.248</td>
<td>7.189</td>
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<tr>
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<td>7.312</td>
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<tr>
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<td>3.375</td>
<td>USP Structural Connectors CIA-GEL 7000</td>
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<td>3.5</td>
<td>Simpson ET</td>
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Appendix B. Conventional Anchorage Design Calculations
Figure B-1. Wisconsin Sloped Face Parapet ‘LF’ Detailed Drawing [64]
Conventional Design Calculations for the Sloped Face Parapet ‘LF’

The highest strength attachment to the bridge deck utilizes 2-#5 bars spaced at 8 in. O.C.

\[ l_{emb} = (8 \text{ in.}) - (2 \text{ in.}) - 0.5 \left( \frac{5}{8} \text{ in.} \right) = 5.6875 \text{ in.} \]
\[ l_{dh} = (8 \text{ in.}) - (2 \text{ in.}) = 6 \text{ in} \]
\[ l_{dh} = \frac{0.02 \psi_e f_y}{\lambda \sqrt{f_c}} d_b \]
\[ \psi_e = 1.2 \text{ for epoxy-coated reinforcement} \]
\[ \lambda = 1.0 \text{ for normalweight concrete} \]
\[ 6 \text{ in.} = \frac{0.02(1.2)f_s}{1.0 \sqrt{4000 \text{ psi}}} (0.625 \text{ in.}) \]
\[ f_s = 25,298 \text{ psi} \]
\[ F = f_s A_s = (25,298 \text{ psi})(0.31 \text{ in.}^2) = 7,842 \text{ lb} = 7,842 \text{ k} \]
The right leg is angled 64° from the concrete slab surface
\[ F_{y-component} = (7,842 \text{ k}) \sin 64^\circ = 7,048 \text{ k} \]
Normalizing to a force per foot of barrier by dividing by the anchor spacing
\[ F_{per\ foot} = \frac{7,048 \text{ k}}{\frac{8}{12} \text{ ft}} = 10.57\text{k/ft} \]
- The distance from the hooked bar to the edge of the concrete slab is approximately 1 ft. The moment strength is calculated by the tensile force times the moment arm. Conservatively assume the moment arm as the distance from the hooked bar to the edge of the concrete.

\[ M_{\text{per foot}} = \left( 10.57 \frac{k}{ft} \right) (1.0 \text{ ft}) = 10.57 \frac{k \cdot \text{ft}}{\text{ft}} \]

**Determine Moment Strength of Hooked Bar for 2.5 in. Clear Cover**

\[ l_{dh} = (8 \text{ in.}) - (2.5 \text{ in.}) = 5.5 \text{ in.} \]

\[ l_{dh} = \frac{0.02f_y}{\lambda \sqrt{f_c}} db(0.7) \]

\[ 5.5 \text{ in.} = \frac{0.02(1.2)f_s}{1.0 \sqrt{4000 \text{ psi}}} (0.625 \text{ in.})(0.7) \]

\[ f_s = 33,129 \text{ psi} \]

\[ F = f_sA_s = (33,129 \text{ psi})(0.31 \text{ in.}^2) = 10,270 \text{ lb} = 10.270 \text{ k} \]

\[ F_{y-component} = (10.270 \text{ k}) \sin 64^\circ = 9.231 \text{ k} \]

Normalize to a force per foot of barrier by dividing by the anchor spacing

\[ F_{\text{per foot}} = \frac{9.231 \text{ k}}{\left( \frac{8}{12} \text{ ft} \right)} = 13.85 \frac{k}{\text{ft}} \]

\[ M_{\text{per foot}} = \left( 13.85 \frac{k}{\text{ft}} \right) (1.0 \text{ ft}) = 13.85 \frac{k \cdot \text{ft}}{\text{ft}} \]

**Determine Shear Strength of Right Leg Based on Shear Friction**

\[ V_n = A_{vf}f_y(\mu \sin \alpha + \cos \alpha) \]

\[ \alpha = 64^\circ \]

- The hooked end of the bar does not have enough length to develop the yield stress of the bar, therefore \( f_s \) will be used in lieu of \( f_y \)
- Assume the edge effects are negligible for the right leg

\[ \mu = 0.6\lambda \]

\[ V_n = (0.31 \text{ in.}^2)(33,129 \text{ psi})((0.6)(1.0) \sin 64^\circ + \cos 64^\circ) = 10,040 \text{ lb} \]

\[ = 10.04 \text{ k} \]

**Determine Shear Strength of Left Leg Based on ACI Appendix D and ICC-ES AC308**

**Steel Strength of Anchor in Shear**

\[ V_{sa} = n0.6A_{se, vf}\]

\[ f_{uta} = 90,000 \text{ psi} \text{ for grade 60 steel} \]

\[ 1.9f_{yta} = 1.9(60,000 \text{ psi}) = 114,000 \text{ psi} \]

\[ f_{uta} \leq 114,000 \text{ psi} \leq 125,000 \text{ psi} \]

\[ V_{sa} = 1(0.6)(0.31 \text{ in.}^2)(90,000 \text{ psi}) = 16,740 \text{ lb} = 16.74 \text{ k} \]
Concrete Breakout of Anchor in Shear

-Assume the anchor is located 2 in. clear from the slab edge
-Assume the concrete is uncracked, this will be conservative in determining the equivalent strength of the barrier

\[ c_{a1} = (2 \text{ in.}) + 0.5 \left( \frac{5}{8} \text{ in.} \right) = 2.3125 \text{ in.} \]

\[ 1.5c_{a1} = 1.5(2.3125 \text{ in.}) = 3.4688 \text{ in.} \]

\[ A_{vc} = 2(1.5c_{a1})(1.5c_{a1}) \]

\[ = 2(3.4688 \text{ in.})(3.4688 \text{ in.}) \]

\[ = 24.07 \text{ in.}^2 \]

\[ A_{vc0} = A_{vc} \text{ for a single anchor with } h_a \geq 1.5c_{a1} \text{ and no corner effects} \]

\[ \psi_{ed,v} = 1.0 \]

\[ \psi_{c,v} = 1.4 \]

\[ \psi_{h,v} \text{ is not applicable for } h_a > 1.5c_{a1} \]

\[ V_b = \left( 7 \left( \frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \right) \lambda \sqrt{f_c(c_{a1})^{1.5}} \]

\[ l_e = h_{ef} = 6 \text{ in.} \]

\[ d_a = 0.625 \text{ in.} \]

\[ V_b = \left( 7 \left( \frac{6 \text{ in.}}{0.625 \text{ in.}} \right)^{0.2} \sqrt{0.625 \text{ in.}} \right)(1.0)\sqrt{4000 \text{ psi}(2.3125 \text{ in.})^{1.5}} \]

\[ = 1935 \text{ lb} \]

\[ V_{cb} = \frac{A_{vc}}{A_{vc0}} \psi_{ed,v} \psi_{c,v} \psi_{h,v} V_b \]

\[ = \frac{24.07 \text{ in.}^2}{24.07 \text{ in.}^2} (1.0)(1.4)(1.0)(1935 \text{ lb}) = 2,709 \text{ lb} = 2.71 \text{ k} \]

Concrete Pryout Strength of Anchor in Shear

-This bar will behave more like an adhesive anchor than a headed or mechanical anchor due to the fact that a full concrete cone will most likely not form because the concentration of stress transfer will not be at the bottom of the anchor. Therefore, the concrete pryout strength for this anchor will be analyzed from the provisions of adhesive anchors (ICC-ES AC308).

\[ V_{cp} = \min \left| k_{cp} N_a; k_{cp} N_{cb} \right| \]

\[ k_{cp} = 2.0 \text{ for } h_{ef} > 2.5 \text{ in.} \]

\[ N_a = \frac{A_{Na}}{A_{Na0}} \psi_{ed,Na} \psi_{p,Na} N_{a0} \]

\[ N_{a0} = \tau_k \pi d h_{ef} \]

\[ -\text{This is equal to the pullout strength of the bar and} \]
will be designed based on the development strength of a straight bar.

\[ l_d = (8 \text{ in.}) - (2 \text{ in.}) \]

\[ l_d = \left( \frac{3 \psi_t \psi_e \psi_s}{40 \sqrt{f_c'} \frac{c_b}{d_b} (c_b + K_{tr} \frac{d_b}{d_b})} \right) d_b \]

\[ K_{tr} = 0 \]

\[ \psi_t = 1.0 \]

\[ 3d_b = 3(0.625 \text{ in.}) = 1.875 \text{ in.} \]

\[ \text{cover} = 2.0 \text{ in.} \]

\[ \psi_e = 1.2 \]

\[ \psi_t \psi_e = 1.2 < 1.7 \]

\[ \psi_s = 0.8 \]

\[ c_b = (2 \text{ in.}) + 0.5(0.625 \text{ in.}) = 2.3125 \text{ in.} \]

\[ \lambda = 1.0 \text{ for normal weight concrete} \]

\[ 6 \text{ in.} = \left( \frac{3 f_s}{40 \sqrt{4000 \text{ psi} \left( \frac{2.3125 \text{ in.} + 0 \text{ in.}}{0.625 \text{ in.}} \right)}} \right) 0.625 \text{ in.} \]

\[ f_s = 31,201 \text{ psi} \]

\[ N_{a0} = f_s A_s = (31,201 \text{ psi})(0.31 \text{ in.}^2) = 9,672 \text{ lb} \]

\[ c_{cr,Na} = \frac{s_{cr,Na}}{2} \]

\[ s_{cr,Na} = 20 d \sqrt{\frac{\tau_k}{1450}} \leq 3h_{ef} \]

- Estimate the bond strength based on the pullout capacity

\[ \tau_k = \frac{N_{a0}}{A_{bond}} = \frac{N_{a0}}{\pi d_h} = \frac{9,672 \text{ lb}}{\pi(0.625 \text{ in.})(6 \text{ in.})} = 821.0 \text{ psi} \]

\[ s_{cr,Na} = 20(0.625 \text{ in.}) \sqrt{\frac{821.0 \text{ psi}}{1450}} = 9.41 \text{ in.} \]

\[ \leq 3(6 \text{ in.}) = 18 \text{ in.} \]

\[ c_{cr,Na} = \frac{9.41 \text{ in.}}{2} = 4.71 \text{ in.} \]

\[ A_{Na} = (4.71 \text{ in.} + 2.3125 \text{ in.})(2(4.71 \text{ in.})) = 66.15 \text{ in.}^2 \]

\[ A_{Na0} = s_{cr,Na}^2 = (9.41 \text{ in.})^2 = 88.55 \text{ in.}^2 \]
\[ c_{a,\text{min}} > 1.5 h_{ef} \]

\[ \psi_{ed,Na} = 0.7 + 0.3 \frac{c_{a,\text{min}}}{c_{cr,Na}} \leq 1.0 \]

\[ \psi_{ed,Na} = 0.7 + 0.3 \frac{2.3125 \text{ in.}}{4.71 \text{ in.}} = 0.85 \]

\[ h \]

\[ h_{ef} = 8 \text{ in.} \]

\[ c_{ac} = 2.5 h_{ef} = 2.5(6 \text{ in.}) = 15 \text{ in.} \]

\[ c_{a,\text{min}} < c_{ac} \]

\[ \psi_{p,Na} = \frac{\max[c_{a,\text{min}}; c_{cr,Na}]}{c_{ac}} \]

\[ \psi_{p,Na} = \frac{4.71 \text{ in.}}{15 \text{ in.}} = 0.31 \]

\[ N_a = \frac{66.15 \text{ in.}^2}{88.55 \text{ in.}^2} (0.85)(0.31)(9.672 \text{ lb}) = 1,904 \text{ lb} \]

\[ = 1.90 \text{ k} \]

\[ N_{cb} = \frac{A_{nc}}{A_{nc0}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \]

\[ N_{cb} = k_c \sqrt[1.5]{h_{ef}} \]

