SIGMA INSULATED PANELS
PIN CONNECTION REVIEW

Portland, Oregon

STRUCTURAL CALCULATIONS

VLMK Project Number: 20170221

Sigma DG
5019 NW 127th St.
Vancouver, WA 98685

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
56359PE
4/28/17
OREGON
JAN. 15, 2002
JASON C. SAHLIN
EXPIRES: 6/30/2018

Prepared By: Reed Newcomer, PE
April 2017

VLMK
ENGINEERING + DESIGN

Project:	Sigma Insulated Panels	*Project Number:*	20170221
Project Address:	Portland, Oregon	*Document:*	Structural Calculations for Building Permit

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DESCRIPTION OF STRUCTURAL SYSTEM

This 'Sigma Insulated Panels' consists of creating standard details for attaching non-composite insulated panels together with Sigma pins for projects located in the Pacific Northwest. The systems consist of an architectural facia panel connected to a concrete structural panel by the Sigma pins separated by an insulation layer. The structural panel supports any out of plane loading of the facia, and lifting loads during construction. The pins are the main element transferring these loads into the structural panels. Specifically, a verification was conducted on a 2 ½" facia with 2" of insulation for a project located near the Portland, Oregon Region. The use of this product on any specific project will need to be verified and provided by the Engineer of Record for their specific project and location.

LIMITATIONS

VLMK Engineering + Design was retained in a limited capacity for this project. The design is based upon information provided by the client, who is solely responsible for the accuracy of the information. No responsibility and/or liability is assumed by, nor is any to be assigned to, VLMK Engineering + Design for items beyond that shown in this Structural Calculation Package.

3933 SW Kelly Avenue Portland, OR 97239 tel: 503.222.4453 fax: 503.248.9263 www.vlmk.com

CODES

2012 International Building Code

DESIGN LOADS

Dead Loads

Wall Weights

Concrete Tilt Panel	12.5 psf/in
Rigid Insulation	0.25 psf/in

Wind (approx.)

Ultimate Design Wind Speed, V_{ult}	120 mph
Nominal Design Wind Speed, V_{asd}	93 mph
Risk Category	II
Wind Exposure	C
Internal Pressure Coefficient	GC_{pi} = +/- 0.18

Seismic (approx.)

Seismic Importance Factor, I_e	1.0
Risk Category	II
Mapped Spectral Response Accelerations	$S_s = 0.98$
	$S_1 = 0.42$
Site Class	D
Spectral Response Coefficients	$S_{ds} = 0.75$
	$S_{d1} = 0.45$

MATERIALS

Concrete

Tilt Panels	f'_c = 4,000 psi

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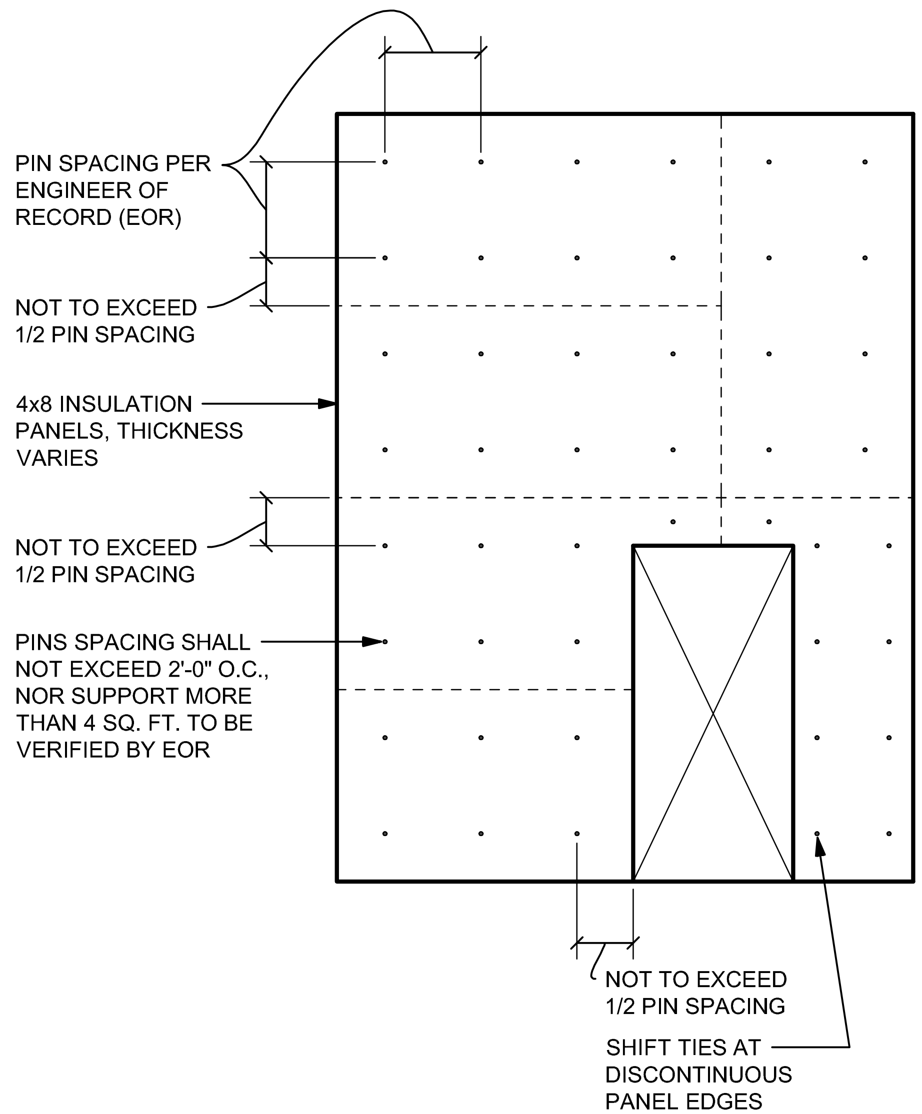

A
P100
TYPICAL INSULATION PANEL LAYOUT
P100 1/4" = 1'-0"

