Sturdi-Wall Design and Use Guide

for drill set & wet set models

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Project Number S021-12

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April 2023

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Design Overview

This guide is intended to be used by post-frame building engineers and designers as a companion document to the ESR-4239 report by *International Code Council Evaluation Services (ICC ES).* The structural analysis described in this guide is based on the load and resistance factor (LRFD) and the allowable strength design (ASD) methodologies in accordance with 2018 International Building Code (IBC). This Design and Use Guide covers properties and design procedures for the Sturdi-Wall and Sturdi-Wall Plus steel brackets.

Important Note: The building must be designed to resist lateral loads through diaphragm action or other bracing means. SW and SWP brackets are not an alternative to this requirement. The SW and SWP brackets are not recommended for columns that do not have a lateral restraint at the top.

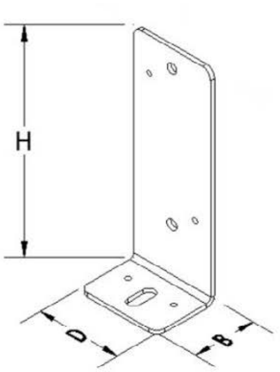
Part 1: Standard Sturdi-Wall (Drill Set) Model

1. Sturdi-Wall Design Overview

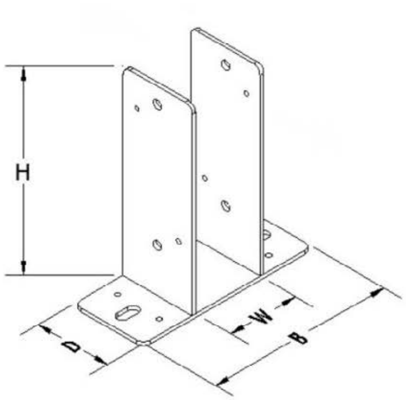
The standard Sturdi-Wall anchor brackets are designed to connect wood columns to a concrete foundation in a typical post-frame building application using a drill set installation method. This section contains drawings and descriptions for each of the Sturdi-Wall models, charts showing shear and uplift strength, descriptions of several methods for attachment to concrete, and discussion of design assumptions. See Part 2 of this manual for information on a wet set installation.

2. Sturdi-Wall Descriptions

Dimensions for the, SW6C, SW60, SW80, SW46, SW55, SW63, SW64, SW65, SW66, SW83, SW84, SW85 and SW88 are given in Figure 2.1. The brackets are constructed with ¼" ASTM A1018, SS designation Grade 40 steel and ¼" fillet welds of E70XX electrodes. Each assembly has a proprietary powder coat finish. The SW46 is to be used with a 4x6 wood post, SW55 with a 5x5 wood post, SW66 with a 6x6 wood post, SW88 with 8x8 wood post, SW63 with a 3-ply 2x6 laminated column, SW64 with a 4-ply 2x6 laminated column, SW65 with a 5-ply 2x6 laminated column, SW83 with a 3-ply 2x8 laminated column, SW84 with a 4-ply 2x8 laminated column, and SW85 with a 5-ply 2x8 laminated column. Laminated column may be mechanically laminated using nails, screws or bolts, or glue laminated (glulam). The pocket width for the mechanically laminated columns is different than one for glulam columns. ***Sturdi-Wall*** models for use with glulam columns are identified with a "GL" at the end of the name. The inside dimension of the brackets allows for an ⅛" total tolerance between the steel bracket and wood column. When 1/8" maximum tolerance is not achieved, appropriately sized wood shims must be added on one or both sides of the pocket to provide a snug fit. The shim should be APA B-C Exterior plywood (or equivalent), no more than ¼" thick, and have the same dimensions as the vertical leg of the bracket. The shim should be fastened to the column with 2 beads of Builders Choice 490 construction adhesive by Liquid Nails (or equal) and (6) 0.113"x2.375" nails. Brackets SW60 and SW80 shall be used as pairs and fastened using the same connectors as the fully assembled brackets. Corner columns require two SW60C brackets located on adjacent faces of the column (not opposite faces).