- Let \( k_c = 17 \) as this anchor will behave more like a post-installed anchor than a cast-in-place anchor

\[ N_b = 17(1.0) \sqrt[15]{4000 \text{ psi}(6 \text{ in.})} = 15,802 \text{ lb} \]

\[ c_{a,\text{min}} < 1.5 h_{ef} \]

\[ \psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,\text{min}}}{1.5 h_{ef}} \]

\[ \psi_{ed,N} = 0.7 + 0.3 \frac{2.3125 \text{ in.}}{1.5(6 \text{ in.})} = 0.78 \]

\[ \psi_{c,N} = 1.4 \]

\[ \psi_{cp,N} = \frac{c_{a,\text{min}}}{c_{ac}} = \frac{2.3125 \text{ in.}}{15 \text{ in.}} = 0.15 \]

\[ A_{nc} = (2.3125 \text{ in.} + 1.5(6 \text{ in.})) \times (2(1.5)(6 \text{ in.})) = 203.63 \text{ in.}^2 \]

\[ A_{nc0} = 9 h_{ef}^2 = 9(6 \text{ in.})^2 = 324 \text{ in.}^2 \]

\[ N_{cb} = \frac{203.63 \text{ in.}^2}{324 \text{ in.}^2} (0.78)(1.4)(0.15)(15,802 \text{ lb}) \]

\[ = 1,627 \text{ lb} = 1.63 \text{ k} \]

\[ V_{cp} = 2(1.63 \text{ k}) = 3.26 \text{ k} \]
**Total Shear Strength of Barrier**

\[ V_n = V_{n,\text{right leg}} + V_{n,\text{left leg}} = 10.04 \, k + 2.71 \, k = 12.75 \, k \]

Normalize to a force per foot of barrier by dividing by the anchor spacing

\[ V_n = \frac{12.75 \, k}{\frac{8}{12} \, ft} = 19.13 \, k/ft \]

**Load Summary**

\[ M_n = 13.85 \, \frac{k \cdot ft}{ft} \]

\[ V_n = 19.13 \, \frac{k}{ft} \]
Notation

\(A_{\text{bond}}\) = Area of bond

\(A_{Na}\) = The projected area of the failure surface for the anchor or group of anchors

\(A_{Na0}\) = The projected area of the failure surface of a single anchor without the influence of proximate edges

\(A_s\) = Area of steel

\(A_{se,V}\) = Effective cross-sectional area of the anchor in shear

\(A_{Vc}\) = Projected concrete area of a single anchor or group of anchors

\(A_{Vc0}\) = Projected concrete failure area of a single anchor

\(A_{vf}\) = Area of shear-friction reinforcement

\(c_{a1}\) = Distance from the center of an anchor shaft to the edge of concrete

\(c_{ac}\) = Critical edge distance required to develop the basic concrete breakout strength

\(c_b\) = The smaller of the distance from the center of a bar to nearest concrete and one-half the center-to-center spacing of bars being developed

\(c_{cr,Na}\) = Critical adhesive anchor edge distance for tension loading

\(d\) = Nominal diameter of the anchor element

\(d_a\) = Outside diameter of anchor

\(d_b\) = Nominal diameter of bar

\(h\) = Thickness of member in which an anchor is installed

\(h_a\) = Thickness of member in which an anchor is located

\(h_{ef}\) = Effective embedment depth, measured from the concrete surface to the deepest point on the anchor element at which a bond to the concrete is established

\(f_c'\) = Specified compressive strength of concrete

\(f_s\) = Stress in steel

\(f_y\) = Specified yield strength of the reinforcement

\(k_c\) = Coefficient for basic concrete breakout strength in tension

\(k_{cp}\) = Coefficient for pryout strength

\(K_{tr}\) = Transverse reinforcement index

\(l_d\) = Development length in tension of a deformed bar

\(l_{dh}\) = Development length in tension of a deformed bar with a standard hook

\(l_e\) = Load bearing length of anchor for shear

\(l_{emb}\) = Embedment length of the anchor

\(n\) = Number of anchors

\(N_a\) = Nominal strength of an adhesive anchor in tension as limited by bond/concrete failure

\(N_{a0}\) = Characteristic tension capacity of a single adhesive anchor between the adhesive and the concrete

\(N_{cb}\) = Nominal concrete strength of a single anchor in tension as limited by concrete cone breakout

\(s_{cr,Na}\) = Critical adhesive anchor spacing for tension loading

\(V_b\) = Basic concrete breakout strength in shear of a single anchor in cracked concrete

\(V_{cp}\) = Nominal concrete pryout strength of a single anchor

\(V_n\) = Nominal shear strength

\(\alpha\) = Angle defining the orientation of reinforcement
\( \lambda \) = Modification factor reflecting the reduced mechanical properties of lightweight concrete relative to normalweight concrete of the same compressive strength
\( \mu \) = Coefficient of friction
\( \tau_k \) = Characteristic bond strength
\( \psi_{c,V} \) = Factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement
\( \psi_{h,V} \) = Factor used to modify shear strength of anchors located in concrete members with \( h_d < 1.5c_{a1} \)
\( \psi_{ed,Na} \) = Factor used to modify the tensile strength of a single or group of anchors based on edge effects
\( \psi_{ed,V} \) = Factor used to modify shear strength of anchors based on proximity to edges of a concrete member
\( \psi_{p,Na} \) = Factor used to modify the tensile strength of a single or group of anchors based on the critical edge distance
\( \psi_s \) = Factor used to modify development length based on reinforcement size
\( \psi_t \) = Factor used to modify development length based on reinforcement location
Appendix C. Conversion of Cook’s Equations from Metric to English Units
Conversion of Cook’s Equations from Metric to English Units

Assume an anchor with the following properties:

\[ d_0 = 0.75 \text{ in} = 19.05 \text{ mm} \]
\[ \tau_0 = 1800 \text{ psi} = 12.41 \text{ MPa} \]
\[ h_{ef} = 5.25 \text{ in} = 133.35 \text{ mm} \]
\[ f_c' = 4000 \text{ psi} = 27.58 \text{ MPa} \]

In metric units Equation (3) is shown below where \( h_{cone} \) is in mm, \( d_0 \) is in mm, \( \tau_0 \) is in MPa, and \( f_c' \) is in MPa.

\[
h_{cone} = \frac{\tau_0 \pi d_0}{1.84 \sqrt{f_c'}} \quad \text{(MPa)} \pi (\text{mm})
\]
\[
\text{mm} = \frac{1.84 \sqrt{\text{MPa}}}{(12.41 \text{ MPa}) \pi (19.05 \text{ mm})}
\]
\[
h_{cone} = \frac{76.86 \text{ mm}}{1.84 \sqrt{27.58 \text{ MPa}}} = 3.03 \text{ in.}
\]

Solving for \( x \)

\[ x = 22.13 \]

Therefore, the equation in English units is:

\[
h_{cone} = \frac{\tau_0 \pi d_0}{22.13 \sqrt{f_c'}}
\]

In metric units Equation (4) is shown below where \( N_t \) is in N, \( h_{ef} \) in is mm, \( f_c' \) is in MPa.

\[
N_t = 0.92 h_{ef} \sqrt{f_c'}
\]
\[
N = 0.92 (\text{mm})^2 \sqrt{\text{MPa}}
\]
\[
N_t = 0.92 (133.35 \text{ mm})^2 \sqrt{27.58 \text{ MPa}} = 85,915 \text{ N} = 19,315 \text{ lb}
\]
\[
19,315 \text{ lb} = x (5.25 \text{ in.})^2 \sqrt{4000 \text{ psi}}
\]

Solving for \( x \)

\[ x = 11.08 \]

Therefore, the equation in English units is:

\[
N_t = 11.08 h_{ef} \sqrt{f_c'}
\]

In metric the expression \( 40 \sqrt{d_0} \) in Equation (5) has \( d_0 \) in units of mm.

\[
40 \sqrt{19.05 \text{ mm}} = 174.59 \text{ mm} = 6.87 \text{ in.}
\]
\[
6.87 \text{ in.} = x \sqrt{0.75 \text{ in.}}
\]

Solving for \( x \)

\[ x = 7.93 \]

Therefore, the expression in English units is:

\[ 7.93 \sqrt{d_0} \]
Appendix D. Static Model Comparison to Hilti HIT-RE 500 Test Data
## Table D-1. Model Comparison Using the Bond Stress Specified in the Hilti Documentation

<table>
<thead>
<tr>
<th>d (in)</th>
<th>d₀ (in)</th>
<th>hₑ (in)</th>
<th>fₑ (psi)</th>
<th>Actual Test Capacity (k)</th>
<th>Full Uniform Bond Model (k)</th>
<th>Concrete Cone Model (k)</th>
<th>Cone or Full Uniform Bond Model (k)</th>
<th>Cone or Cone Plus Partial Uniform Bond Model with Calculated Cone Height (k)</th>
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<td>0.4375</td>
<td>1.75</td>
<td>1800</td>
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COV = 0.27
Mean = 1.00
St Dev = 0.19
COV = 0.19
Table D-1. Model Comparison Using the Bond Stress Specified in the Hilti Documentation (continued)

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Table D-2. Model Comparison Using the Bond Stress Specified in ICC-ES ESR-2322

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<th>d (in)</th>
<th>d₀ (in)</th>
<th>hₐf (in)</th>
<th>x₀ (in)</th>
<th>Actual Test Capacity (k)</th>
<th>Full Uniform Bond Model Capacity (k)</th>
<th>Test-to-Predicted Ratio</th>
<th>Concrete Cone Model Capacity (k)</th>
<th>Test-to-Predicted Ratio</th>
<th>Cone or Full Uniform Bond Model Capacity (k)</th>
<th>Test-to-Predicted Ratio</th>
<th>Cone or Cone Plus Partial Uniform Bond Model with Calculated Cone Height Capacity (k)</th>
<th>Test-to-Predicted Ratio</th>
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Appendix E. Preliminary Dynamic Testing Calculations
Single Bar Tension Calculations (WEAB-1 to WEAB-4)

Steel Strength of the Anchor in Tension
- Assume the anchor is a #5 grade 60 deformed reinforcing bar

\[ N_{sa} = n A_{se,N} f_{uta} \psi_{d,N} \]

\[ A_{se,N} = 0.31 \text{ in.}^2 \text{ for a #5 reinforcing bar} \]
\[ f_{uta} = 90,000 \text{ psi} \leq 1.9 f_{yta} = 114,000 \text{ psi} \leq 125,000 \text{ psi} \]
\[ n = 1 \]
\[ \psi_{d,N} = 1.25 \]
\[ N_{sa} = (1)(0.31 \text{ in.}^2)(90,000 \text{ psi})(1.25) = 34,875 \text{ lb} = 34.88 \text{ k} \]

Bond Strength of the Anchor in Tension
- Assume the concrete to be uncracked
- Assume epoxy to be the Hilti HIT-RE 500

\[ N_{ba} = \frac{A_{Nc}}{A_{Nc0}} \psi_{ed,N} \psi_{ec,N} \psi_{d,N} \psi_{x,N} N_{t} \]
\[ N_{t} = \tau_{0} \pi d_{0} h_{ef} \]
\[ \tau_{0} = 2,145 \text{ psi} \]

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Table 25

\[ d_{0} = d + \frac{1}{8} \text{ in.} = \frac{5}{8} \text{ in.} = \frac{1}{8} \text{ in.} = 0.75 \text{ in.} \]
\[ N_{t} = (2145 \text{ psi})\pi (0.75 \text{ in.})(5.25 \text{ in.}) = 26,534 \text{ lb} \]
\[ c_{cr} = 10d \left( \frac{\tau_{0}}{1450} \right)^{0.5} \leq 1.5h_{ef} \]
\[ c_{cr} = 10(0.625 \text{ in.}) \left( \frac{2,145 \text{ psi}}{1450} \right)^{0.5} = 7.60 \text{ in.} \leq 1.5(5.25 \text{ in.}) = 7.88 \text{ in.} \]
$c_{a,min} = \infty$
 $\psi_{ed,N} = 1.0$
 $\psi_{ec,N} = 1.0$
 $\psi_{d,N} = 1.25$ for dynamic loads
 $\psi_{x,N} = 0.8$ for epoxy-coated bars

$A_{Nc} = A_{Nc0} \therefore \frac{A_{Nc}}{A_{Nc0}} = 1.0$

Static strength of uncoated bar

$N_{ba} = (1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(26,534 \text{ lb}) = 26,534 \text{ lb}$
$= 26.53 \text{ k}$