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STANDARD DETAILS

DATE	APR, 2017
SCALE	AS NOTED
PROJ. NO.	20170221
DRAWN	JRN
CHECKED	JCS

SIGMA DG CORPORATION

P100

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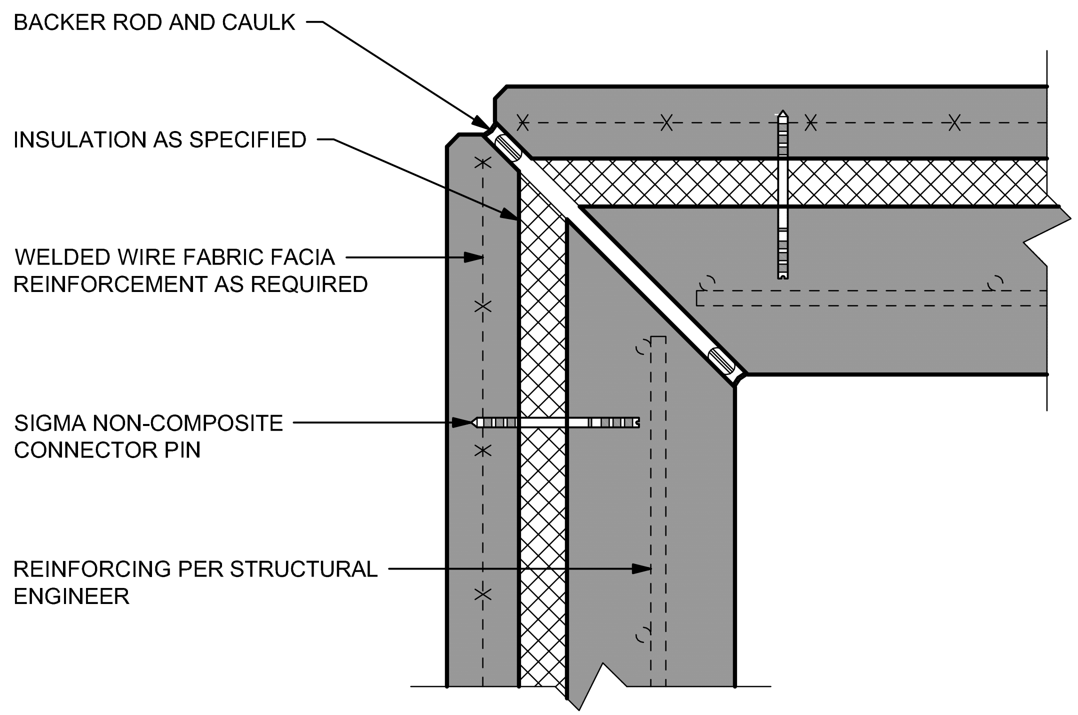

A / P101 EXT. CORNER - MITERED

P101 1-1/2" = 1'-0"

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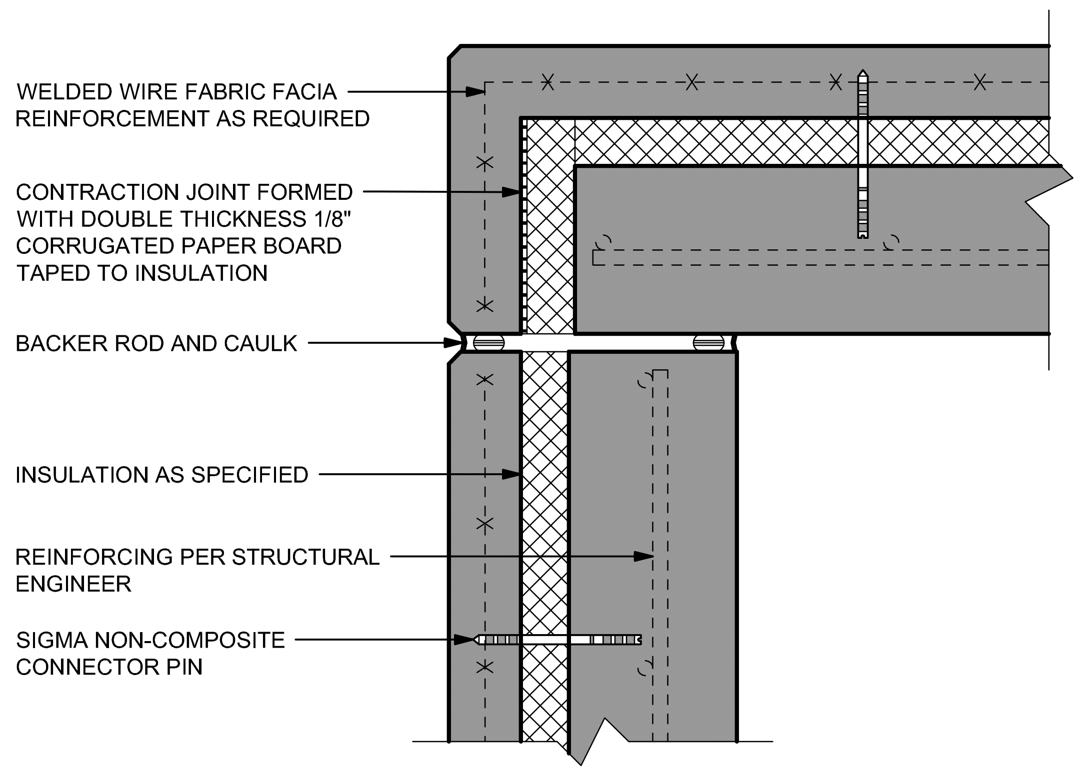

A
P102
EXT. CORNER - 90° BUTT JOINT

P102 1-1/2" = 1'-0"

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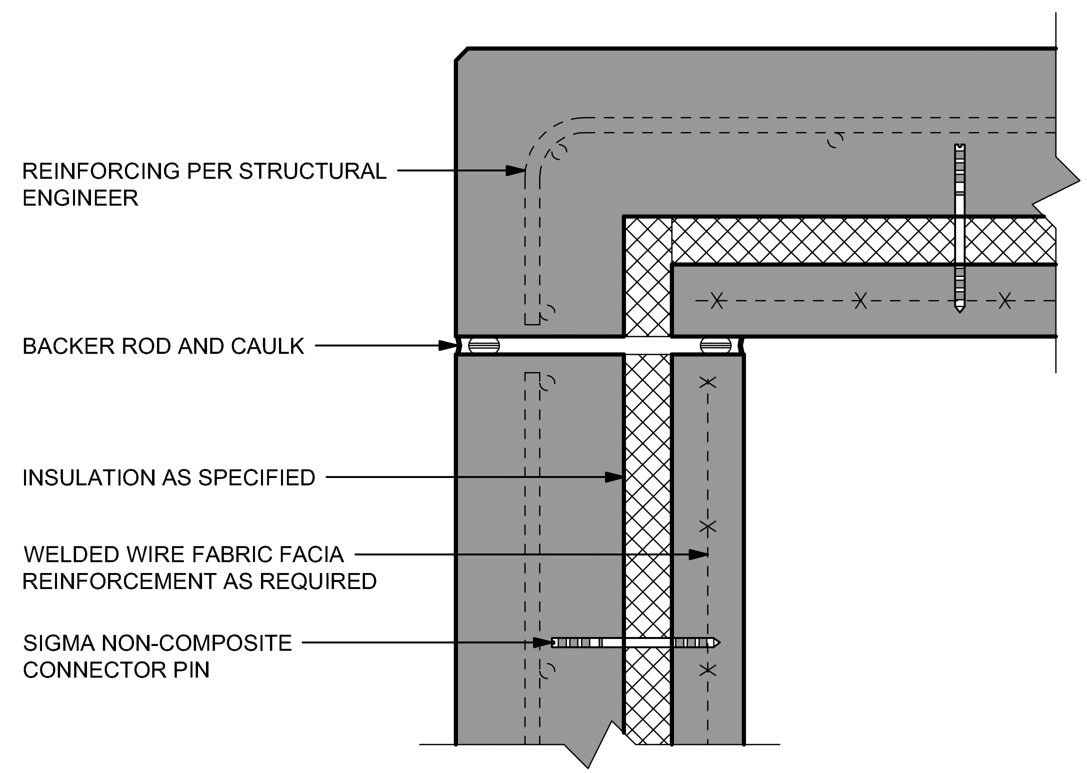