SW60/SW80



SSW6X/SW8X SERIES

SW6X/SW8X

	Type	W (Standard)	W (Glulam)	B	D	H	Anchor Center
SW6C	3, 4 or 5 ply 2x6 or 2x8, 4x6, 6x6, 8x8	n/a	n/a	3 1/2″	4″	13″	n/a
SW60	3, 4 or 5 ply 2x6 or 2x8, 4x6, 6x6, 8x8	n/a	n/a	3 1/2″	5″	13″	n/a
SW80	3, 4 or 5 ply 2x6 or 2x8, 4x6, 6x6, 8x8	n/a	n/a	3 1/2″	7″	18″	n/a
SW46	4″ X 6″ Post	3 5/8″	n/a	12 1/8″	5″	13″	9 3/8″
SW55	5″ X 5″ Post	4 5/8″	n/a	12 1/8″	5″	13″	9 3/8″
SW66	6″ X 6″ Post	5 5/8″	n/a	13 5/8″	5″	13″	10 7/8″
SW63	3 PLY 6″ Lam Col	4 5/8″	4 3/16″	12 1/8″	5″	13″	9 3/8″
SW64	4 PLY 6″ Lam Col	6 1/8″	5 9/16″	13 5/8″	5″	13″	10 7/8″
SW65	5 PLY 6″ Lam Col	7 5/8″	7 5/16″	15 1/8″	5″	13″	12 3/8″
SW83	3 PLY 8″ Lam Col	4 5/8″	4 3/16″	12 1/8″	7″	18″	9 3/8″
SW84	4 PLY 8″ Lam Col	6 1/8″	5 9/16″	13 5/8″	7″	18″	10 7/8″
SW85	5 PLY 8″ Lam Col	7 5/8″	7 5/16″	15 1/8″	7″	18″	12 3/8″
SW88	8″ X 8″ Post	8 1/8″	n/a	15 1/8″	7″	18″	12 3/8″

Figure 2.1: Sturdi-Wall Descriptions

3. Steel Bracket Design

The forces applied from the building columns to Sturdi-Wall brackets are a vertical uplift force, a downward gravity force, and a horizontal shear force perpendicular to the plane of the wall. The wood columns need direct bearing on the bottom to transfer axial loads directly into the concrete foundation. The Sturdi-Wall brackets are assumed to have no moment capacity. The building must be designed to resist lateral loads through diaphragm action or other bracing means. All mechanical fasteners are to be installed as per the manufacturer’s recommendations and this design guide. The brackets consist of ¼” ASTM A1018, SS designation grade 40 steel with ⅝” diameter holes for the bolts in the vertical leg, and ¾” x 1 ½” slotted holes for concrete anchors in the base. The brackets also have holes for screws near the bolts in the vertical legs.

4. Concrete Anchors

SW brackets may be attached to concrete with anchor “L” bolts, epoxy anchors and screw anchors.

4.1 Anchor “L” Bolts

The anchor “L” bolts are to be typical ASTM F1554 36 grade or ASTM A36 right angle bend cast in place anchor bolts. These are set in wet concrete and must be placed within the tolerance of the slotted hole in the bottom of the bracket.

4.2 Epoxy Anchors

Epoxy or adhesive anchors must be installed in a properly sized and prepared hole per the manufacturer’s specifications and within a set temperature range in order to be effective.

4.3 Screw Anchors

Screw anchors have a hex head and a threaded shaft which can be installed with an impact wrench or conventional hand socket. There are no expansion forces transferred to the concrete base material so they can be installed closer to the edge than traditional expansion anchors.

4.4 Expansion Anchors – Not recommended for SW application

Important Note: The prying forces in the base plate of the SW bracket increase the uplift forces on the anchors by approximately 100% (a factor of 2). For example, if the net uplift force at the base of the column is 1000 lb, the uplift force on the two anchors is then approximately 2000 lb, or 1000 lb at each anchor.

5. Wood Connection

The steel-to-wood connection is made with (2) ½” diameter SAE J429 Grade 5 bolts in double shear and ¼” x 3” strong drive screws (SDS) by Simpson Strong Tie or equal PC-approved (Perma-Column approved) screws in single shear installed from each side. The screws have a one-inch long 0.242-inch to 0.249-inch diameter unthreaded shank before the root diameter is reduced at the threads. The highest concentration of stresses is located near the face of the wood column along the unthreaded segment of the screw. The stresses dissipate significantly at the end of the unthreaded shank segment (beginning of threaded segment). When compared to standard wood screws, the SDS and other PC-approved structural screws have a significantly greater shear strength values in a steel-to-wood application. This difference in strength is attributed primarily to two factors: SDS and other PC-approved screws have a high specified bending yield strength and a long large-diameter unthreaded shank. Typically, one screw is installed from each side of the bracket at each bolt except the SW8 series has 2 screws on each side at each bolt. Screws help prevent stress concentration around the bolt which may otherwise cause splitting of the wood members. The wood-to-steel connection was analyzed per the National Design Specification for Wood Construction 2018 edition by the American Wood Council using Southern Yellow Pine wood columns (Specific Gravity = 0.55). Wet service reductions have not been applied since the wood portion is not in contact with the soil or concrete and the column is assumed to be used in an enclosed building. If the brackets are to be used in an environment where the moisture content of the wood in service will exceed 19% for an extended period of time, pressure treated wood and galvanized or stainless steel bolts should be used, and a wet service factor should be applied to the shear and uplift values in Tables 6.1 and 6.2. **In addition, a barrier membrane should be applied between the pressure treated wood post and the Sturdi-Wall bracket to provide corrosion protection.** Consult your local supplier for a suitable barrier. The design of the wood columns, the concrete foundation and the lateral force resisting system of the building is the responsibility of the building designer.