Static strength of epoxy-coated bar

$N_{ba} = (1.0)(1.0)(1.0)(1.0)(0.8)(26,534 \text{ lb}) = 21,227 \text{ lb}$
$= 21.23 \text{ k}$

Dynamic strength of uncoated bar

$N_{ba} = (1.0)(1.0)(1.0)(1.25)(1.0)(26,534 \text{ lb}) = 33,168 \text{ lb}$
$= 33.17 \text{ k}$

Dynamic strength of epoxy-coated bar

$N_{ba} = (1.0)(1.0)(1.0)(1.25)(0.8)(26,534 \text{ lb}) = 26,534 \text{ lb}$
$= 26.53 \text{ k}$

**Comparison with Hilti Documentation and Software**

- The Hilit documentation stated that the pullout capacity of a #5 bar with 5.625 in. embedment = 29.42 k (static)  \[19\]
- The Hilti PROFIS Anchor Software version 2.0.7 calculated the max capacities for a #5 grade 60 deformed reinforcing bar with an embedment depth of 5.25 in.
  - Max steel capacity = 27.90 k (static)
  - Max bond capacity = 22.11 k (static)
  - Max cone capacity = 18.26 k (static) \[91\]
Single Bar Shear Calculations (WEAB-5 and WEAB-6)

Steel Strength of the Anchor in Shear

\[ V_{sa} = n0.6A_{se,v}f_{u,t,a} \]

\[ 1.9f_{yta} = 1.9(60,000 \text{ psi}) = 114,000 \text{ psi} \]

\[ V_{sa} = (1)(0.6)(0.31 \text{ in.}^2)(90,000 \text{ psi}) = 16,740 \text{ lb} = 16.74 \text{ k (static)} \]

Estimate the dynamic increase factor for shear to be 1.2

\[ V_{sa} = 1.2(16,740 \text{ lb}) = 20,088 \text{ lb} = 20.09 \text{ k (dynamic)} \]

Concrete Breakout Strength of an Anchor in Shear

The limit states of concrete breakout strength of an anchor in shear do not apply for an anchor located an infinite distance from an edge.

Concrete Pryout Strength of an Anchor in Shear

\[ V_{cp} = \min \left| k_{cp}N_a; k_{cp}N_{cb} \right| \]

\[ k_{cp} = 2.0 \text{ for } h_{ef} > 2.5 \text{ in.} \]

\[ N_a = \frac{A_{Na}}{A_{Na0}} \psi_{ed,Na} \psi_{ed,Na} \psi_{d,N} \psi_{x,N} N_t \]

\[ N_t = \tau_0 \pi d_0 h_{ef} = (2,145 \text{ psi})(\pi)(0.75 \text{ in.})(5.25 \text{ in.}) \]

\[ = 26,534 \text{ lb} \]

\[ A_{Na} = A_{Na0} \text{ for a single anchor with infinite edge distance} \]

\[ \therefore \frac{A_{Na}}{A_{Na0}} = 1.0 \]

\[ \psi_{ed,Na} = 1.0 \text{ for } c_{a,min} > c_{cr,Na} \]
\[ \psi_{ec,Na} = 1.0 \text{ for an anchor loaded without an eccentricity} \]
\[ \psi_{d,N} = 1.25 \text{ for dynamic loading conditions} \]
\[ \psi_{X,N} = 0.80 \text{ for epoxy-coated reinforcing bars} \]

If the bar coating does not affect the strength of the anchor
\[ N_a = (1.0)(1.0)(1.0)(1.25)(1.0)(26,534 \text{ lb}) = 33,168 \text{ lb} \]
\[ = 33.17 \text{ k} \]

If the bar coating does affect the strength of the anchor
\[ N_a = (1.0)(1.0)(1.0)(1.25)(0.80)(26,534 \text{ lb}) = 26,534 \text{ lb} \]
\[ = 26.53 \text{ k} \]

\[ N_{cb} = \frac{A_{Nc}}{A_{Nc0}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} \psi_{d,N} \psi_{X,N} N_b \]
\[ A_{Nc} = A_{Nc0} \text{ for a single anchor with infinite edge distance} \]
\[ \therefore \frac{A_{Nc}}{A_{Nc0}} = 1.0 \]
\[ N_b = k_c \lambda \sqrt{f_c k_{ef}^{1.5}} \]
\[ k_c = 17 \text{ for post-installed anchors} \]
\[ \lambda = 1.0 \text{ for normalweight concrete} \]
\[ N_b = (17)(1.0)\sqrt{4000 \text{ psi}(5.25 \text{ in.})^{1.5}} = 12,934 \text{ lb} \]
\[ = 12.93 \text{ k} \]
\[ \psi_{ed,N} = 1.0 \text{ for an anchor with } c_{a,min} > 1.5 h_{ef} \]
\[ \psi_{c,N} = 1.4 \text{ for a post-installed anchor} \]
\[ \psi_{cp,N} = 1.0 \text{ for an anchor with } c_{a,min} > c_{ac} \]
\[ \psi_{d,N} = 1.25 \text{ for dynamic loading conditions} \]
\[ \psi_{X,N} = 0.80 \text{ for epoxy-coated reinforcement} \]

If the bar coating does not affect the strength of the anchor
\[ N_{cb} = (1.0)(1.0)(1.4)(1.25)(1.0)(12,934 \text{ lb}) = 22,635 \text{ lb} \]
\[ = 22.64 \text{ k} \]

If the bar coating does affect the strength of the anchor
\[ N_{cb} = (1.0)(1.0)(1.4)(1.25)(0.80)(12,934 \text{ lb}) = 18,108 \text{ lb} \]
\[ = 18.11 \text{ k} \]

If the bar coating does not affect the strength of the anchor
\[ V_{cp} = (2.0)(22,635 \text{ lb}) = 45,270 \text{ lb} = 45.27 \text{ k} \]

If the bar coating does affect the strength of the anchor
\[ V_{cp} = (2.0)(18,108 \text{ lb}) = 36,216 \text{ lb} = 36.22 \text{ k} \]
Double Bar Tension Calculations (WEAB-7 and WEAB-8)

Steel Strength of the Anchor in Tension
- Assume the anchors are #5 grade 60 deformed reinforcing bars

\[ N_{sa} = nA_{se,N}f_{uta}\psi_{d,N} \]

\[ A_{se,N} = 0.31 \text{ in.}^2 \text{ for a #5 reinforcing bar} \]

\[ f_{uta} = 90,000 \text{ psi} \leq 1.9f_{yta} = 114,000 \text{ psi} \leq 125,000 \text{ psi} \]

\[ n = 2 \]

\[ \psi_{d,N} = 1.25 \]

\[ N_{sa} = (2)(0.31 \text{ in.}^2)(90,000 \text{ psi})(1.25) = 69,750 \text{ lb} = 69.75 \text{ k} \]

Bond Strength of the Anchor in Tension
- Assume the concrete to be uncracked
- Assume epoxy to be the Hilti HIT-RE 500

\[ N_{ba} = \frac{A_{NC}}{A_{Nc0}}\psi_{ed,N}\psi_{ec,N}\psi_{d,N}\psi_{x,N}N_{\tau} \]

\[ c_{cr} = 10d\left(\frac{\tau_0}{1450}\right)^{0.5} \leq 1.5\psi_{ef} \]

\[ c_{cr} = 10(0.625 \text{ in.})\left(\frac{2,145 \text{ psi}}{1450}\right)^{0.5} = 7.60 \text{ in.} \leq 1.5(5.25 \text{ in.}) \]

\[ = 7.88 \text{ in.} \]

\[ s_{cr} = 2c_{cr} = 2(7.60 \text{ in.}) = 15.20 \text{ in.} \]

\[ A_{Nc0} = s_{cr}^2 = (15.20 \text{ in.})^2 = 231.04 \text{ in.}^2 \]

\[ A_{NC} = (7.60 \text{ in.} + 8 \text{ in.} + 7.60 \text{ in.})(2(7.60 \text{ in.})) = 352.64 \text{ in.}^2 \]

Calculate \( N_{\tau} \) based on several different models

\[ \tau_0 = 2,145 \text{ psi} \text{ for #5 reinforcing bars in uncracked concrete} \]
Table 25

1.) Cone or full uniform bond stress model

\[ h_{cone} = \frac{\tau_0 \pi d_0}{22.13 \sqrt{f_c'}} = \frac{(2,145 \text{ psi})(\pi)(0.75 \text{ in.})}{22.13 \sqrt{4000 \text{ psi}}} = 3.61 \text{ in.} \]

\[ h_{ef} = 5.25 \text{ in.} > h_{cone} = 3.61 \text{ in.} \]

\[ N_t = \tau_0 \pi d_0 h_{ef} = (2,145 \text{ psi})(\pi)(0.75 \text{ in.})(5.25 \text{ in.}) \]

\[ = 26,534 \text{ lb} = 26.53 \text{ k} \]

2.) Cone + partial uniform bond stress model with calculated cone height

\[ h_{cone} = \frac{\tau_0 \pi d_0}{22.13 \sqrt{f_c'}} = \frac{(2,145 \text{ psi})(\pi)(0.75 \text{ in.})}{22.13 \sqrt{4000 \text{ psi}}} = 3.61 \text{ in.} \]

\[ h_{u,crit} = 7.93 \sqrt{d_0} + h_{cone} = 7.93 \sqrt{0.75 \text{ in.} + 3.61 \text{ in.}} \]

\[ = 10.48 \text{ in.} \]

\[ h_{ef} = 5.25 \text{ in.} < 10.48 \text{ in.} \]

\[ N_t = \tau_0 \pi d_0 (h_{ef} - h_{cone}) \]

\[ + 11.08 h_{cone}^2 \frac{7.93 \sqrt{d_0} - (h_{ef} - h_{cone})}{7.93 \sqrt{d_0}} \]

\[ N_t = (2,145 \text{ psi})(\pi)(0.75 \text{ in.})(5.25 \text{ in.} - 3.61 \text{ in.}) \]

\[ + 11.08(3.61 \text{ in.})^2 \sqrt{4000 \text{ psi}} \frac{7.93 \sqrt{0.75 \text{ in.} - (5.25 \text{ in.} - 3.61 \text{ in.})}}{7.93 \sqrt{0.75 \text{ in.}}} \]

\[ N_t = 15,240 \text{ lb} = 15.24 \text{ k} \]

3.) Cone or cone + partial uniform bond stress model with assumed cone height

\[ h_{ef} = 5.25 \text{ in.} > h_{cone} = 2 \text{ in.} \]

\[ N_t = \tau_0 \pi d_0 (h_{ef} - 2 \text{ in.}) \]

\[ + 11.08(2 \text{ in.})^2 \sqrt{f_c'} \frac{7.93 \sqrt{d_0} - (h_{ef} - 2 \text{ in.})}{7.93 \sqrt{d_0}} \]

\[ N_t = (2,145 \text{ psi})(\pi)(0.75 \text{ in.})(5.25 \text{ in.} - 2 \text{ in.}) \]

\[ + 11.08(2 \text{ in.})^2 \sqrt{4000 \text{ psi}} \frac{7.93 \sqrt{0.75 \text{ in.} - (5.25 \text{ in.} - 2 \text{ in.})}}{7.93 \sqrt{0.75 \text{ in.}}} \]

\[ N_t = 17,902 \text{ lb} = 17.90 \text{ k} \]

4.) Cone or partial uniform bond stress model with assumed cone height

\[ h_{ef} = 5.25 \text{ in.} > h_{cone} = 2 \text{ in.} \]

\[ N_t = \tau_0 \pi d_0 (h_{ef} - 2 \text{ in.}) \]

\[ N_t = (2,145 \text{ psi})(\pi)(0.75 \text{ in.})(5.25 \text{ in.} - 2 \text{ in.}) \]

\[ = 16,425 \text{ lb} = 16.43 \text{ k} \]

5.) Modified cone or cone + partial uniform bond stress model with assumed cone height

\[ d_0 = d + \frac{1}{8} \text{ in.} = \frac{5}{8} \text{ in.} + \frac{1}{8} \text{ in.} = \frac{6}{8} \text{ in.} = 0.75 \text{ in.} \]
6.) Estimate bond stress from Hilti test 5/8 in. bar with 5.625 in. embedment and apply a uniform bond stress model.