A
P103

EXT. CORNER - 90° INTERIOR BUTT JOINT

P103 1-1/2" = 1'-0"

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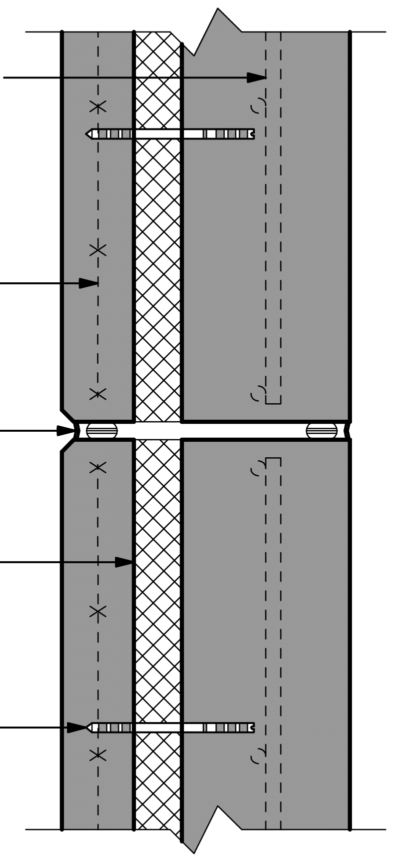

A
P104

TYPICAL PANEL JOINT

P104 1-1/2" = 1'-0"

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SIGMA PINS

THE PINS THAT HOLD SANDWICH PANELS ARE BEING EVALUATED FOR THE PANEL LIFTING CONDITION, AND OUT OF PLANE WIND/SEISMIC FORCES. WIND + SEISMIC LOADS ARE ASSUMED TO BE FROM THE PORTLAND OREGON REGION. THE MAXIMUM FACIA WALL IS ASSUMED TO NOT EXCEED 3½" + THE INSULATION IS ASSUMED NOT TO EXCEED 2".

LOAD CASE I - PANEL LIFTED FROM GROUND

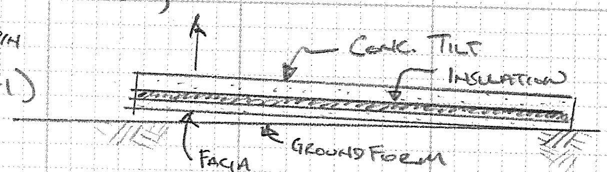
TILT PANELS ARE TYPICALLY CAST W/ FACIA SIDE DOWN, IF THE FACIA IS CAST UPWARDS THE FAILURE MODE CHANGES. THIS ANALYSIS ONLY LOOKS AT A TYPICAL CONDITION.

DEAD LOAD $= (\gamma = 0.15\text{pcf})(t = 3.5''/12)(A = 4\text{ SQ.FT}) = 175\#$ (PANEL)
$= (\gamma = 0.25\text{ PSF/IN})(t = 2\text{ IN})(A = 4\text{ SQ. FT}) = 2\#$ (INSULATION)
$177\#/\text{PIN}$

PULLOUT CAPACITY = 2025# (SIGMA PINS SPEC. SHEET R-1)

$T_{ALLOW} = \dfrac{2025\#}{F.S. = 4} = 506\# > 177\#$ ∴ OKAY

↳ ICC AC320 TABLE 7



LOAD CASE II - PANEL LIFTED UPRIGHT

DEAD LOAD = 177#/PIN

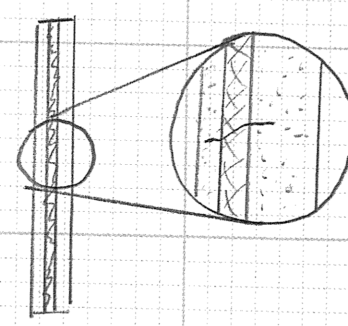
$I_x = \pi(d = 0.362\text{IN})^4/64 = 0.00084\text{IN}^4$ MOMENT OF INERTIA

$S_x = I_x/0.5d = 0.00465\text{IN}^3$ ELASTIC SECTION MODULUS

$d_A = d_d + \dfrac{2h_v}{3}\left[1 - \dfrac{1}{1 + h_v/d_d}\right] = 2'' + \dfrac{2(2'')}{3}\left(1 - \dfrac{1}{2}\right) = 2.67''$ BENDING LENGTH

$\sigma = -\dfrac{W d_A}{2 * S_x} = \dfrac{177\#(2.67\text{IN})}{2(0.00465\text{IN}^3)} = -50737\text{ psi}$

$\tau = \dfrac{W}{A} = \dfrac{177\#}{\frac{\pi\, d_d^2}{4}} = 1720\text{ psi}$



INTERACTION $= \dfrac{50737}{130500/_{F.S.=2}} + \dfrac{1720}{23200/_{F.S.=2}} = 0.93 > 1.0$ ∴ OKAY

$\Delta = \dfrac{W d_A^3}{12 E I_x} = \dfrac{177(2.67\text{IN})^3}{12(0.95 * 7687000)(0.00084\text{IN}^4)} = 0.045\text{ IN} < 0.1\text{ IN}$ ∴ OKAY

LOAD CASE II CONT.

PRYOUT STRENGTH - REFER TO ACI 318-14

$N_b = K_c \lambda \sqrt{f'_c}\ h_{ef}^{1.5} = 24(0.75)\sqrt{4000}\ (2)^{1.5} = 3220\#$ (17.4.2.2a)

$A_{NCO} = 9 h_{ef}^2 = 36 in^2 = A_{NC}$ (17.4.2.1c)

$V_{CP} = K_{CP} \frac{A_{NC}}{A_{NCO}} \psi_{ed,N} \psi_{cN} \psi_{cpN} N_b = 3220\#$ (17.5.3.1b)

$\phi V_{CP} = 0.70(3220\#) = 2254\# > (1.4)177\# = 250\#$ ∴ OKAY

LOAD CASE III - WIND / SEISMIC

$F_p = 0.4(0.75)(1.0)W = 0.3(127\#) = 38.1$ PSF <ULT> ⇒ 26.7 PSF <ASD>

WIND: 120 MPH, EXP. C, RISK CAT. II, END ZONE, $z = 40'$

$W = 31.0$ PSF (4 SQ FT) = 124# <ASD> < 500# ∴ OKAY

Sigma Connector Results

Based on a limited analysis of the pin system, they appear to have adequate capacity to resist the facia panel weight and out-of plane wind and seismic loads for typical applications.