6. Sturdi-Wall Bracket Design Chart

Tables 6.1 and 6.2 shows the shear and uplift strength of the steel brackets, steel-to-wood connection, and steel-to-concrete connection (anchors). The shear and uplift strength values for the steel-to-wood connection have been adjusted by the Load Duration Factor, C_D, of 1.6 (ASD) and the Time Effect Factor, λ, of 1.0 (LRFD) consistent with short duration loads. No further increases are permitted. The steel-to-concrete strength values are for concrete with a minimum compressive strength, f'_c, of 3 ksi. Anchor calculations are for 1/2-inch Ø cast-in-place anchors or screw anchors with the Nominal Tensile Strength, F_{nt}, of 45 ksi, the Nominal Shear Strength, F_{nv}, of 24 ksi, and a minimum of 5-3/4" embedment into the concrete (h_{eff} = 4.3 inches). The steel-to-concrete values include the tensile and shear strength of the anchors (including prying action, see the boxed Important Note below), and the concrete breakout strength for wall thicknesses of 8 inches, 10 inches and 12 inches with anchors centered in the wall. **The steel-to-concrete values in Tables 6.1 and 6.2 are intended only for demonstrational purposes. The anchor and concrete calculations are the responsibility of the building designer.**

Important Note: The prying forces in the base plate of the SW bracket increase the uplift forces on the anchors by approximately 100% (a factor of 2). For example, if the net uplift force at the base of the column is 1000 lb, the uplift force on the two anchors is then approximately 2000 lb, or 1000 lb at each anchor.

Table 6.1, ASD: Allowable Shear and Uplift Strength for Sturdi-Wall Anchor Brackets

	Strength of Steel Bracket		**Steel-to-Wood Connection**		**Steel-to-Concrete Connection**					
					1/2" Anchor in 8" Concrete Wall		**1/2" Anchor in 10" Concrete Wall**		**1/2" Anchor in 12" Concrete Wall**	
Model	***Shear (lb)***	***Uplift (lb)***	***Shear (lb)***	***Uplift (lb)***	***Shear (lb)***	***Uplift (lb)***	***Shear (lb)***	***Uplift (lb)***	***Shear (lb)***	***Uplift (lb)***
SW46	36100	2970	2100	6050	2600	1770	3310	2210	3310	2650
SW55	36100	3330	2100	6050	2600	1870	3310	2330	3310	2800
SW63	36100	3680	2100	6050	2600	1960	3310	2450	3310	2940
SW64	36100	3620	2100	6050	2780	2080	3520	2600	3520	3120
SW65	36100	3440	2100	6050	2920	2130	3720	2700	3720	3240
SW66	36100	3640	2100	6050	2780	2080	3520	2600	3520	3120
SW83	52600	4980	3030	8490	2600	1930	3310	2410	3310	2890
SW84	52600	4880	3030	8490	2780	2040	3520	2550	3520	3060
SW85	52600	4820	3030	8490	2920	2130	3720	2700	3720	3240
SW88	52600	4800	3030	8490	2920	2130	3790	2750	3790	3300
SW60 (pair)	1310	3400	2100	6050	2600	1890	2600	2360	2600	2840
SW80 (pair)	2210	4500	3030	8490	2600	1840	3310	2300	3310	2760
SW6C (pair)	720	1300	1120	7310	1460	1070	1860	1340	1460	1610

Table 6.2, LRFD: Design Shear and Uplift Strength for Sturdi-Wall Anchor Brackets

	Strength of Steel Bracket		Steel-to-Wood Connection		Steel-to-Concrete Connection					
					1/2" Anchor in 8” Concrete Wall		1/2" Anchor in 10” Concrete Wall		1/2" Anchor in 12” Concrete Wall	
Model	*Shear (lb)*	*Uplift (lb)*	*Shear (lb)*	*Uplift (lb)*	*Shear (lb)*	*Uplift (lb)*	*Shear (lb)*	*Uplift (lb)*	*Shear (lb)*	*Uplift (lb)*
SW46	54200	4460	2830	8160	4160	2830	5300	3540	3540	4240
SW55	54200	5010	2830	8160	4160	2990	5300	3730	3730	4480
SW63	54200	5530	2830	8160	4160	3140	5300	3920	3920	4740
SW64	54200	5450	2830	8160	4450	3330	5630	4160	4160	4990
SW65	54200	5170	2830	8160	4670	3400	5960	4310	4310	5180
SW66	54200	5470	2830	8160	4450	3330	5630	4170	4170	5000
SW83	79000	7480	4080	11450	4160	3090	5300	3860	3860	4630
SW84	79000	7330	4080	11450	4450	3270	5630	4080	4080	4900
SW85	79000	7240	4080	11450	4670	3400	5960	4310	4310	5180
SW88	79000	7210	4080	11450	4670	3400	6060	4400	4400	5280
SW60 (pair)	1970	5110	2830	8160	4160	3020	5300	3780	3780	4540
SW80 (pair)	3320	6760	4080	11450	4160	2940	5300	3680	3680	4410
SW6C (pair)	1080	1950	1510	9850	2340	1720	2980	2140	2140	2570