\[ P_{\text{max}} = 29,420 \text{ lb} \] for 5/8 in. bar with 5.625 in. embedment

\[ \tau_{0,\text{test}} = \frac{P_{\text{max}}}{\pi d_0 h_{ef}} = \frac{29,420 \text{ lb}}{\pi (0.75 \text{ in.})(5.625 \text{ in.})} = 2,220 \text{ psi} \]

\[ N_t = \tau_{0,\text{test}} \pi d_0 h_{ef} = (2,220 \text{ psi})(\pi)(0.75 \text{ in.})(5.25 \text{ in.}) = 27,461 \text{ lb} = 27.46 \text{ k} \]

7.) Concrete cone model

\[ N_t = 11.08 h_{ef}^2 \sqrt{f_c'} \]

\[ N_t = 11.08(5.25 \text{ in.})^2 \sqrt{4000 \text{ psi}} = 19,315 \text{ lb} = 19.32 \text{ k} \]

Based on analysis of these models on the test data obtained from the 2008 Hilti North American Product Technical Guide, the cone or uniform full bond model showed good results with the smallest coefficient of variation. Therefore, the capacity will be predicted by that model.

\[ \psi_{ed,N} = 1.0 \text{ for edge distance of } \infty \]
\[ \psi_{ec,N} = 1.0 \text{ for an anchor loaded without an eccentricity} \]
\[ \psi_{d,N} = 1.25 \text{ for dynamic loading conditions} \]
\[ \psi_{x,N} = 0.80 \text{ for epoxy-coated reinforcing bars} \]

If the bar coating does not affect the strength of the anchors

\[ N_{ba} = \frac{352.64 \text{ in.}^2}{231.04 \text{ in.}^2} (1.0)(1.0)(1.0)(1.25)(26,534 \text{ lb}) = 50,624 \text{ lb} \]
\[ = 50.62 \text{ k} \]

If the bar coating does affect the strength of the anchors

\[ N_{ba} = \frac{352.64 \text{ in.}^2}{231.04 \text{ in.}^2} (1.0)(1.0)(1.0)(0.80)(26,534 \text{ lb}) = 40,499 \text{ lb} \]
\[ = 40.50 \text{ k} \]
Appendix F. Test Jig Design Calculations and Drawings
Tensile Test Jig Calculations

**Estimate Loads**

The estimated pullout capacity for two 5.25 in. embedded anchors is 53 k. The test jig will be designed to a safety factor of 2. Therefore the downward force at the anchor will be \( P = 106 \) k

\[
\begin{align*}
\uparrow \sum F_y &= 0 = (-106 \text{ k}) + \frac{1}{2} L w f_{bearing} \\
&
\end{align*}
\]

\( w = 13 \text{ in.} \)

\( f_{bearing} = 0.85 f_c' \quad \text{ACI 318-08 10.14.1} \)

\[
(106 \text{ k}) = \frac{1}{2} L (13 \text{ in.})(0.85)(4 \text{ ksi})
\]

\( \therefore L = 4.8 \text{ in.} \)

\[
\begin{align*}
\uparrow \sum M_0 &= 0 \\
&= (106 \text{ k})(4.6 \text{ in.}) - F(21.66 \text{ in.}) \\
&+ (106 \text{ k}) \left( 4.275 \text{ in.} - \frac{4.80 \text{ in.}}{3} \right)
\end{align*}
\]

\( F = 35.60 \text{ k} \)

\( M_{max} = (35.60 \text{ k})(21.66 \text{ in.}) = 771.10 \text{ k \cdot in.} = 64.26 \text{ k \cdot ft} \)

\( V_{max} = 35.60 \text{ k} \)

**Design using a W6x25 for the I-beam** (\( f_y = 50 \text{ ksi} \))

Check limiting width-thickness ratios \( \text{AISC 360-05 [92] Table B4.1} \)

\[
\frac{b}{t} = 6.68
\]
\( \lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 0.38 \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 9.15 > 6.68 \)  

\( \therefore \text{The flanges are compact for flexure} \)

\( \frac{h}{t_w} = 15.5 \)

\( \lambda_p = 3.76 \sqrt{\frac{E}{F_y}} = 3.76 \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 90.55 > 15.5 \)  

\( \therefore \text{The web is compact for flexure} \)

Yielding

\[ Z_x = 18.9 \text{ in.}^3 \]

\[ M_n = M_p = F_y Z_x = (50 \text{ ksi})(18.9 \text{ in.}^3) = 945 \text{ k \cdot in.} \]

\[ = 78.8 \text{ k \cdot ft} \]

Lateral-Torsional Buckling

\[ L_b = 21.66 \text{ in.} \]

\[ r_y = 1.52 \text{ in.} \]

\[ L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76(1.52 \text{ in.}) \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 64.43 \text{ in.} \]

\[ > 21.66 \text{ in.} \]

\( \therefore \text{Lateral-torsional buckling does not apply} \)

The section is compact so local buckling does not apply

\[ M_u = 64.26 \text{ k \cdot ft} < M_n = 78.8 \text{ k \cdot ft} \]

\( \therefore \text{The beam is adequate for the anticipated loading} \)

**Punching Capacity of Base Plate**

The estimated maximum strength of any anchor is governed by the tensile capacity of the steel anchor = 35 k.

By multiplying the maximum strength by a safety factor of 2, the ultimate capacity is \( F_u = 70 \text{ k} \).

Diameter of coupler = 1.38 in.

Thickness of baseplate = \( t = 1.0 \text{ in.} \).

Yield Stress of baseplate = \( f_y = 36 \text{ ksi} \)

\[ A_p = \pi dt = \pi(1.38 \text{ in.})(1.0 \text{ in.}) = 4.34 \text{ in.}^2 \]

Allowable shear stress = \( f_{v,allow} = 0.6f_y = 0.6(36 \text{ ksi}) = 21.6 \text{ ksi} \)

\[ V_{allow} = A_b f_{v,allow} = (4.34 \text{ in.}^2)(21.6 \text{ ksi}) = 93.74 k > 70 \text{ k} \]

**Bending Capacity of Base Plate**

The estimated maximum load applied to the outside holes is 26.5 k.

Assume that the load will be carried by one-way bending of the baseplate and half the load will go to each gusset.

\[ I_{x1} = \frac{bh^3}{12} = \frac{(3.0 \text{ in.})(1.0 \text{ in.})^3}{12} = 0.25 \text{ in.}^4 \]

\( c = 0.5 \text{ in.} \)
These stress values are both slightly above the yield stress of the steel, however they are unconservative values since they are based on one-way cantilever bending and are still well below the ultimate strength of the steel.

**Tensile Weld Strength**

A 0.375 in. weld is used around the I-beam and the gusset plates.

\[
(\text{throat depth}) = 0.707(\text{weld size}) = 0.707(0.375 \text{ in.}) = 0.265 \text{ in.}
\]

\[
F_{weld} = 0.3(F_{XX}) = 21 \text{ ksi}
\]

\[
f_{weld} = f_v(\text{throat depth}) = (21 \text{ ksi})(0.265 \text{ in.}) = 5.57 \text{ k/in.}
\]

Total length of weld above neutral axis = 4(2.75 in.) + 2(12.5 in.) + 0.5(6.38 in.) − 0.46 in. = 38.73 in.

\[
F_{weld} = f_{weld}L_{weld} = (5.57 \text{ k/in.})(38.73 \text{ in.}) = 216 k > 106 k
\]

**Shear Strength of Anchors on Kick Plate**

Use two 3/4 in. Power Fasteners wedge bolts

Shear capacity/bolt = 21.96 k

\[
V_n = 2(21.96 k) = 43.92 k > 35.6 k
\]
Shear Test Jig Calculations

Estimate Loads

The estimated shear capacity is 20.09 k
The test jig will be designed to a safety factor of 2. Therefore the reaction in
the x-direction will be \( R_x = 40.18 \text{ k} \)
\[ + \sum F_x = 0 = F - 40.18 \text{ k} \]
\[ \therefore F = 40.18 \text{ k} \]
\[ + \sum M_p = 0 \]
\[ = (-40.18 \text{ k})(7.06 \text{ in.}) + (-40.18 \text{ k})(0.5 \text{ in.}) \]
\[ + R_y(49.34 \text{ in.}) \]
\[ \therefore R_y = 6.16 \text{ k} \]
\[ + \sum M_F = 0 \]
\[ = (-40.18 \text{ k})(7.56 \text{ in.}) + (6.16 \text{ k})(37.95 \text{ in.}) \]
\[ + P(11.39 \text{ in.}) \]
\[ \therefore P = 6.14 \text{ k} \]

Design Tapcon Screws for Uplift at Load \( P \)

Tensile strength of one 3/8”x2” screw in 4,000 psi concrete
\[ = 2.55 \text{ k} \]

Strength of 4 tapcons \[ = 4(2.55 \text{ k}) = 10.2 \text{ k} > 6.14 \text{ k} \]
Use a 3/8”x3” tapcon screw since the screw will not be bonded for the top
1.25”
Estimate Loads on Strap

![Diagram](image)

\[ +\sum M_{Rs} = 0 = (-40.18\ k)(4.5\ in.) + F(1.35\ in.) \]
\[ \therefore F = 133.9\ k \]
\[ +\sum M_{Fs} = 0 = R_s(1.35\ in.) - (40.18\ k)(3.15\ in.) \]
\[ \therefore R_s = 93.75\ k \]

The strap is angled 48° from the end of the channels. Each side of the strap will take half the load.

\[ R_{s1} = \frac{R_s}{2} = \frac{93.75\ k}{2} = 46.88\ k \]

Calculate the tension in the angled portions of the strap.

\[ T = \frac{46.88\ k}{\sin 48°} = 63.08\ k \]

**Fracture of Strap at Angled Section**

The strap will be made of a 3 in. x 0.5 in. A36 plate.

\[ P_n = A_{\text{strap}}f_u = (3\ in.)(0.5\ in.)(60\ ksi) = 90\ k > 63.08\ k \]

**Fracture of Strap at Bolts**

Use 0.75 in. bolts. The diameter of the bolt hole will be 0.875 in.

\[ P_n = (A_{\text{strap}} - A_{\text{bolt\ hole}})f_u = ((3\ in. - 0.875\ in.)(0.5\ in.))(60\ ksi) \]
\[ = 63.75\ k > 46.88\ k \]

**Shear at Bolts**

Use three 0.75 in. grade 5 bolts.

\[ V_n = 0.6A_{\text{bolt}}f_u \]
\[ A_{\text{bolt}} = 0.334\ in.^2 \]
\[ f_u = 120\ ksi \]
\[ V_n = 0.6(0.334\ in.^2)(120\ ksi) = 24.05\ k > \frac{46.88\ k}{3} = 15.63\ k \]
Figure F-1. Tensile Test Jig
Figure F-2. Tensile Test Jig Weld Details
Figure F-3. Tensile Test Jig Base Plate Detail
Figure F-4. Tensile Test Jig Kick Plate Detail
Figure F-5. Tensile Test Jig W6x25 Beam Detail
Figure F-6. Tensile Test Jig Plate Gusset Detail
Figure F-7. Tensile Test Jig Post Gusset Detail
Figure F-8. Tensile Test Jig Post Stiffener Detail
Figure F-9. Tensile Test Jig Wedge Bolt Detail

Notes: (1) Powers Fasteners $\frac{3}{4}'' \times 6''$ long Wedge Bolt Screw Anchor
## Epoxy Anchor Tension Test Fixture

<table>
<thead>
<tr>
<th>Item No.</th>
<th>QTY.</th>
<th>Description</th>
<th>Material Spec</th>
</tr>
</thead>
<tbody>
<tr>
<td>a1</td>
<td>1</td>
<td>Base Plate 13x11x1</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a2</td>
<td>1</td>
<td>Kick Plate 13x3x1</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a3</td>
<td>1</td>
<td>Post W6x25x28</td>
<td>ASTM A992 or ASTM A572 (50 ksi strength)</td>
</tr>
<tr>
<td>a4</td>
<td>2</td>
<td>Wedge Bolt φ 3/4 x 6 in. long</td>
<td>Powers Fasteners</td>
</tr>
<tr>
<td>a5</td>
<td>2</td>
<td>Plate Gusset 6x2.75x0.5</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a6</td>
<td>4</td>
<td>Post Gusset 5.5x2.5x0.25</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a7</td>
<td>1</td>
<td>Post Stiffener 12.5x6x0.5</td>
<td>ASTM A36</td>
</tr>
</tbody>
</table>