We observed that the controlling condition is the pin stress when they cantilever out to support the facia panels gravity loads. The facia weight and cantilever distance will most likely determine the maximum pin spacing. Due to a lack of code resources for bending on fiberglass pins, we referenced AISC 360 for allowable stress design factor of saftey. AISC uses a factor of safety of 1.67 for bending and shear. We elected to bump up the safety factor to 2.0 due to the uncertainty and variability of industry practices. The safety factor used should be further investigated.

Based on initial calculations provide by Sigma, the pins were designed to be limited to a maximum deflection of 0.1in. The limitation is recommended by ICC AC320. With the cantilever effect we calculated that the condition analyzed has a deflection of 0.045" (L/44). As this is acceptable by ICC AC320, and may seem like a small deflection, the pin stiffness would be considered too small for structural applications. At these large ratios, the may pin rotate enough at the free end to not exhibit fixed-fixed behavior and would potentially amplify these deflections.

We would recommend testing to verify the effects of deflection and capacity due to the stand-off caused by the insulation prior to field use.

REFERENCE ONLY

Non Composite Concrete Tilt Up Panel Connectors

10mm Sigma Connector Pin

The Sigma Connector Pin is composed of epoxy modified vinyl ester resin and E-CR glass for the highest durability for fiberglass composites. The ground in deformations have been optimized for bonding with concrete. The modified drive head and tapered end enables the pin to be spun through the foam without pre-drilling with our patent pending socket adapter. Our standard connectors come in lengths suitable for 2”, 3”, and 4” foam.

	Metric	Imperial	Test Method
Tensile Strength	850 MPa	123,000 psi	ASTM D7205 / ACI 440.3R-4 B.2
Tensile Modulus	57 GPa	8,267,000 psi	ASTM D7205 / ACI 440.3R-4 B.2
Flexural Strength	900 MPa	130,500 psi	ASTM D790
Flexural Modulus	53 GPa	7,687,000 psi	ASTM D790
Compressive Strength	400 MPa	58,000 psi	ASTM D695
Shear Strength	>160 MPa	>23,200 psi	ACI 440.3R-4 B.4
Moisture Absorption Max	< 0.1%		ASTM D570 / ISO 62-1980
Minimum Core Diameter	9.2mm	.362 in	
Minimum Cross Section	66.48 mm^2	0.103 in^2	
Minimum Shear Load	10.64 kN	2390 lbs	
Pull Out Capacity - 27 MPa / 4000 psi concrete	9 kN	2025 lbs	

**All Mateen products have been tested/estimated according to AASHTO , ACI and recommended ASTM methods.
Mateen Products are sold subject to Pultron’s standard warranty and nothing herein shall expand or extend such warranty.
The data contained herein is considered representative of present production and believed to be reliable. Pultron Composites Limited reserves the right to make improvements in the product and process which may result in benefits and/or changes to some physical and mechanical properties.**

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R - 2

REFERENCE ONLY

mateenbar™

pultrusion technology pultron COMPOSITES

Mechanical Properties

Property / Diameter	Unit	6	8	10	12	14	16	17.5	18	19	21	22	25	27.5	32	38	Standard
Product		Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	Bar	
Root Diameter	mm	5.2	7.2	9.2	11.0	13.2	15.2	16.7	17.2	18.2	20.0	21.0	24.2	26.7	30.6	36.7	
Outside Diameter	mm	6.0	8.0	10.0	11.8	14.0	16.0	17.5	18.0	19.0	20.8	21.8	25.0	27.5	31.4	37.5	
Nominal Area (Af)	mm^2	21.2	40.7	66.5	95.0	136.8	181.5	219.0	232.4	260.2	314.2	346.4	460.0	559.9	735.4	1057.8	
Ultimate Tensile Load (Average)	kN	19.8	37.9	61.8	88.4	127.3	168.8	203.7	216.1	242.0	292.2	322.2	427.8	520.8	684.0	983.9	ASTM D7205, ACI 440.3R-04
Ultimate Tensile Strength (Average)	MPa	930	930	930	930	930	930	930	930	930	930	930	930	930	930	930	ASTM D7205, ACI 440.3R-04
Ultimate Tensile Load (Guaranteed)	kN	19	37	60	86	123	163	196	207	231	278	305	400	482	620	860	ASTM D7205, ACI 440.3R-04
Ultimate Tensile Strength (Guaranteed)	MPa	911	910	907	904	900	896	893	891	889	884	881	870	860	844	813	ASTM D7205, ACI 440.3R-04
Tensile Modulus of Elasticity (Guaranteed)	kN	59	59	59	59	59	59	59	59	59	59	59	59	59	59	59	ASTM D7205, ACI 440.3R-04
Ultimate Tensile Rupture Strain		0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016	ACI 440.6-08 8.4
Ultimate Elongation (Guaranteed)	%	1.545	1.542	1.537	1.533	1.526	1.519	1.513	1.511	1.506	1.498	1.492	1.474	1.458	1.430	1.378	
Transverse Shear Strength (Average)	MPa	190	190	190	190	189	179	174	173	170	166	165	161	159	157	154	ACI 440.3R-04
Flexural Strength	MPa	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	> 900	ASTM D790
Flexural Modulus	GPa	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	> 53	ASTM D790
Compressive Strength	MPa	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	> 400	ASTM D695
Short Beam Shear Strength (Average)	kN	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	> 80	ASTM D447
Bond Strength at Failure (Average)	kN	24.6	22.4	20.4	18.6	16.7	15.1	14.0	13.7	13.0	11.9	11.3	9.8	8.9	8.0	7.7	ACI 440.3R-04
Bond-dependent Coefficient kb (Average)		0.59	0.63	0.66	0.70	0.74	0.78	0.81	0.82	0.84	0.87	0.89	0.95	1.00	1.07	1.19	ACI 440.1R-06
Barcol Hardness		> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	> 60	ASTM D2583
Glass Transition Temperature (Minimum)	°C	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	ASTM D3418 by DSC
Thermal Expansion Coefficient Transverse	/ °C x10^{-6}	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	ASTM D 696
Thermal Expansion Coefficient Longitudinal	/ °C x10^{-6}	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	< 30	ASTM D 696
Volume Resistivity	Ω.m x 10^9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	DIN 53 481
Dielectric Strength	kV/m	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	ASTM D149
Moisture Uptake (24 hour)	%	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	<0.025	ASTM D570 at 50°C
Moisture Uptake at Saturation (Maximum)	%	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	<0.125	ASTM D570 at 50°C
Specific Gravity		2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	CAN/CSA-S806 (Annex A)
Weight	kg/m	0.043	0.087	0.144	0.208	0.302	0.402	0.486	0.516	0.578	0.699	0.771	1.026	1.249	1.643	2.365	

[1] Guaranteed Ultimate Tensile Strength: CSA S807-10 - 8.2 Classification Based on Tensile Strength.
[2] The tensile properties of 38mm MateenBar cannot be guaranteed due to inability to achieve a valid bar as per the requirements of ASTM D7205 and ACI440.3R-04.