Table 6.1 and 6.2 Footnotes:

1. This chart is for Sturdi-Wall brackets for use in post-frame building applications to connect wood columns to a concrete wall or foundation.
2. Loads applied to the brackets from the columns are a vertical uplift force and a horizontal shear force
3. Steel-to-Wood connections were calculated as per the NDS 2018 using columns with Specific Gravity of 0.55, dry service conditions
4. The allowable loads in concrete have been divided by 1.6 to convert from LRFD to ASD.
5. Concrete design numbers are based on a minimum concrete compressive strength of 3000 psi
6. Bolts are 1/2" diameter SAE J429 Grade 5 with hex nuts
7. Screws are 1/4" diameter x 3" Strong Drive Screws (SDS) by Simpson Strong Tie, or alternative screws approved by Perma-Column
8. Wood column above the bracket, anchors and concrete foundation below the bracket to be designed by others
9. Install all fasteners as per the manufacturer’s specifications and these notes
10. Final bracket design should include a complete building analysis performed by a design professional

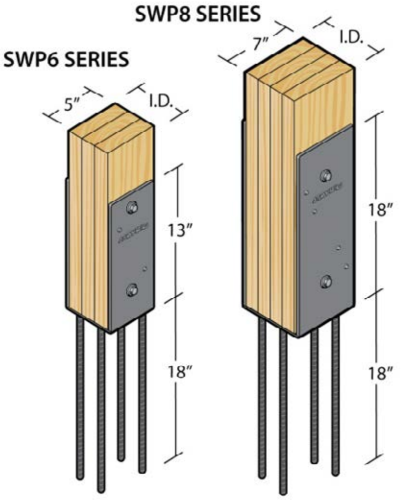
Part 2: Sturdi-Wall Plus (Wet Set) Models

7. Sturdi-Wall Plus Design Overview

The Sturdi-Wall Plus anchor brackets are designed to connect wood columns to a concrete foundation in a typical post frame building application using a wet set installation method. This guide contains drawings and descriptions for each of the Sturdi-Wall Plus models, a chart showing shear, uplift, and bending (moment) strength for each Sturdi-Wall Plus bracket, description of method for attachment to concrete, and discussion of design assumptions. Drill set installation is covered in Part 1 of this guide.

8. Sturdi-Wall Plus Descriptions

Dimensions for the SWP46, SWP63, SWP64, SWP66, SWP83, SWP84, SWP85 and SWP88 are given in Figure 8.1. The brackets are constructed with ¼” ASTM A1018, SS designation Grade 40 steel and ¼” fillet welds of E70XX electrodes. Each assembled bracket has a proprietary powder coat finish. The SWP46 is to be used with a 4x6 wood post, SWP66 with a 6x6 wood post, SWP88 with an 8x8 post, SWP63 with a 3-ply 2x6 laminated column, SWP64 with a 4-ply 2x6 laminated column, SWP83 with a 3-ply 2x8 laminated column, SWP84 with a 4-ply 2x8 laminated column, and SWP85 with a 5-ply 2x8 laminated column. Laminated column may be mechanically laminated using nails, screws or bolts, or glue laminated (glulam). The pocket width for the mechanically laminated columns is different than one for glulam columns. ***Sturdi-Wall* Plus** models for use with glulam columns are identified with a “GL” at the end of the name. The inside dimension of the brackets allows for an ⅛” total tolerance between the steel bracket and wood column. When 1/8” maximum tolerance is not achieved, appropriately sized wood shims must be added on one or both sides of the pocket to provide a snug fit. The shim should be APA B-C Exterior plywood (or equivalent), no more than ¼” thick, and have the same dimensions as the vertical leg of the bracket. The shim should be fastened to the column with 2 beads of Builders Choice 490 construction adhesive by Liquid Nails (or equal) and (6) 0.113”x2.375” nails.