Figure F-10. Tensile Test Jig Bill of Materials
Figure F-11. Shear Test Jig

Note: (1) Shims (g12) may be added to assembly as shown to make test article snug. Dimensions as follows: minimum height 6 1/4 in., maximum width 3 1/8 in., and maximum thickness 5/8 in. or to snug secure fit. Shims can be multiple pieces and should extend outward to strap.
Figure F-12. Shear Test Jig Weld Details
Figure F-13. Shear Test Jig Base Plate Detail
Figure F-14. Shear Test Jig Front Gusset Detail
Figure F-15. Shear Test Jig Skid Plate Detail
Figure F-16. Shear Test Jig Skid Tube Channel Detail
Figure F-17. Shear Test Jig Top Gusset Detail
Figure F-18. Shear Test Jig Coupler Strap Detail
Figure F-19. Shear Test Jig End Plate Detail
Figure F-20. Shear Test Jig Fixture Guide Detail
## Epoxy Anchor Shear Test Fixture

<table>
<thead>
<tr>
<th>Item No.</th>
<th>QTY.</th>
<th>Description</th>
<th>Material Spec</th>
</tr>
</thead>
<tbody>
<tr>
<td>a1</td>
<td>1</td>
<td>Base Plate 24x5x0.5</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a2</td>
<td>1</td>
<td>Vertical Plate 10x4x0.5</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a3</td>
<td>2</td>
<td>Front Gusset 5.375x8.5x0.5</td>
<td>ASTM A36</td>
</tr>
<tr>
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<td>Skid Plate 6x7x0.5</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a5</td>
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<td>Right Skid Tube Channel C6x8.2x36</td>
<td>ASTM A36</td>
</tr>
<tr>
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<td>2</td>
<td>Top Gusset 3.5x8x0.5</td>
<td>ASTM A36</td>
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<td>Fixture Guide Lower Plate 2.75x4x0.75</td>
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<td>Fixture Guide Upper Plate 11x4x0.375</td>
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</tr>
<tr>
<td>a9</td>
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<td>Coupler Strap 22.75x3x0.5</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a10</td>
<td>1</td>
<td>Left Skid Tube Channel C6x8.2x36</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a11</td>
<td>1</td>
<td>End Plate 3.875x6.3125x0.5</td>
<td>ASTM A36</td>
</tr>
<tr>
<td>a12</td>
<td>1</td>
<td>Shims</td>
<td>Steel</td>
</tr>
<tr>
<td>b3</td>
<td>3</td>
<td>Hex Head Bolt 0.7500–10x6.5x2–N</td>
<td>Grade 5</td>
</tr>
<tr>
<td>b4</td>
<td>6</td>
<td>Flat Washer 0.75</td>
<td>Grade 5</td>
</tr>
<tr>
<td>b5</td>
<td>3</td>
<td>Hex Nut 0.75</td>
<td>Grade 5</td>
</tr>
<tr>
<td>b6</td>
<td>4</td>
<td>Concrete Screw 3/8&quot; x 3&quot;</td>
<td>Tapcon Concrete Screw</td>
</tr>
</tbody>
</table>

Figure F-21. Shear Test Jig Bill of Materials
Appendix G. Test Setup Drawings
Epoxy Anchor Rebar Bolting Tensile Test Setup
1. Bolting No. 3 - Small Bolting with standard round impact head
2. Speed = 15 mph
3. One high-speed digital camera perpendicular
4. One high-speed digital camera perpendicular zoomed in on the tensile test jig and rebar.
5. JVC digital video
6. DTS and EDR-3
7. Note any installation issues or problems in fieldbook.
8. Note failure mode in fieldbook.
9. Each test must be at least 2' from any previous test anchor holes to prevent anchor spacing and edge spacing issues from affecting results.
10. Kick plate anchors must be replaced each test.
11. Epoxy and epoxy-coated rebar will be donated.
12. Before installation of anchors, measure the bar deformation rib height as well as the epoxy-coating thickness.
13. Epoxy used for first round of testing will be Hilti HIT-RE 500.

Figure G-1. Tension Test Setup, Test Nos. WEAB-1 Through WEAB-4 and WEAB-7 Through WEAB-8
Figure G-2. Shear Test Setup, Test Nos. WEAB-5 Through WEAB-6
Figure G-3. Anchor Attachment Details, Test Nos. WEAB-1 Through WEAB-8
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Bar Size</th>
<th>Bar Coating</th>
<th>Embedment Depth</th>
<th>Spacing</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-1</td>
<td>Tensile</td>
<td>#5</td>
<td>None</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
<tr>
<td>WEAB-2</td>
<td>Tensile</td>
<td>#5</td>
<td>None</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
<tr>
<td>WEAB-3</td>
<td>Tensile</td>
<td>#5</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
<tr>
<td>WEAB-4</td>
<td>Tensile</td>
<td>#5</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
<tr>
<td>WEAB-5</td>
<td>Shear</td>
<td>#5</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
<tr>
<td>WEAB-6</td>
<td>Shear</td>
<td>#5</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
<tr>
<td>WEAB-7</td>
<td>Tensile</td>
<td>#5</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>2 @ 8&quot;</td>
<td>10 mph</td>
</tr>
<tr>
<td>WEAB-8</td>
<td>Tensile</td>
<td>#5</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>2 @ 8&quot;</td>
<td>10 mph</td>
</tr>
</tbody>
</table>
Figure G-5. Tension Test Setup, Test Nos. WEAB-9 Through WEAB-12
Figure G-6. Shear Test Setup, Test No. WEAB-13
Figure G-7. Anchor Attachment Details, Test Nos. WEAB-9 Through WEAB-13
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Bar Size</th>
<th>Bar Coating</th>
<th>Embedment Depth</th>
<th>Spacing</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-9</td>
<td>Tensile</td>
<td>#6</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>15 mph</td>
</tr>
<tr>
<td>WEAB-10</td>
<td>Tensile</td>
<td>#6</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>15 mph</td>
</tr>
<tr>
<td>WEAB-11</td>
<td>Tensile</td>
<td>#6</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>2 @ 8&quot;</td>
<td>15 mph</td>
</tr>
<tr>
<td>WEAB-12</td>
<td>Tensile</td>
<td>#6</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>2 @ 8&quot;</td>
<td>15 mph</td>
</tr>
<tr>
<td>WEAB-13</td>
<td>Shear</td>
<td>#6</td>
<td>Epoxy</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
</tbody>
</table>

Figure G-8. Test Matrix, Test Nos. WEAB-9 Through WEAB-13
Epoxy Anchor Rebar Bogie Testing—Tensile Test Setup
1. Bogie No. 3 — Small Bogie with standard round impact head
2. Speed = 15 mph
3. One high-speed digital camera perpendicular
4. One high-speed digital camera perpendicular zoomed in on the tensile test jig and anchor.
5. JVC digital video
6. DTS and EDR-3
7. Note any installation issues or problems in fieldbook.
8. Note failure mode in fieldbook.
9. Each test must be at least 2' from any previous test anchor holes to prevent anchor spacing and edge spacing issues from affecting results.
10. Kick plate anchors must be replaced each test.
11. Anchors must be installed while the concrete temperature remains above 45° during curing time for the epoxy. If required, heat the concrete to the desired temperature.
12. Anchors are to be installed in same concrete slabs as used in test nos. WEAB 1–13.
13. Epoxy used for third round of testing will be Hilti HIT—RE 500–SD.
14. 1' 1/8" A307 rods with 5 1/4" embedment.
15. Tension jig must be modified to have larger slot. See sheet 3 of "EpoxyAnchor Tensile_R3".

Figure G-9. Tension Test Setup, Test No. WEAB-14
Figure G-10. Shear Test Setup, Test No. WEAB-15
Figure G-11. Anchor Attachment Details, Test Nos. WEAB-14 and WEAB-15
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Threaded Rod</th>
<th>Bar Coating</th>
<th>Embedment Depth</th>
<th>Spacing</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-14</td>
<td>Tensile</td>
<td>1 1/8&quot;</td>
<td>None</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>15 mph</td>
</tr>
<tr>
<td>WEAB-15</td>
<td>Shear</td>
<td>1 1/8&quot;</td>
<td>None</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>10 mph</td>
</tr>
</tbody>
</table>

Figure G-12. Test Matrix, Test Nos. WEAB-14 and WEAB-15
Figure G-13. Tension Test Setup, Test No. WEAB-16
Figure G-14. Anchor Attachment Details, Test No. WEAB-16

Notes:  
1. Coupler is not welded to test jigs.  
2. Refer to PDF titled "DS_Bar_Lock_End_Actor_TDS[1]" for maximum rebar shear lip diameter, bolt tightening pattern, and torque specifications.  
3. Stop pin needs to be removed.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Bar Size</th>
<th>Bar Coating</th>
<th>Embedment Depth</th>
<th>Spacing</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-16</td>
<td>Tensile</td>
<td>#6</td>
<td>None</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>15 mph</td>
</tr>
</tbody>
</table>

Figure G-15. Test Matrix, Test No. WEAB-16
Epoxy Anchor Rebar Static Testing—Tensile Test Setup

1. Hydraulic Ram
2. One high-speed digital camera perpendicular
3. One high-speed digital camera perpendicular zoomed in on the tensile test jig and rebar.
4. JVC digital video
5. Two Load Cells
6. Note any installation issues or problems in fieldbook.
7. Note failure mode in fieldbook.
8. Each test must be at least 2' from any previous test anchor holes to prevent anchor spacing and edge spacing issues from affecting results.
9. Kick plate anchors must be replaced each test.
10. Before installation of anchors, measure the bar deformation rib height.
11. Anchors must be installed while the concrete temperature remains above 45° during curing time for the epoxy. If required, heat the concrete to the desired temperature.
12. Anchors are to be installed in same concrete slabs as used in test nos. WEAB 1–15.
13. Epoxy used for fourth round of testing will be Hilti HIT-RE 500–50.

Figure G-16. Tension Test Setup, Test No. WEAB-17
Figure G-17. Anchor Attachment Details, Test No. WEAB-17
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Bar Size</th>
<th>Bar Coating</th>
<th>Embedment Depth</th>
<th>Spacing</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-17</td>
<td>Tensile</td>
<td>#6</td>
<td>None</td>
<td>5 1/4&quot;</td>
<td>Single</td>
<td>Static</td>
</tr>
</tbody>
</table>

Figure G-18. Test Matrix, Test No. WEAB-17
Appendix H. Material Specifications
**Figure H-1. Concrete Cylinder Test Results**
## Compression Test of Cylindrical Concrete Specimens

**Client:** UNL  
**Date:** December 13, 2010  
**Project:** MuRSF  
**Placement Location:** Wi: Epoxy West 4 & 5

### Mix Type: Class Mix No.

<table>
<thead>
<tr>
<th>Type of Forms</th>
<th>Cement Factor, Sls/Yd</th>
<th>Water-Cement Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Admixture Quantity</td>
<td>na</td>
<td>Slump inches</td>
</tr>
<tr>
<td>Admixture Type</td>
<td>na</td>
<td>Unit Wt, lbs/cu. Ft.</td>
</tr>
<tr>
<td>Admixture Quantity</td>
<td>na</td>
<td>Air Content, %</td>
</tr>
<tr>
<td>Average Field Temperature</td>
<td>na</td>
<td>Batch Volume, Cu. Yds.</td>
</tr>
<tr>
<td>Temperature of Concrete F</td>
<td>na</td>
<td>Ticket No.</td>
</tr>
<tr>
<td>Identification Laboratory</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Date Cast</td>
<td>12/13/2010</td>
<td>12/13/2010</td>
</tr>
<tr>
<td>Date Received in Laboratory</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Date Tested</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Days Cured in Field</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Days Cured in Laboratory</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Age of Test, Days</td>
<td>na</td>
<td>na</td>
</tr>
<tr>
<td>Length, in.</td>
<td>8.05</td>
<td>8.05</td>
</tr>
<tr>
<td>Average Width (1), in.</td>
<td>3.37</td>
<td>3.30</td>
</tr>
<tr>
<td>Cross-Sectional Area, sq. in.</td>
<td>11.977</td>
<td>11.552</td>
</tr>
<tr>
<td>Maximum Load, lbf</td>
<td>71,250</td>
<td>71,630</td>
</tr>
<tr>
<td>Compressive Strength, psi</td>
<td>5,970</td>
<td>5,950</td>
</tr>
<tr>
<td>Length/Diameter Ratio</td>
<td>2.061</td>
<td>2.065</td>
</tr>
<tr>
<td>Correction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corrected Compressive Strength, psi</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Type of Fracture</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Required Strength, psi</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Remarks:

All concrete break data in this report was produced by Benesch personnel using ASTM Standard Methods and Practices unless otherwise noted.