REFERENCE ONLY

ICC EVALUATION SERVICE

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ACCEPTANCE CRITERIA FOR FIBER-REINFORCED COMPOSITE CONNECTORS ANCHORED IN CONCRETE

AC320

Approved June 2006

Effective July 1, 2006

(Editorially revised November 2009)

PREFACE

Evaluation reports issued by ICC Evaluation Service, Inc. (ICC-ES), are based upon performance features of the International family of codes and other widely adopted code families, including the Uniform Codes, the BOCA National Codes, and the SBCCI Standard Codes. Section 104.11 of the *International Building Code*® reads as follows:

> The provisions of this code are not intended to prevent the installation of any materials or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.

Similar provisions are contained in the Uniform Codes, the National Codes, and the Standard Codes.

This acceptance criteria has been issued to provide all interested parties with guidelines for demonstrating compliance with performance features of the applicable code(s) referenced in the acceptance criteria. The criteria was developed and adopted following public hearings conducted by the ICC-ES Evaluation Committee, and is effective on the date shown above. All reports issued or reissued on or after the effective date must comply with this criteria, while reports issued prior to this date may be in compliance with this criteria or with the previous edition. If the criteria is an updated version from the previous edition, a solid vertical line (**|**) in the margin within the criteria indicates a technical change, addition, or deletion from the previous edition. A deletion indicator (→) is provided in the margin where a paragraph has been deleted if the deletion involved a technical change. This criteria may be further revised as the need dictates.

ICC-ES may consider alternate criteria, provided the report applicant submits valid data demonstrating that the alternate criteria are at least equivalent to the criteria set forth in this document, and otherwise demonstrate compliance with the performance features of the codes. Notwithstanding that a product, material, or type or method of construction meets the requirements of the criteria set forth in this document, or that it can be demonstrated that valid alternate criteria are equivalent to the criteria in this document and otherwise demonstrate compliance with the performance features of the codes, ICC-ES retains the right to refuse to issue or renew an evaluation report, if the product, material, or type or method of construction is such that either unusual care with its installation or use must be exercised for satisfactory performance, or if malfunctioning is apt to cause unreasonable property damage or personal injury or sickness relative to the benefits to be achieved by the use of the product, material, or type or method of construction.

Acceptance criteria are developed for use solely by ICC-ES for purposes of issuing ICC-ES evaluation reports.

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3.3.1 General: The information in Sections 3.3.2, 3.3.3, 4.4.4 and 6.6.4 shall be applied in determining allowable service loads. The adjustment for wind or seismic load set forth in Section 6.6.4 is permitted in accordance with Section 1612.3.2 of the UBC if cyclic tests described in Section 4.7 of this criteria are conducted.

3.3.2 Allowable Service Load Determination:

3.3.2.1 For tension and shear, the allowable service load shall be calculated using the average adjusted or unadjusted ultimate load, as applicable, and a factor of safety in accordance with Table 7 of this criteria.

3.3.2.2 For tension and shear, the displacement at the allowable design load shall be determined, and the average displacement for each test series shall be calculated.

3.3.3 Adjustment Factor Considerations:

3.3.3.1 Installation Parameters: When the load test program evaluates the connector with variations in installation parameters such as spacing, edge distance, embedment, and slab thickness, allowable loads may need corresponding adjustment factors to reflect capacity reductions. Test load results shall be analyzed by comparing loads corresponding to the various installation parameters and developing appropriate load adjustment factors, which are applied to the optimum allowable connector load.

When more than one load adjustment factor is applied, the product of the factors is used to determine design loads. Examples include connectors installed at reduced spacings and reduced edge distances.

3.3.3.2 Compressive Strength: Where connector values are desired in concrete of varying compressive strengths, such values may be derived by interpolation from test results for two concrete compressive strengths, providing the range in mix design strength from one group of tests to another does not exceed 2,000 psi (13.8 MPa).

3.3.3.3 Capacity Reductions: In lieu of direct testing, to determine service conditions for tension capacity where edge distance is less than embedment length, Eq-1 shall be used to determine the capacity reduction factor to be multiplied by the mean seismic tension loads determined in Section 4.7 of this criteria if seismic recognition is desired. Otherwise average static tension loads can be used:

$$C_{es} = \frac{d_e}{h_v} \leq 1.0 \qquad \text{(Eq-1)}$$

where:

C_{es} = Capacity reduction factor to be multiplied by cyclic tension load for seismic recognition under the UBC, or static tension load if only static recognition is desired.

d_e = Distance from centerline of connector to concrete edge measured perpendicular to edge.

h_v = Connector embedment length.

To determine critical edge distance for shear capacity, Eq-2 shall be used:

$$d_e = \left(\frac{\varphi \cdot V_c}{\varphi \cdot 12.5 \cdot \sqrt{f'_c}} \right)^{2/3} \qquad \text{(Eq-2)}$$

where:

d_e = Distance from centerline of connector to concrete edge measured perpendicular to edge.

φ = Concrete strength reduction factor = 0.85.

V_c = Average shear strength of connector obtained from cyclic shear testing for seismic recognition under the UBC, or static shear load if only static recognition is desired.

f'_c = Concrete strength for which testing was performed and recognition is desired.

The resultant edge distance will be multiplied by a factor of 4.0 to obtain the critical edge distance of the connector for shear capacity. If this result is greater than h_v then the procedure in this section can be ignored and h_v shall be the critical edge distance. If recognition for a smaller critical edge distance is desired, tension testing must be conducted to determine critical edge distance.

3.3.3.4 Adjustment of Shear Values Due to Bending of Connector: The fiber-reinforced composite connector used in the intended application resists shear loads in bending rather than pure shear. Therefore, a limiting displacement value of 0.1 inch (2.54 mm) due to gravity loads is placed on the connector. When the connector displacement exceeds the limiting value of 0.1 inch (2.54 mm) due to gravity loads, the free end of the connector shall be supported by other means. The displacement shall be calculated in accordance with Eq-3 (neglecting any contribution from the insulation in the intended application):

$$\Delta_g = \frac{Q_g \cdot d^3{}_A}{12E_{Ab} \cdot I_A} \qquad \text{(Eq-3)}$$

where:

Δ_g = Displacement due to gravity load, inch or mm

Q_g = Gravity load on the connector, typically the weight of the fascia layer of the tributary area for the connector, lb or kg = tabγ.

where:

t = Thickness of the fascia layer, feet or mm.

a = Horizontal spacing of the connector, feet or mm.

b = Vertical spacing of the connector, feet or mm.