	Type	I.D. Standard	I.D. Glulam
SWP46	4″ X 6″ Post	3 5/8″	n/a
SWP63	3 PLY 6″ Lam Col	4 5/8″	4 3/16″
SWP64	4 PLY 6″ Lam Col	6 1/8″	5 9/16″
SWP66	6″ X 6″ Post	5 5/8″	n/a
SWP83	3 PLY 8″ Lam Col	4 5/8″	4 3/16″
SWP84	4 PLY 8″ Lam Col	6 1/8″	5 9/16″
SWP85	5 PLY 8″ Lam Col	7 5/8″	6 15/16″
SWP88	8″x 8″ Post	8 1/8″	n/a

Figure 8.1: Sturdi-Wall Plus Descriptions

9. Steel Bracket Design

The forces applied from the building columns to Sturdi-Wall Plus brackets are a vertical uplift force, a downward gravity force, a horizontal shear force, and a moment about the strong axis of the column. The wood columns must have a direct bearing on the bottom to transfer axial loads directly into the concrete wall or foundation. Unlike the Standard Sturdi-Wall brackets, the Sturdi-Wall Plus brackets are a moment resisting brackets with defined rotational stiffness (semi-rigid) and moment strength. All mechanical fasteners are to be installed as per the manufacturer’s recommendations and this design guide. The brackets consist of ¼” A1018, SS designation Grade 40 steel with 18-inch long A706 weldable reinforcing bars welded to the base of the bracket, #4 (½”) for the SWP 4 and 6 Series brackets and #5 (⅝”) for the SWP 8 Series brackets and ⅝” diameter holes for the ½” diameter bolts in the vertical legs. The brackets also have holes for screws near the bolts in the vertical legs. The building must be designed to resist lateral loads through diaphragm action or other bracing means.

10. Rebar Connection

The base of the u-shaped steel bracket is reinforced with an additional flat ¼” thick steel stiffener plate located “inside” the pocket. The bottom of the u-shaped steel bracket has four holes to accept the rebar. Four weldable ASTM A706 grade 60, 18-inch long deformed rebars are welded to the steel stiffener plate inside the bracket and to the u-shaped bracket such that the weld material fills the gap between the rebar and the edges of the holes. The rebar is cast in place by being set in wet concrete shortly after a wall or foundation pour. To achieve tabulated values, a minimum of 2-inch concrete cover is required. Concrete cover is defined as the distance from the nearest edge of the concrete to the side of the rebar. The cast-in-place rebar allows for bending moments to transfer from the steel bracket into the concrete wall or foundation.

11. Wood Connection

The steel-to-wood connection is made with (2) ½” diameter SAE J429 Grade 5 bolts in double shear and ¼” x 3” strong drive screws (SDS) by Simpson Strong Tie or equal PC-approved (Perma-Column approved) screws in single shear installed from each side. The screws have a one-inch long 0.242-inch to 0.249-inch diameter unthreaded shank before the root diameter is reduced at the threads. The highest concentration of stresses is located near the face of the wood column along the unthreaded segment of the screw. The stresses dissipate significantly at the end of the unthreaded shank segment (beginning of threaded segment). When compared to standard wood screws, the SDS and other PC-approved structural screws have a significantly greater shear strength values in a steel-to-wood application. This difference in strength is attributed primarily to two factors: SDS and other PC-approved screws have a high specified bending yield strength and a long large-diameter unthreaded shank. Typically, one screw is installed from each side of the bracket at each bolt except the SW8 series has 2 screws on each side at each bolt. Screws help prevent stress concentration around the bolt which may otherwise cause splitting of the wood members. The wood-to-steel connection was analyzed per the National Design Specification for Wood Construction 2018 edition by the American Wood Council using Southern Yellow Pine wood columns (Specific Gravity = 0.55). Wet service reductions have not been applied since the wood portion is not in contact with the soil or concrete and the column is assumed to be used in an enclosed building. If the brackets are to be used in an environment where the moisture content of the wood in service will exceed 19% for an extended period of time, pressure treated wood and galvanized or stainless steel bolts should be used, and a wet service factor should be applied to the shear, moment, and uplift values in Table 15.1. **In addition, a barrier membrane should be applied between the pressure treated wood post and the Sturdi-Wall bracket to provide corrosion protection.** Consult your local supplier for a suitable barrier. The design of the wood columns, the concrete foundation and the lateral force resisting system of the building is the responsibility of the building designer.

12. Recommended Installation Details

ACI 318-14, Section 20.6 contains concrete cover requirements for protection of reinforcement against moisture from weather and earth. For cast-in-place concrete, the minimum cover is 3” when cast against and permanently exposed to earth, and 1-1/2” when exposed to earth and weather. Since foundation walls and piers into which the Sturdi-Wall Plus brackets are placed are normally formed, not cast against earth, they fall into the second category. However, 1-1/5-inch concrete cover may not be thick enough to ensure full development of rebar in tension; a minimum concrete cover of 2 inches is recommended for SWP brackets. The values in Tables 15.1 and 15.2 are based on a concrete cover of 2 inches.