This report shall not be reproduced except in full, without the written approval of Alfred Benesch & Company.

---

**ALFRED BENESCH & COMPANY**  
**CONSTRUCTION MATERIALS LABORATORY**  
By: [Signature]

---

Figure H.2. Concrete Cylinder Test Results
<table>
<thead>
<tr>
<th>HEAT NUM.</th>
<th>DESCRIPTION</th>
<th>PHYSICAL TESTS</th>
<th>CHEMICAL TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>10079201</td>
<td>Nuco Steel - Kankakee Inc 10/95 Rebar 80° A615M Gr 420 (Gr60) ASTM A615/A615M-68Gr 80(420)</td>
<td>Yield 88,608 Tensile 103,937 Elong 13.8% Bend OK Wt% Def. 3.0% Cr 0.38 Mn 0.88 Mo 0.013 Si 0.051 Ni 0.40 P 0.020 Cu 0.34 Co 0.50</td>
<td>C 0.35 Ni 0.24 Mn 0.24 Cr 0.14 Mo 0.080 Si 0.007 Cu 0.001</td>
</tr>
<tr>
<td>0000079203</td>
<td>Nuco Steel - Kankakee Inc 10/95 Rebar 80° A515M Gr 420 (Gr60) ASTM A515/A515M-08Gr 60(420)</td>
<td>Yield 74,128 Tensile 108,072 Elong 10.0% Bend OK Wt% Def. 3.2% Cr 0.40 Mn 0.94 Mo 0.015 Si 0.047 Ni 0.19 Cu 0.41 Co 0.50</td>
<td>C 0.34 Ni 0.24 Mn 0.24 Cr 0.14 Mo 0.080 Si 0.007 Cu 0.001</td>
</tr>
</tbody>
</table>

Figure H-3. Reinforcing Steel Specifications, Test Nos. WEAB-1 Through WEAB-8
Figure H-4. Reinforcing Steel Specifications, Test Nos. WEAB-9 Through WEAB-13

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
<th>Characteristic</th>
<th>Value</th>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength test 1</td>
<td>62.9ksi</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Tensile Strength test 1</td>
<td>100.4ksi</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Elongation test 1</td>
<td>15%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elongation Gage Lgh test 1</td>
<td>SIN</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bend Test Diameter</td>
<td>3.750IN</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bend Test</td>
<td>Passed</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure H-5. Reinforcing Steel Specifications, Test Nos. WEAB-9 Through WEAB-13
ABC COATING COMPANY, INC.

P.O. BOX 9693
TULSA, OKLAHOMA 74157-0693
(918) 585-2507
FAX (918) 585-8131

AN ACUÑA CO.

DATE SHIPPED: 1/24/2011
INVOICE NO: 37859
OUR JOB NO: NE 447 #1
CUSTOMER: CONCRETE INDUSTRIES

CONTRACTOR: CONCRETE INDUSTRIES
COUNTY: LINCOLN, NE
PROJECT: CUSTOMER STOCK 201
CUSTOMER PO: 8000

Gentlemen:

This is to certify that the materials used, the preparation of the bars, coating and curing were done in accordance with the Nebraska State Highway Department Specifications for Epoxy Coated Reinforcing Steel (6-14-0379) for the above referenced project. No rebar contains more than two (2) holidays per lineal foot.

<table>
<thead>
<tr>
<th>MILL</th>
<th>SIZE</th>
<th>HEAT</th>
<th>WEIGHT</th>
<th>LOT NO.</th>
<th>POWDER</th>
</tr>
</thead>
<tbody>
<tr>
<td>CMC-TX</td>
<td>#5(10MM)</td>
<td>3019166</td>
<td>24,406</td>
<td>H1010057437</td>
<td>DUPONT</td>
</tr>
<tr>
<td>CMC-TX</td>
<td>#6(19MM)</td>
<td>3011610</td>
<td>12,016</td>
<td>H1010057848</td>
<td>DUPONT</td>
</tr>
</tbody>
</table>

TOTAL 30,422 #

STATE OF OKLAHOMA
COUNTY OF ROGERS

SUBSCRIBED AND SWORN TO BEFORE ME, a Notary Public in and for said County and State, on this the 18th Day of January, 2011.

Notary Public in and for ROGERS County
State of Oklahoma

My commission No. 02012302 expires 8-24-2014.

Figure H-6. Reinforcing Steel Specifications, Test Nos. WEAB-9 Through WEAB-13
TO WHOM IT MAY CONCERN:

This is to certify that the batch number of Nap-Gard 7-2719 Rebar listed below is chemically the same material that was tested by Valley Forge Laboratories Inc. of Devon, Pennsylvania to A775. I certify that it meets the requirements of Annex A1 of A775-00. Nap-Gard 7-2750 Rebar also meets the requirements of ASTM D3963-93a and AASHTO M284-95.

The following batch was manufactured in the United States.

<table>
<thead>
<tr>
<th>Lot Number</th>
<th>Batch Number</th>
<th>Batch Size (Lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1010057848</td>
<td>071001101021B</td>
<td>5000</td>
</tr>
</tbody>
</table>

Sincerely,

[Signature]

Mike Willenhegen
Quality Control Manager

WARRANTY POLICY: Seller certifies that all coatings delivered to Contractor is temporarily sands filled and meets all present quality standards presented in its current published literature. Seller reserves the right to make all modifications to the coating as reasonably required to satisfy contractual obligations. The Seller shall not be liable for any damages, direct or indirect, that may result from the use of the product, whether it is of the Seller's or the Buyer's manufacture. Buyer assumes all liability for the final coating, including, but not limited to, the Seller's product, the Buyer's product, and any other products used in conjunction with the Seller's product.

Figure H-7. Reinforcing Steel Specifications, Test Nos. WEAB-9 Through WEAB-13
Appendix I. Bogie Test Results
Figure I-1. Results of Test No. WEAB-1 (EDR-3)
Figure I-2. Results of Test No. WEAB-1 (DTS)
Figure I-3. Results of Test No. WEAB-2 (EDR-3)
### Test Information

<table>
<thead>
<tr>
<th>Test Number:</th>
<th>WEAB-2</th>
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<tbody>
<tr>
<td>Test Date:</td>
<td>4-Jan-201</td>
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<tr>
<td>Failure Type:</td>
<td>Steel Rupture</td>
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</tbody>
</table>

### Anchor Properties

<table>
<thead>
<tr>
<th>Anchor Test Type:</th>
<th>Single Tensile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Size:</td>
<td>5/8 in. (15.88 mm)</td>
</tr>
<tr>
<td>Anchor Coating:</td>
<td>None</td>
</tr>
<tr>
<td>Embedment Depth:</td>
<td>5.25 in. (13.3 cm)</td>
</tr>
<tr>
<td>Bonding Agent:</td>
<td>Hilti HIT-RE 500</td>
</tr>
</tbody>
</table>

### Soil Properties

| Gradation:  | NA |
| Moisture Content: | NA |
| Compaction Method: | NA |
| Soil Density, γd: | NA |

### Bogie Properties

| Impact Velocity: | 10.4 mph (15.3 fps) |
| Impact Height:   | 21.625 in. (54.9 cm) |
| Bogie Mass:      | 1484.6 lbs (673.4 kg) |

### Data Acquired

| Acceleration Data: | DTS |
| Camera Data:       | AOS-5 Perpendicular - 104 1/2" |
|                    | AOS-6 Perpendicular - 145" |

### Test Results Summary

| Max Deflection: | 3.1 in. |
| Peak Force:     | 16.9 k |
| Total Energy:   | 36.0 k-in. |

### Test Results Summary

<table>
<thead>
<tr>
<th>Force vs. Deflection At Impact Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection (in.)</td>
</tr>
<tr>
<td>Time (s)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bogie Acceleration vs. Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acceleration (g's)</td>
</tr>
<tr>
<td>Time (s)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bogie Velocity vs. Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity (ft/s)</td>
</tr>
<tr>
<td>Time (s)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Energy vs. Deflection At Impact Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection (in.)</td>
</tr>
<tr>
<td>Time (s)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Deflection at Impact Location vs. Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection (in.)</td>
</tr>
<tr>
<td>Time (s)</td>
</tr>
</tbody>
</table>

Figure I-4. Results of Test No. WEAB-2 (DTS)
### Test Information

| Test Number: | WEAB-3 |
| Test Date: | 4-Jan-2011 |
| Failre Type: | Steel Rupture |

### Anchor Properties

- Anchor Test Type: Single Tensile
- Anchor Size: 5/8 in. 15.88 mm
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. 13.3 cm
- Bonding Agent: Hilti HIT-RE 500

### Soil Properties

- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, γd: NA

### Bogie Properties

- Impact Velocity: 9.8 mph (14.4 fps) 4.38 m/s
- Impact Height: 21.625 in. 54.9 cm
- Bogie Mass: 1484.6 lbs 673.4 kg

### Data Acquired

- Acceleration Data: EDR-3
- Camera Data: AOS-5 Perpendicular - 87 1/2°
  AOS-6 Perpendicular - 139°

### Test Results Summary

| Test Results Summary | Max. Deflection: 3.6 in. |
| | Peak Force: 15.2 k |
| | Total Energy: 23.7 k-in. |

### Graphs

- **Force vs. Deflection At Impact Location**
- **Bogie Acceleration vs. Time**
- **Bogie Velocity vs. Time**
- **Energy vs. Deflection At Impact Location**
- **Deflection at Impact Location vs. Time**

---

Figure I-5. Results of Test No. WEAB-3 (EDR-3)
## MIDWEST ROADSIDE SAFETY FACILITY

### Test Information

<table>
<thead>
<tr>
<th>Test Number</th>
<th>WEAB-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Date</td>
<td>4-Jan-2011</td>
</tr>
<tr>
<td>Failure Type</td>
<td>Steel Rupture</td>
</tr>
</tbody>
</table>

### Anchor Properties

<table>
<thead>
<tr>
<th>Anchor Test Type</th>
<th>Single Tensile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Size</td>
<td>5/8 in.</td>
</tr>
<tr>
<td>Anchor Coating</td>
<td>Epoxy</td>
</tr>
<tr>
<td>Embedment Depth</td>
<td>5.25 in.</td>
</tr>
<tr>
<td>Bonding Agent</td>
<td>Hilti HIT-RE 500</td>
</tr>
</tbody>
</table>

### Soil Properties

- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, γd: NA

### Bogie Properties

- Impact Velocity: 9.8 mph (14.4 fps)
- Impact Height: 21.625 in. 54.9 cm
- Bogie Mass: 1484.6 lbs 673.4 kg

### Data Acquired

- Acceleration Data: DTS
- Camera Data: AOS-5 Perpendicular - 87 1/2°
  AOS-6 Perpendicular - 139°

### Test Results Summary

<table>
<thead>
<tr>
<th>Max. Deflection</th>
<th>3.4 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Force</td>
<td>15.1 k</td>
</tr>
<tr>
<td>Total Energy</td>
<td>23.0 k-in.</td>
</tr>
</tbody>
</table>

### Graphs

- **Bogie Acceleration vs. Time**
- **Bogie Velocity vs. Time**
- **Force vs. Deflection At Impact Location**
- **Energy vs. Deflection At Impact Location**
- **Deflection at Impact Location vs. Time**

Figure I-6. Results of Test No. WEAB-3 (DTS)
**Figure I-7. Results of Test No. WEAB-4 (EDR-3)**
Figure I-8. Results of Test No. WEAB-4 (DTS)
### Test Information