γ = Density of concrete, lb/ft^3 or kg/mm^3

TABLE 5—TENSION CYCLIC LOAD PROGRAM

LOAD LEVEL	NUMBER OF CYCLES
N_s	10
Ni	30
Nm	100

where:

N_i = A load midway between N_s and N_m.
N_m = One-fourth the average ultimate tension load, T_{ref}, in concrete of the tested strength.
N_s = The maximum tension test load.

TABLE 6—SHEAR CYCLIC LOAD PROGRAM

LOAD LEVEL	NUMBER OF CYCLES
$\pm V_s$	10
$\pm V_i$	30
$\pm V_m$	100

where:

V_i = A load midway between V_s and V_m.
V_m = One-fourth the average ultimate shear load, V_{ref}, in concrete of the tested strength.
V_s = The maximum shear test load.

TABLE 7—FACTORS OF SAFETY

MATERIAL	TENSION		SHEAR	
	UBC	IBC	UBC	IBC
Concrete with special inspection	4	4	4	4

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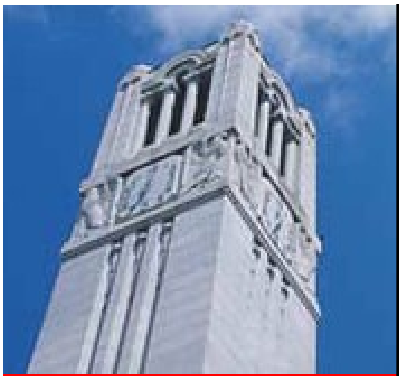

NC STATE UNIVERSITY

Constructed Facilities Laboratory
Department of Civil, Construction,
and Environmental Engineering

Technical Report
No. IS-17-08

TENSION TESTING OF GFRP ANCHORS EMBEDDED IN CONCRETE

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March 26th, 2017

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Executive Summary

The tension capacity of cylindrical GFRP anchors embedded in concrete was investigated and compared to an anchor having a rectangular cross section. Five specimens were prepared with cylindrical anchors, and these specimens all withstood a higher load than did the specimen prepared with a rectangular anchor. The average peak load for the five cylindrical anchor specimens was 2,765 lbs. versus 2,337 lbs. for the rectangular anchor.

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1. Introduction

1.1. Objective

The purpose of the testing program was to investigate the direct tensile capacity of cylindrical GFRP anchors partially embedded in concrete.

1.2. Scope

Twelve GFRP anchors were partially embedded in six square reinforced concrete blocks. Each block measured 10”x10”x5”, and one anchor was centered on each 10” x 10” surface. Blocks were cast using a concrete mixture similar to what might be considered typical at a precast concrete facility. Once the concrete reached a nominal compressive strength of 5,000 psi (4 days after casting), the anchors were loaded in direct tension with a universal testing machine. Load was applied steadily to failure, and the peak applied load and failure mode were noted for each test. All tests were performed at the Constructed Facilities Laboratory at North Carolina State University on March 13th, 2017.

2. Test Program

2.1. Specimen Fabrication

Two types of GFRP anchors were provided to the laboratory by the client. One type had a circular cross section, milled grooves, and a nominal length of 7”. One end of this anchor was flat the other had a pointed tip. The second type of anchor had a square cross section with notches and plastic threads, and was 5 ¾” in length. Photographs of each anchor are shown in Figure 2-1 and Figure 2-2.

Figure 2-1: Cylindrical GFRP Anchor

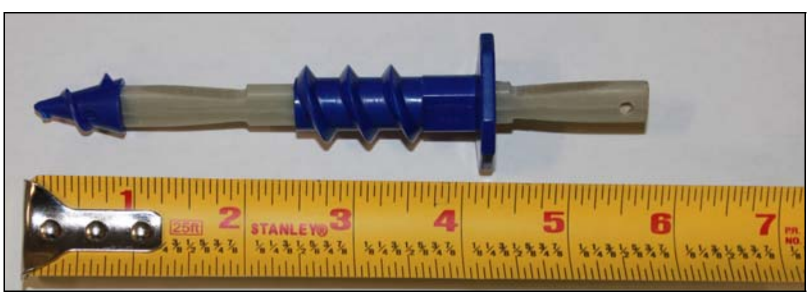

Figure 2-2: Square GFRP Anchor

Six test specimens were prepared. Each specimen consisted of a 10”x10”x5” concrete block with a GFRP anchor embedded 2” into the center of the two opposing square surfaces. The first five test specimens used two cylindrical anchors each. For those five specimens, the flat end of the anchor was embedded into the bottom 2” of concrete, and the conical end of another anchor was embedded into the top 2” of concrete. In this way, each specimen tested both ends of the cylindrical style anchor. The sixth specimen used a cylindrical anchor in the bottom and a rectangular anchor in the top (only one rectangular anchor was available). For each specimen, a layer of 4x4 W2.5 x W2.5 WWR was placed in a plane perpendicular to the primary axis of the anchors at the mid-height of the block. A sketch of the typical specimen is shown in Figure 2-3.

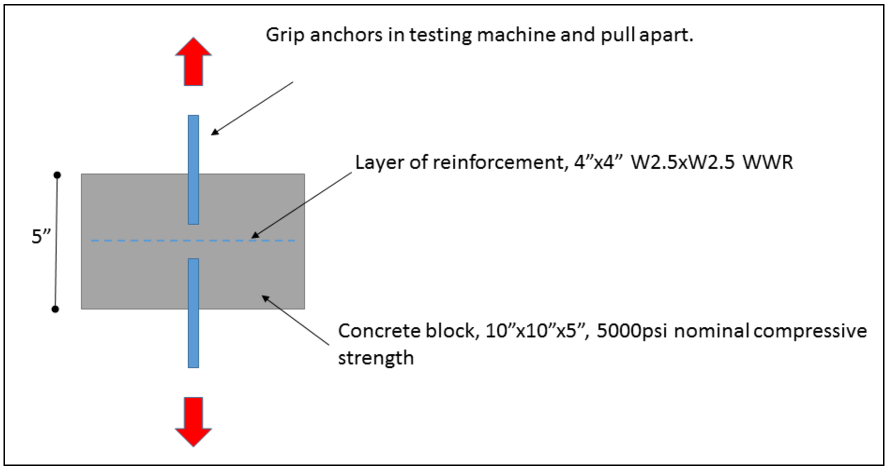

Figure 2-3: Drawing of a Typical Test Specimen

Rigid foam was used to secure each anchor in the right location with respect to the formwork, while maintaining a 2” concrete embedment length. Photographs depicting the various stages of specimen construction are shown in Figure 2-4 through