Figures 12.1 and 12.2 illustrate the recommended installation of an SWP 4/6 Series and an SWP 8 Series bracket, respectively. The concrete foundation shown may be a wall or a pier. An 8-inch concrete thickness is shown for the SWP 4/6 Series and a 10-inch thickness is shown for the SWP 8 Series brackets. The clear cover to the reinforcing bars of the SWP brackets for both cases is greater than 2 inches - adequate for concrete walls and piers poured into forms and exposed to earth or weather. Figure 12.3 applies to a bracket installed close to the edge of an opening in a foundation wall. This detail may also apply to a bracket installed on a concrete pier.

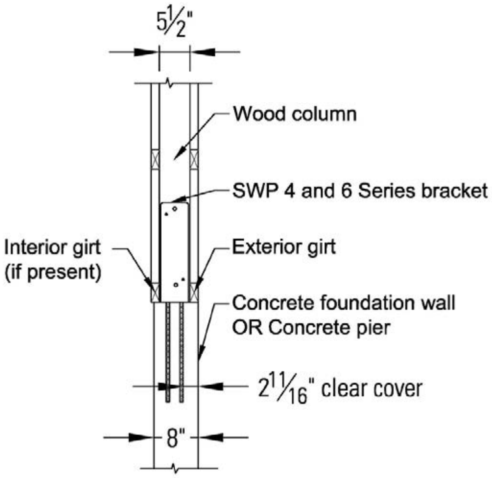


Figure 12.1: Recommended Installation For SWP 4 and 6 Series Brackets

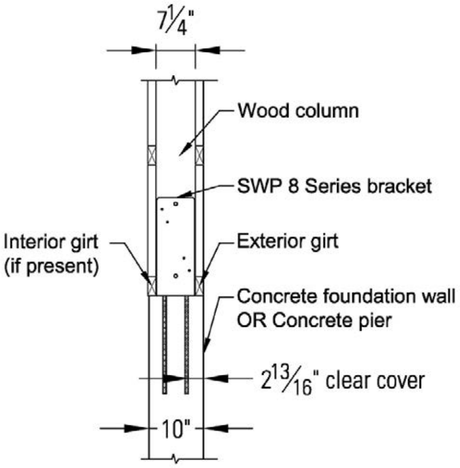


Figure 12.2: Recommended Installation For SWP 8 Series Brackets

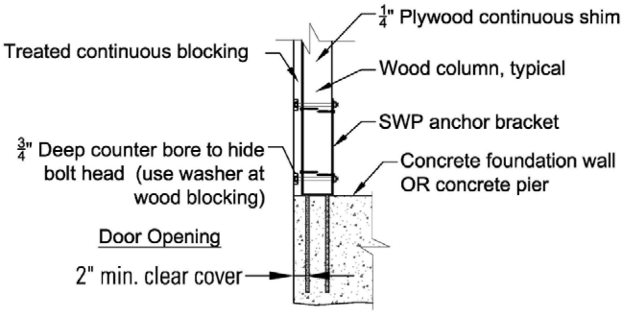


Figure 12.3: SWP Door Edge Detail

When Sturdi-Wall Plus brackets are placed in wet concrete, care shall be taken to ensure that the concrete is not so wet that the brackets sink below the base plate and not so dry that the concrete will not flow around and adhere to the rebar. Ideally, the consistency of the concrete should be such that the rebar can be easily inserted in the concrete and the base of the steel bracket can float on the surface of the concrete. The entire length of rebar shall be embedded in the concrete such that the bottom of the steel base plate bears on the surface of the concrete.

13. Modeling

In the structural computer program, the joint between the concrete foundation and the wood column, representing the SWP bracket, should be modeled as a "semi-rigid joint" using rotational stiffness values in Table 15.1. If the designer's computer program does not have the capability to model semi-rigid joints directly, the designer may create a *joint member* in between the concrete base and the wood column with carefully selected structural and geometrical properties to mimic the behavior of the semi-rigid joint using equation 13-1. Table 13.1 shows the recommended properties for the vertical joint member that is 1-inch long and is made of steel (E=29,000,000 psi). For example, a semi-rigid joint between a 3-ply 2x6 wood column and a concrete foundation using SWP63 bracket can be modeled as a 1-inch long (tall), 0.942-inch wide and 0.942-inch deep vertical member, made of steel material (for ex. ASTM 1018 or A36), rigidly connected to the concrete base below and the wood column above (Figure 13.1). The joint in this example will produce the same results as the joint that is directly assigned a rotational stiffness value of 1900 in-k/rad in Table 15.1.