<table>
<thead>
<tr>
<th>Test Number</th>
<th>WEAB-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Date</td>
<td>6-Jan-2011</td>
</tr>
<tr>
<td>Failure Type</td>
<td>Steel Shear Fracture</td>
</tr>
<tr>
<td>Anchor Test Type</td>
<td>Single Shear</td>
</tr>
<tr>
<td>Anchor Size</td>
<td>5/8 in. 15.88 mm</td>
</tr>
<tr>
<td>Anchor Coating</td>
<td>Epoxy</td>
</tr>
<tr>
<td>Embedment Depth</td>
<td>5.25 in. 13.3 cm</td>
</tr>
<tr>
<td>Bonding Agent</td>
<td>Hilti HIT-RE 500</td>
</tr>
</tbody>
</table>

### Soil Properties

- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, $\gamma_d$: NA

### Bogie Properties

- Impact Velocity: 9.64 mph (14.1 fps) 4.31 m/s
- Impact Height: 7.3125 in. 18.6 cm
- Bogie Mass: 1734.6 lbs 786.8 kg

### Data Acquired

- Acceleration Data: EDR-3
- Camera Data: AOS-5 Perpendicular - 202°, AOS-6 Perpendicular - 43°

### Test Results Summary

<table>
<thead>
<tr>
<th>Test Results Summary</th>
<th>Max. Deflection: 2.1 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Force: 25.7 k</td>
</tr>
<tr>
<td></td>
<td>Total Energy: 29.1 k-in.</td>
</tr>
</tbody>
</table>

### Force vs. Deflection At Impact Location

![Force vs. Deflection At Impact Location](image)

### Energy vs. Deflection At Impact Location

![Energy vs. Deflection At Impact Location](image)

### Deflection at Impact Location vs. Time

![Deflection at Impact Location vs. Time](image)

### Bogie Acceleration vs. Time

![Bogie Acceleration vs. Time](image)

### Bogie Velocity vs. Time

![Bogie Velocity vs. Time](image)

Figure I-9. Results of Test No. WEAB-5 (EDR-3)
<table>
<thead>
<tr>
<th>Test Information</th>
<th>Test Results Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Number:</td>
<td>WEAB-5</td>
</tr>
<tr>
<td>Test Date:</td>
<td>6-Jan-2011</td>
</tr>
<tr>
<td>Failure Type:</td>
<td>Steel Shear Fracture</td>
</tr>
</tbody>
</table>

**Anchor Properties**
- Anchor Test Type: Single Shear
- Anchor Size: 5/8 in. (15.88 mm)
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. (13.3 cm)
- Bonding Agent: Hilti HIT-RE 500

**Soil Properties**
- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, y'd: NA

**Bogie Properties**
- Impact Velocity: 9.64 mph (14.1 fps), 4.33 m/s
- Impact Height: 7.3125 in. (18.6 cm)
- Bogie Mass: 1734.6 lbs (786.8 kg)

**Data Acquired**
- Acceleration Data: DTS
- Camera Data: AOS-5 Perpendicular - 202°
  - AOS-6 Perpendicular - 43°

**Graphs**
- Force vs. Deflection At Impact Location
- Energy vs. Deflection At Impact Location
- Bogie Acceleration vs. Time
- Bogie Velocity vs. Time
- Deflection at Impact Location vs. Time

Figure I-10. Results of Test No. WEAB-5 (DTS)
Figure I-11. Results of Test No. WEAB-6 (EDR-3)
**Test Results Summary**

<table>
<thead>
<tr>
<th>Test Information</th>
<th>Test Results Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Number: WEAB-6</td>
<td>Max. Deflection: 1.7 in.</td>
</tr>
<tr>
<td>Test Date: 6-Jan-2011</td>
<td>Peak Force: 28.4 k</td>
</tr>
<tr>
<td>Failure Type: Steel Shear Fracture</td>
<td>Total Energy: 27.3 k-in.</td>
</tr>
</tbody>
</table>

**Anchor Properties**
- Anchor Test Type: Single Shear
- Anchor Size: 5/8 in. 15.88 mm
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. 13.3 cm
- Bonding Agent: Hilti HIT-RE 500

**Soil Properties**
- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, y'd: NA

**Bogie Properties**
- Impact Velocity: 9.71 mph (14.2 fps) 4.34 m/s
- Impact Height: 7.3125 in. 18.6 cm
- Bogie Mass: 1734.6 lbs 786.8 kg

**Data Acquired**
- Acceleration Data: DTS
- Camera Data: AOS-5 Perpendicular - 168°
- AOS-6 Perpendicular - 97 1/2°

---

**Figure I-12. Results of Test No. WEAB-6 (DTS)**
**Figure I-13. Results of Test No. WEAB-7 (EDR-3)**
Figure I-14. Results of Test No. WEAB-7 (DTS)
Figure I-15. Results of Test No. WEAB-8 (EDR-3)
### Test Results Summary

<table>
<thead>
<tr>
<th>Test Number</th>
<th>WEAB-8</th>
<th>Max. Deflection</th>
<th>7.4 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Date</td>
<td>6-Jan-2011</td>
<td>Peak Force</td>
<td>31.3 k</td>
</tr>
<tr>
<td>Failure Type</td>
<td>Steel Rupture, Anchor Pullout</td>
<td>Total Energy</td>
<td>63.5 k-in.</td>
</tr>
</tbody>
</table>

### Anchor Properties

<table>
<thead>
<tr>
<th>Anchor Test Type</th>
<th>Double Tensile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Size</td>
<td>5/8 in. 15.88 mm</td>
</tr>
<tr>
<td>Anchor Coating</td>
<td>Epoxy</td>
</tr>
<tr>
<td>Embedment Depth</td>
<td>5.25 in. 13.3 cm</td>
</tr>
<tr>
<td>Bonding Agent</td>
<td>Hilti HIT-RE 500</td>
</tr>
</tbody>
</table>

### Soil Properties

- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, γd: NA

### Bogie Properties

- Impact Velocity: 14.05 mph (20.6 fps) 6.28 m/s
- Impact Height: 21.625 in. 54.9 cm
- Bogie Mass: 1484.6 lbs 673.4 kg

### Data Acquired

- Acceleration Data: DTS
- Camera Data: AOS-5 Perpendicular - 82°
  - AOS-6 Perpendicular - 129°

### Graphs

- **Force vs. Deflection At Impact Location**
- **Bogie Acceleration vs. Time**
- **Bogie Velocity vs. Time**
- **Energy vs. Deflection At Impact Location**
- **Deflection at Impact Location vs. Time**

---

**Figure I-16. Results of Test No. WEAB-8 (DTS)**
Figure I-17. Results of Test No. WEAB-9 (EDR-3)
Figure I-18. Results of Test No. WEAB-9 (DTS)
**Figure I-19. Results of Test No. WEAB-10 (EDR-3)**

<table>
<thead>
<tr>
<th>Test Information</th>
<th>Test Results Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Number: WEAB-10</td>
<td>Max. Deflection: 9.3 in.</td>
</tr>
<tr>
<td>Test Date: 6-Apr-2011</td>
<td>Peak Force: 18.5 k</td>
</tr>
<tr>
<td>Failure Type: Anchor Pullout</td>
<td>Total Energy: 41.8 k-in.</td>
</tr>
</tbody>
</table>

**Anchor Properties**
- Anchor Test Type: Single Tensile
- Anchor Size: 3/4 in. (19.05 mm)
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. (13.3 cm)
- Bonding Agent: Hilti HIT-RE 500-SD

**Soil Properties**
- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, γₕ: NA

**Bogie Properties**
- Impact Velocity: 15.73 mph (23.1 fps) or 7.03 m/s
- Impact Height: 21.625 in. (54.9 cm)
- Bogie Mass: 1726.6 lbs (783.2 kg)

**Data Acquired**
- Acceleration Data: EDR-3
- Camera Data: AOS-5 Perpendicular - 146°
  - AOS-6 Perpendicular - 55°

---

![Force vs. Deflection At Impact Location](image1)

![Bogie Acceleration vs. Time](image2)

![Bogie Velocity vs. Time](image3)

![Energy vs. Deflection At Impact Location](image4)

![Deflection at Impact Location vs. Time](image5)
Test Information
- Test Number: WEAB-10
- Test Date: 6-Apr-2011
- Failure Type: Anchor Pullout

Anchor Properties
- Anchor Test Type: Single Tensile
- Anchor Size: 3/4 in. (19.05 mm)
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. (13.3 cm)
- Bonding Agent: Hilti HIT-RE 500-SD

Soil Properties
- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, ρd: NA

Bogie Properties
- Impact Velocity: 15.73 mph (23.1 fps) (7.03 m/s)
- Impact Height: 21.625 in. (54.9 cm)
- Bogie Mass: 1726.6 lbs (783.2 kg)

Data Acquired
- Acceleration Data: DTS
- Camera Data: AOS-5 Perpendicular - 146°

Test Results Summary
- Max. Deflection: 8.8 in.
- Peak Force: 19.1 k
- Total Energy: 38.4 k-in.

Figure I-20. Results of Test No. WEAB-10 (DTS Set 1)
### Test Information

<table>
<thead>
<tr>
<th>Anchor Properties</th>
<th>Soil Properties</th>
<th>Bogie Test Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Test Type: Single Tensile</td>
<td>Gradation: NA</td>
<td>Test Number: WEAB-10</td>
</tr>
<tr>
<td>Anchor Size: 3/4 in. 19.05 mm</td>
<td>Moisture Content: NA</td>
<td>Max. Deflection: 8.7 in.</td>
</tr>
<tr>
<td>Anchor Coating: Epoxy</td>
<td>Compaction Method: NA</td>
<td>Peak Force: 19.2 k</td>
</tr>
<tr>
<td>Embedment Depth: 5.25 in. 133 cm</td>
<td>Soil Density, γd: NA</td>
<td>Total Energy: 38.5 k-in.</td>
</tr>
<tr>
<td>Bonding Agent: Hilti HIT-RE 500-SD</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Test Results Summary

- **Test Number**: WEAB-10
- **Max. Deflection**: 8.7 in.
- **Test Date**: 6-Apr-2011
- **Peak Force**: 19.2 k
- **Failure Type**: Anchor Pullout
- **Total Energy**: 38.5 k-in.
- **Anchor Size**: 3/4 in. 19.05 mm
- **Anchor Coating**: Epoxy
- **Embedment Depth**: 5.25 in. 133 cm
- **Bonding Agent**: Hilti HIT-RE 500-SD
- **Gradation**: NA
- **Moisture Content**: NA
- **Compaction Method**: NA
- **Soil Density, γd**: NA
- **Impact Velocity**: 15.73 mph (23.1 fps) 7.03 m/s
- **Impact Height**: 21.625 in. 54.9 cm
- **Bogie Mass**: 1726.6 lbs 783.2 kg
- **Acceleration Data**: DTS
- **Camera Data**: AOS-5 Perpendicular - 146°
  - AOS-6 Perpendicular - 55°

### Graphs

1. **Force vs. Deflection At Impact Location**
2. **Energy vs. Deflection At Impact Location**
3. **Bogie Acceleration vs. Time**
4. **Deflection at Impact Location vs. Time**
5. **Bogie Velocity vs. Time**
6. **Deflection at Impact Location vs. Time**

Figure I-21. Results of Test No. WEAB-10 (DTS Set 2)
**Figure I-22. Results of Test No. WEAB-11 (EDR-3)**

<table>
<thead>
<tr>
<th>Test Information</th>
<th>Test Results Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Number: WEAB-11</td>
<td></td>
</tr>
<tr>
<td>Test Date: 6-Apr-2011</td>
<td></td>
</tr>
<tr>
<td>Failure Type: Anchor Pullout/Concrete Cone Failure</td>
<td></td>
</tr>
<tr>
<td>Max. Deflection: 3.0 in.</td>
<td></td>
</tr>
<tr>
<td>Peak Force: 26.4 k</td>
<td></td>
</tr>
<tr>
<td>Total Energy: 36.1 k-in.</td>
<td></td>
</tr>
</tbody>
</table>