Figure 2-4: Photograph Showing at Typical Bottom Pin in Typical Formwork

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Figure 2-5: Welded Wire Reinforcement Placed Just above Bottom Pin

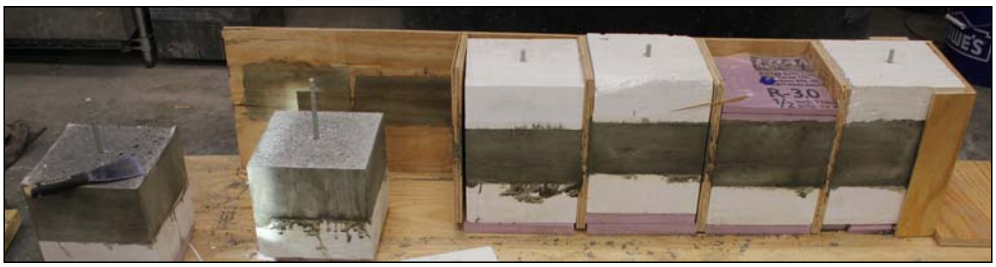

Figure 2-6: All Test Specimens being Demolded

Figure 2-7: All Specimens Prior to Testing

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2.2. Testing Procedure

All tests were conducted in a universal testing machine (UTM) with hydraulic wedge grips and closed-loop control. Prior to placing a specimen into the machine, a thin layer of aluminum was wrapped around body of each GFRP anchor to prevent the serrated jaws from cutting into the anchor. The GFRP anchors were then gripped directly by the machine, and were tested in the same orientation they were cast. A typical specimen in the test setup is shown in Figure 2-8.

Figure 2-8: Typical Specimen in the Test Machine

After being secured in the testing machine, the anchors were loaded in direct tension at a constant displacement rate of 0.05 in/min. until failure occurred. On the same day the anchor tests were performed, six 4”x8” concrete cylinders, cast and cured alongside the anchor specimens, were tested to determine the concrete compressive strength.

3. Test Results

3.1. Concrete Strength

Results of the concrete cylinder compressive tests are shown in Table 3-1.

Table 3-1: Results of Compression Tests

Cylinder #	Peak Load (lbs.)	Peak Stress (psi)	Average Compressive Strength (psi)
1	57,453	4,570	4,660
2	58,895	4,690	
3	58,886	4,690	
4	58,131	4,630	
5	57,934	4,610	
6	60,111	4,780	

3.2. Anchor Testing Results

Results of the anchor tests are shown in Table 3-2.

Table 3-2: Anchor Test Results

Specimen #	Peak Load (lbs.)	Average Peak Load (lbs.)
1	2,523	2,765
2	2,790	
3	2,466	
4	3,066	
5	2,980	
6*	2,337	2,337

* anchor with rectangular cross section

A plot depicting the load versus the UTM crosshead displacement is shown in Figure 3-1. The failure modes for each anchor test are shown in Table 3-3.

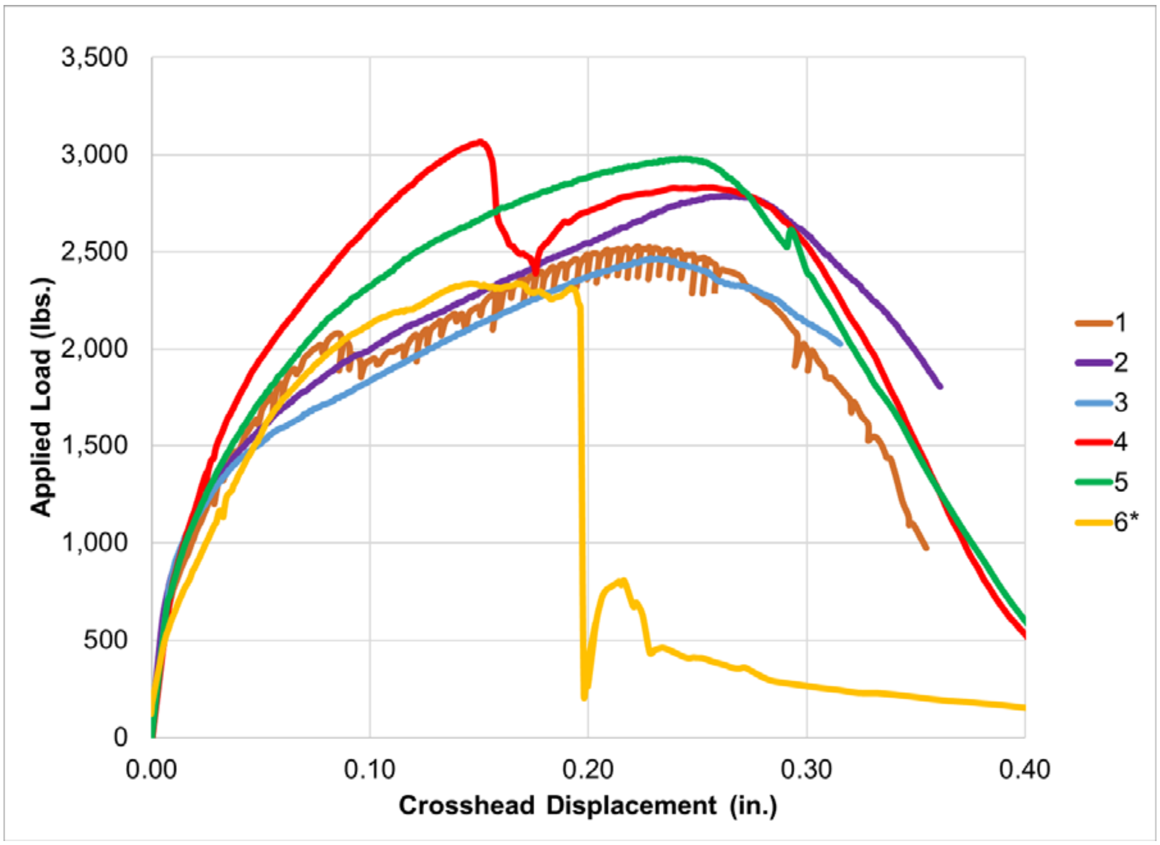

Figure 3-1: Plot of Load vs. Crosshead Displacement Data for all Tests

* anchor with rectangular cross section

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Table 3-3: Failure Modes

Specimen #	Failure Mode	Photograph of Failure
1	Tension failure in concrete, top surface	
2	Tension failure in concrete, bottom surface	
3	Debonding/slip of anchor in bottom surface	
4	Tension failure in concrete, top surface	
5	Pull-out of top anchor; shearing of anchor ridges	
6*	Tensile failure of GFRP anchor near grip; Prior slip of anchor in concrete	

* anchor with rectangular cross section

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European Organisation for Technical Approvals
Europäische Organisation für Technische Zulassungen
Organisation Européenne pour l'Agrément Technique

ETAG 001

Edition 1997

GUIDELINE FOR EUROPEAN TECHNICAL APPROVAL
OF

METAL ANCHORS FOR USE IN CONCRETE

Annex C: DESIGN METHODS FOR ANCHORAGES

Amended October 2001
2nd Amendment November 2006
3rd Amendment August 2010

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a) The fixture shall be made of metal and in the area of the anchorage be fixed directly to the concrete either without an intermediate layer or with a levelling layer of mortar (compression strength ≥ 30 N/mm²) with a thickness < d/2.

b) The fixture shall be in contact with the anchor over its entire thickness.