$$EI = (M/\theta)_e L \qquad \text{(Eq. 13-1)}$$

Where,

E = elastic modulus of the *joint member*
I = moment of inertia of the *joint member's* profile
L = length of the *joint member*
$(M/\theta)_e$ = effective rotational rigidity of the *joint member*
$= [1/(M/\theta)_b + 1/(M/\theta)_w]^{-1}$
$(M/\theta)_b$ = rotational stiffness of the steel bracket (Table 15.1)
$(M/\theta)_w$ = rotational rigidity of the wood segment that is being replaced by *joint member*

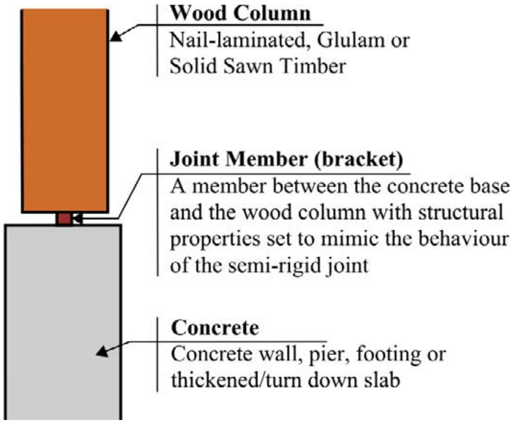


Figure 13.1 ***Joint Member*** **between concrete foundation and wood column**

Table 13.1: Recommended Joint Member Properties

Series	Width (in)	Depth (in)	Length (height) (in)	E (psi)
SWP46	0.936	0.936	1.0	29,000,000
SWP63	0.937	0.937	1.0	29,000,000
SWP64	1.009	1.009	1.0	29,000,000
SWP66	0.944	0.944	1.0	29,000,000
SWP83	1.175	1.175	1.0	29,000,000
SWP84	1.164	1.164	1.0	29,000,000
SWP85	1.162	1.162	1.0	29,000,000
SWP88	1.159	1.159	1.0	29,000,000

IMPORTANT NOTE: SWP brackets must be modeled using the specified rotational stiffness. Assuming a joint with zero stiffness or a fully rigid joint is not an acceptable alternative.

14. Foundation Design

A concrete wall or foundation must be designed in accordance with the established engineering standards to resist the shear, uplift, bending (moment) and downward forces that are transferred from the column. The foundation must have sufficient rotational rigidity to ensure that the inflection point, a point of zero moment, is located above the SWP bracket – not below (Figure 14.1). The latter requirement ensures that shear and bending forces may be applied to the bracket simultaneously without any reduction to the maximum shear and maximum moment strength reported in Table 15.1. The maximum shear and moment strength values in Table 15.1 are not applicable to load cases where this requirement is not satisfied. This subject is explained further in Figure 14.2 and the discussion of load cases that follows.