**Anchor Properties**
- Anchor Test Type: Double Tensile
- Anchor Size: 3/4 in. 19.05 mm
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. 13.3 cm
- Bonding Agent: Hilti HIT-RE 500-SD

**Soil Properties**
- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, γd: NA

**Bogie Properties**
- Impact Velocity: 15.12 mph (22.2 fps) 6.76 m/s
- Impact Height: 21.625 in. 54.9 cm
- Bogie Mass: 1726.6 lbs 783.2 kg

**Data Acquired**
- Acceleration Data: EDR-3
- Camera Data: AOS-5 Perpendicular AOS-6 Perpendicular

**Graphs**
- Bogie Acceleration vs. Time
- Bogie Velocity vs. Time
- Force vs. Deflection At Impact Location
- Energy vs. Deflection At Impact Location
- Deflection at Impact Location vs. Time
**Test Information**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>Test Number</td>
<td>WEAB-11</td>
</tr>
<tr>
<td>Max. Deflection</td>
<td>8.6 in.</td>
</tr>
<tr>
<td>Test Date</td>
<td>6-Apr-2011</td>
</tr>
<tr>
<td>Peak Force</td>
<td>26.2 k</td>
</tr>
<tr>
<td>Failure Type</td>
<td>Anchor Pullout/Concrete Cone Failure</td>
</tr>
<tr>
<td>Total Energy</td>
<td>33.3 k-in.</td>
</tr>
<tr>
<td>Anchor Test Type</td>
<td>Double Tensile</td>
</tr>
<tr>
<td>Anchor Size</td>
<td>3/4 in. 19.05 mm</td>
</tr>
<tr>
<td>Anchor Coating</td>
<td>Epoxy</td>
</tr>
<tr>
<td>Embedment Depth</td>
<td>5.25 in. 13.3 cm</td>
</tr>
<tr>
<td>Bonding Agent</td>
<td>Hilti HIT-RE 500-SD</td>
</tr>
<tr>
<td>Gradation</td>
<td>NA</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>NA</td>
</tr>
<tr>
<td>Compaction Method</td>
<td>NA</td>
</tr>
<tr>
<td>Soil Density, γd</td>
<td>NA</td>
</tr>
<tr>
<td>Impact Velocity</td>
<td>15.12 mph (22.2 fps) 6.76 m/s</td>
</tr>
<tr>
<td>Impact Height</td>
<td>21.625 in. 54.9 cm</td>
</tr>
<tr>
<td>Bogie Mass</td>
<td>1726.6 lbs 783.2 kg</td>
</tr>
</tbody>
</table>

**Anchor Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Test Type</td>
<td>Double Tensile</td>
</tr>
<tr>
<td>Anchor Size</td>
<td>3/4 in. 19.05 mm</td>
</tr>
<tr>
<td>Anchor Coating</td>
<td>Epoxy</td>
</tr>
<tr>
<td>Embedment Depth</td>
<td>5.25 in. 13.3 cm</td>
</tr>
<tr>
<td>Bonding Agent</td>
<td>Hilti HIT-RE 500-SD</td>
</tr>
</tbody>
</table>

**Soil Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gradation</td>
<td>NA</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>NA</td>
</tr>
<tr>
<td>Compaction Method</td>
<td>NA</td>
</tr>
<tr>
<td>Soil Density, γd</td>
<td>NA</td>
</tr>
</tbody>
</table>

**Bogie Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impact Velocity</td>
<td>15.12 mph (22.2 fps) 6.76 m/s</td>
</tr>
<tr>
<td>Impact Height</td>
<td>21.625 in. 54.9 cm</td>
</tr>
<tr>
<td>Bogie Mass</td>
<td>1726.6 lbs 783.2 kg</td>
</tr>
</tbody>
</table>

**Data Acquired**

<table>
<thead>
<tr>
<th>Data Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acceleration Data</td>
<td>DTS</td>
</tr>
<tr>
<td>Camera Data</td>
<td>AOS-5 Perpendicular</td>
</tr>
<tr>
<td></td>
<td>AOS-6 Perpendicular</td>
</tr>
</tbody>
</table>

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**Figure I-23. Results of Test No. WEAB-11 (DTS Set 1)**
## Test Results Summary

<table>
<thead>
<tr>
<th>Test Information</th>
<th>Wisconsin Epoxy Concrete Anchor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Number:</td>
<td>WEAB-11</td>
</tr>
<tr>
<td>Test Date:</td>
<td>6-Apr-2011</td>
</tr>
<tr>
<td>Failure Type:</td>
<td>Anchor Pullout/Concrete Cone Failure</td>
</tr>
</tbody>
</table>

### Anchor Properties

- Anchor Test Type: Double Tensile
- Anchor Size: 3/4 in. (19.05 mm)
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. (13.3 cm)
- Bonding Agent: Hilti HIT-RE 500-SD

### Soil Properties

- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, $\gamma_d$: NA

### Bogie Properties

- Impact Velocity: 15.12 mph (22.2 fps) 6.76 m/s
- Impact Height: 21.625 in. (54.9 cm)
- Bogie Mass: 1726.6 lbs (783.2 kg)

### Data Acquired

- Acceleration Data: DTS
- Camera Data: AOS-5 Perpendicular, AOS-6 Perpendicular

### Force vs. Deflection At Impact Location

### Energy vs. Deflection At Impact Location

### Deflection at Impact Location vs. Time

---

Figure I-24. Results of Test No. WEAB-11 (DTS Set 2)
Figure I-25. Results of Test No. WEAB-12 (EDR-3)
<table>
<thead>
<tr>
<th>Test Information</th>
<th>Test Results Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Number:</td>
<td>Max. Deflection: 8.1 in.</td>
</tr>
<tr>
<td>Test Date:</td>
<td>Peak Force: 32.8 k</td>
</tr>
<tr>
<td>Failure Type:</td>
<td>Total Energy: 55.2 k-in.</td>
</tr>
</tbody>
</table>

**Anchor Properties**
- Anchor Test Type: Double Tensile
- Anchor Size: 3/4 in. 19.05 mm
- Anchor Coating: Epoxy
- Embedment Depth: 5.25 in. 13.3 cm
- Bonding Agent: Hilti HIT-RE 500-SD

**Soil Properties**
- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, $\gamma_d$: NA

**Bogie Properties**
- Impact Velocity: 15.08 mph (22.1 fps) 6.74 m/s
- Impact Height: 21.625 in. 54.9 cm
- Bogie Mass: 1726.6 lbs 783.2 kg

**Data Acquired**
- Acceleration Data: DTS
- Camera Data: AOS-5 Perpendicular - 112°
- AOS-6 Perpendicular - 77°

![Bogie Acceleration vs. Time](image)

![Bogie Velocity vs. Time](image)

![Force vs. Deflection At Impact Location](image)

![Energy vs. Deflection At Impact Location](image)

![Deflection at Impact Location vs. Time](image)

Figure I-26. Results of Test No. WEAB-12 (DTS Set 1)
Figure I-27. Results of Test No. WEAB-12 (DTS Set 2)
Figure I-28. Results of Test No. WEAB-13 (EDR-3)
Figure I-29. Results of Test No. WEAB-13 (DTS Set 1)
**Figure I-30. Results of Test No. WEAB-13 (DTS Set 2)**
Figure I-31. Results of Test No. WEAB-14 (EDR-3)
Figure I-32. Results of Test No. WEAB-14 (DTS Set 1)
Figure I-33. Results of Test No. WEAB-14 (DTS Set 2)
# Figure I-34. Results of Test No. WEAB-15 (EDR-3)

## Test Results Summary

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Max. Deflection</th>
<th>Test Date</th>
<th>Peak Force</th>
<th>Total Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-15</td>
<td>2.2 in.</td>
<td>17-Jun-2011</td>
<td>43.6 k</td>
<td>56.0 k-in.</td>
</tr>
</tbody>
</table>

## Anchor Properties
- **Anchor Test Type**: Single Shear
- **Anchor Size**: 1 1/8 in. (28.58 mm)
- **Anchor Coating**: None
- **Embedment Depth**: 5.25 in. (13.3 cm)
- **Bonding Agent**: Hilti HIT-RE 500-SD

## Soil Properties
- **Gradation**: NA
- **Moisture Content**: NA
- **Compaction Method**: NA
- **Soil Density, γd**: NA

## Bogie Properties
- **Impact Velocity**: 9.28 mph (13.6 fps) / 4.15 m/s
- **Impact Height**: 7.3125 in. (18.6 cm)
- **Bogie Mass**: 1740.6 lbs (789.5 kg)

## Data Acquired
- **Acceleration Data**: EDR-3
- **Camera Data**: AOS-5 Perpendicular - 153°
  - AOS-7 Perpendicular - 98°

## Graphs
- **Force vs. Deflection At Impact Location**
- **Bogie Acceleration vs. Time**
- **Bogie Velocity vs. Time**
- **Energy vs. Deflection At Impact Location**
- **Deflection at Impact Location vs. Time**
Figure I-35. Results of Test No. WEAB-15 (DTS Set 1)
Figure I-36. Results of Test No. WEAB-15 (DTS Set 2)
Figure I-37. Results of Test No. WEAB-16 (EDR-3)
**MIDWEST ROADSIDE SAFETY FACILITY**

### Test Information

<table>
<thead>
<tr>
<th>Test Number</th>
<th>WEAB-16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Date</td>
<td>25-Jul-2011</td>
</tr>
<tr>
<td>Failure Type</td>
<td>Anchor Pullout/Concrete Breakout</td>
</tr>
</tbody>
</table>

### Anchor Properties

- Anchor Test Type: Single Tensile
- Anchor Size: 3/4 in. 19.05 mm
- Anchor Coating: None
- Embedment Depth: 5.25 in. 13.3 cm
- Bonding Agent: Hilti HIT-RE 500-SD

### Soil Properties

- Gradation: NA
- Moisture Content: NA
- Compaction Method: NA
- Soil Density, γd: NA

### Bogie Properties

- Impact Velocity: 15.9 mph (23.3 fps) 7.11 m/s
- Impact Height: 21.625 in. 54.9 cm
- Bogie Mass: 1722.6 lbs 781.4 kg

### Data Acquired

- Acceleration Data: DTS
- Camera Data: AOS-6 Perpendicular - 62.5°
  - AOS-7 Perpendicular - 90°

### Test Results Summary

<table>
<thead>
<tr>
<th>Max. Deflection</th>
<th>6.2 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Force</td>
<td>20.4 k</td>
</tr>
<tr>
<td>Total Energy</td>
<td>44.3 k-in.</td>
</tr>
</tbody>
</table>

---

**Figure I-38. Results of Test No. WEAB-16 (DTS Set 1)**
**MIDWEST ROADSIDE SAFETY FACILITY**

### Test Number Summary

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Max. Deflection</th>
<th>Peak Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEAB-16</td>
<td>6.2 in.</td>
<td>19.5 k</td>
</tr>
</tbody>
</table>

### Test Information

- **Wisconsin Epoxy Concrete Anchor**
- **Test Number:** WEAB-16
- **Test Date:** 27-Jul-2011
- **Failure Type:** Anchor Pullout/Concrete Breakout

### Anchor Properties

- **Anchor Test Type:** Single Tensile
- **Anchor Size:** 3/4 in. / 9.05 mm
- **Anchor Coating:** None
- **Embedment Depth:** 5.25 in. / 13.3 cm
- **Bonding Agent:** Hilti HIT-RE 500-SD

### Soil Properties

- **Gradation:** NA
- **Moisture Content:** NA
- **Compaction Method:** NA
- **Soil Density, γd:** NA

### Bogie Properties

- **Impact Velocity:** 15.9 mph (23.3 fps) / 7.11 m/s
- **Impact Height:** 21.625 in. / 54.9 cm
- **Bogie Mass:** 1722.6 lbs / 781.4 kg

### Data Acquired

- **Acceleration Data:** DTS
- **Camera Data:** AOS-6 Perpendicular - 62 1/2" / AOS-7 Perpendicular - 96"

### Graphs

- **Force vs. Deflection At Impact Location**
- **Bogie Acceleration vs. Time**
- **Bogie Velocity vs. Time**
- **Energy vs. Deflection At Impact Location**
- **Deflection at Impact Location vs. Time**

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Figure I-39. Results of Test No. WEAB-16 (DTS Set 2)