4.2.2.4 Shear loads with lever arm

If the conditions a) and b) of 4.2.2.3 are not fulfilled the lever arm is calculated according to Equation (4.2) (see Figure 4.8)

$$\ell = a_3 + e_1 \tag{4.2}$$

with
e_1 = distance between shear load and concrete surface
a_3 = 0.5 d
a_3 = 0 if a washer and a nut are directly clamped to the concrete surface (see Figure 4.8b)
d = nominal diameter of the anchor bolt or thread diameter (see Figure 4.8a)

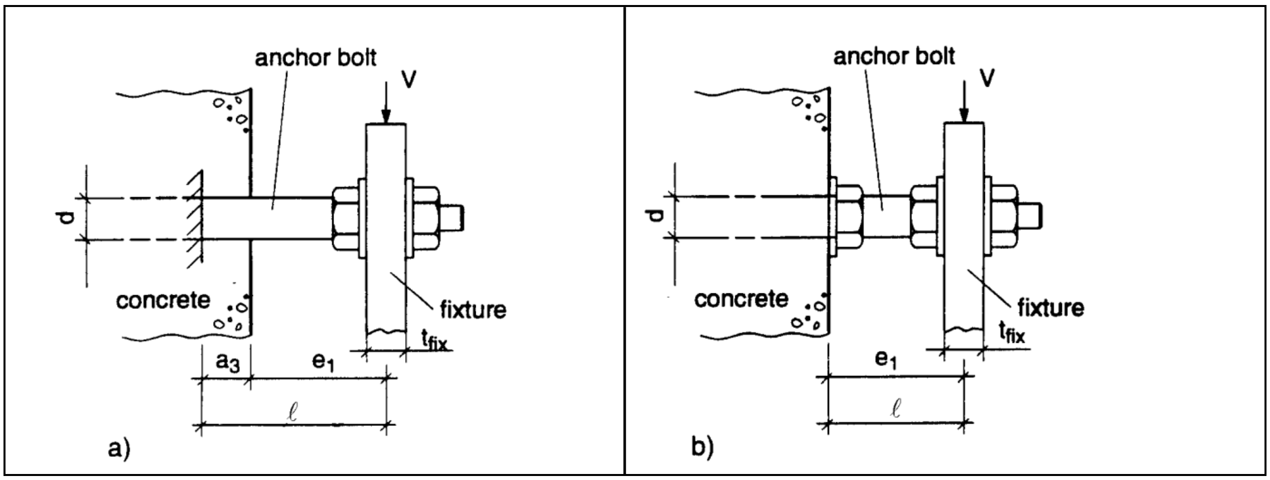

Figure 4.8 Definition of lever arm

The design moment acting on the anchor is calculated according to Equation (4.3)

$$M_{Sd} = V_{Sd} \cdot \frac{\ell}{\alpha_M} \tag{4.3}$$

The value α_M depends on the degree of restraint of the anchor at the side of the fixture of the application in question and shall be judged according to good engineering practice.
No restraint (α_M = 1.0) shall be assumed if the fixture can rotate freely (see Figure 4.9a). This assumption is always conservative.
Full restraint (α_M = 2.0) may be assumed only if the fixture cannot rotate (see Figure 4.9b) and the hole clearance in the fixture is smaller than the values given in Table 4.1 or the anchor is clamped to the fixture by nut and washer (see Figure 4.8). If restraint of the anchor is assumed the fixture shall be able to take up the restraint moment.

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5.2.3 Resistance to shear loads

5.2.3.1 Required proofs

	single anchor	anchor group	
steel failure, shear load without lever arm	$V_{Sd} \leq V_{Rk,s} / \gamma_{Ms}$	$V^{h}_{Sd} \leq V_{Rk,s} / \gamma_{Ms}$	
steel failure, shear load with lever arm	$V_{Sd} \leq V_{Rk,s} / \gamma_{Ms}$	$V^{h}_{Sd} \leq V_{Rk,s} / \gamma_{Ms}$	
concrete pry-out failure	$V_{Sd} \leq V_{Rk,cp} / \gamma_{Mc}$		$V^{g}_{Sd} \leq V_{Rk,cp} / \gamma_{Mc}$
concrete edge failure	$V_{Sd} \leq V_{Rk,c} / \gamma_{Mc}$		$V^{g}_{Sd} \leq V_{Rk,c} / \gamma_{Mc}$

5.2.3.2 Steel failure

a) Shear load without lever arm

The characteristic resistance of an anchor in case of steel failure, $V_{Rk,s}$ is given in the relevant ETA.

The value $V_{Rk,s}$ for anchors according to current experience is obtained from Equation (5.4)

$$V_{Rk,s} = 0.5 \cdot A_s \cdot f_{uk} \quad [N] \qquad (5.4)$$

Equation (5.4) is not valid for anchors with a significantly reduced section along the length of the bolt (e.g. in case of bolt type expansion anchors).

In case of anchor groups, the characteristic shear resistance given in the relevant ETA shall be multiplied by a factor 0.8, if the anchor is made of steel with a rather low ductility (rupture elongation $A_5 \leq 8\%$)

b) Shear load with lever arm

The characteristic resistance of an anchor, $V_{Rk,s}$, is given by Equation (5.5).

$$V_{Rk,s} = \frac{\alpha_M \cdot M_{Rk,s}}{\ell} \quad [N] \qquad (5.5)$$

where α_M = see 4.2.2.4

ℓ = lever arm according to Equation (4.2)

$M_{Rk,s}$ = $M^0_{Rk,s}$ $(1 - N_{Sd}/N_{Rd,s})$ [Nm] (5.5a)

$N_{Rd,s}$ = $N_{Rk,s} / \gamma_{Ms}$

$N_{Rk,s}$, γ_{Ms} to be taken from the relevant ETA

$M^0_{Rk,s}$ = characteristic bending resistance of an individual anchor

The characteristic bending resistance $M^0_{Rk,s}$ is given in the relevant ETA.

The value of $M^0_{Rk,s}$ for anchors according to current experience is obtained from Equation (5.5b).

$$M^0_{Rk,s} = 1.2 \cdot W_{el} \cdot f_{uk} \quad [Nm] \qquad (5.5b)$$

Equation (5.5b) may be used only if the anchor has not a significantly reduced section along the length of the bolt.