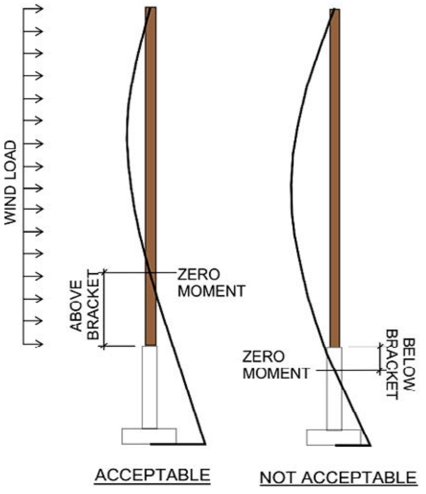


Figure 14.1

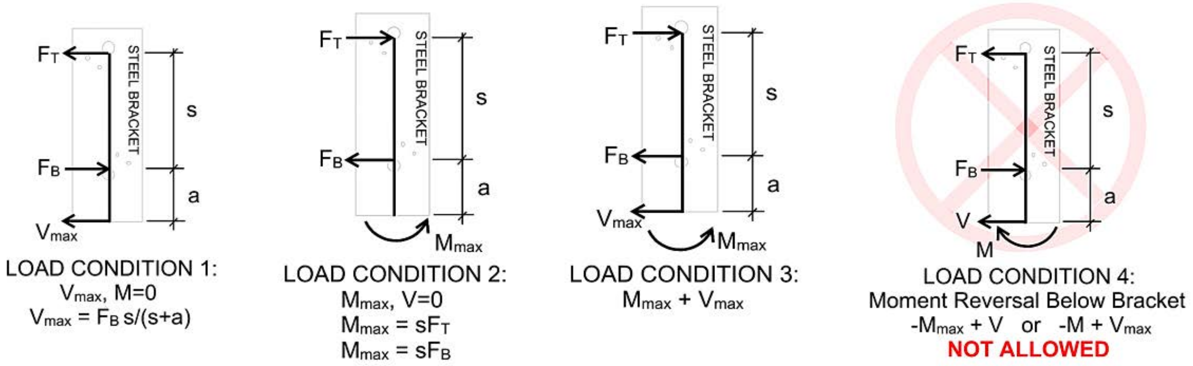


Figure 14.2

In Figure 14.2, Load Case 1 defines the maximum shear strength, V_{max}, of the column-to-bracket connection in absence of moment forces. Load Case 2 defines the maximum moment strength, M_{max}, of the column-to-bracket connection in absence of shear forces. Load Case 3 is a combination of Load Case 1 and Load Case 2 where a maximum moment and a maximum shear force are applied to the bracket simultaneously. In all load cases, the maximum shear strength, V_{max}, and the maximum moment strength, M_{max}, are defined such that the magnitude of the resulting forces F_T (force at the top fastener group) and F_B (force at the bottom fastener group) does not exceed the lateral strength of each respective fastener group.

The resulting forces F_T and F_B in Load Case 1 are acting in opposite directions from the resulting forces F_T and F_B in Load Case 2. This means that adding a shear load to the connection that is loaded with the maximum moment force will result in reduction in forces F_T and F_B. Similarly, adding a moment force to the connection that is loaded with the maximum shear force will result in reduction in forces F_T and F_B. Therefore, when the inflection point (point of zero moment) is located above the SWP bracket, V_{max} and M_{max} loading may be applied to the bracket simultaneously without any reduction in strength. Load Case 4 represents the condition in which the moment reversal occurs below the bracket. In this load condition, M_{max}, as determined by Load Condition 2, cannot be used in combination with a shear force of any magnitude and V_{max}, as determined by Load Condition 1, cannot be used in combination with a moment force of any magnitude. With load condition, as shear force increases moment strength decreases, and as moment force increases shear strength decreases. Therefore, when the inflection point (point of zero moment) is located below the SWP bracket, V_{max} and M_{max} loading may NOT be applied to the bracket simultaneously without any reduction in strength. This condition is rare and should not occur when foundation is correctly designed.

15. Sturdi-Wall Plus Bracket Design Chart

Table 15.1 shows the shear, uplift, and bending strength for the SWP brackets. The values in Table 15.1 are based on calculations for the steel bracket, the steel-to-wood connection and the steel-to-concrete connection. The strength values for the steel-to-wood connection have been adjusted by the Load Duration Factor, C_D, of 1.6 (ASD) and the Time Effect Factor, λ, of 1.0 (LRFD) consistent with short duration loads. No further increases are permitted. The steel-to-concrete design values are for concrete with a minimum compressive strength, f'_c, of 3 ksi and a minimum concrete cover of two inches. The allowable loads for the concrete connection have been divided by 1.6 to convert from Load and Resistance Factor Design (LRFD) to Allowable Stress Design (ASD). The column base reactions may not exceed values in Table 15.1.

The steel-to-concrete calculations for Table 15.1 are limited only to the strength of rebar that is part of SWP bracket, compressive strength of concrete and rebar development. The foundation designer must ensure that concrete has sufficient strength at the bracket to receive the shear, bending (moment), uplift and downward forces and specify vertical, horizontal and transverse (stirrups) rebar as required by analysis.

Table 15.1: Shear, Uplift, and Bending Strength for Sturdi-Wall Plus Anchor Brackets

MODEL NO.	POST SIZE	ASD (Allowable Strength Design)			LRFD (Load and Resistance Factor Design)			Effective Rotational Stiffness	
		Bending (in-lb)	Shear (lb)	Uplift (lb)	Bending (in-lb)	Shear (lb)	Uplift (lb)	M/θ (in-k/deg)	M/θ (in-k/rad)
SWP46	4x6	24960	2100	4835	33670	2830	6515	34.8	1900
SWP63	3-ply 2x6	24960	2100	4835	33670	2830	6515	34.8	1900
SWP64	4-ply 2x6	31190	2380	4835	46880	3200	6515	45.4	2550
SWP66	6x6	24960	2100	4835	33670	2830	6515	35.6	1950
SWP83	3-ply 2x8	49430	3030	8490	66670	4080	11450	83.2	4700
SWP84	4-ply 2x8	49430	3030	8490	66670	4080	11450	79.7	4500
SWP85	5-ply 2x8	49430	3030	8210	66670	4080	11450	78.3	4450
SWP88	8x8	49430	3030	8010	66670	4080	11450	78.0	4400

Table 15.1 Footnotes:

1. This chart is for Sturdi-Wall Plus brackets for use in post-frame building applications to connect wood columns to a concrete wall or foundation.
2. Loads applied to the brackets from the columns are a vertical uplift force, horizontal shear force, and a moment about the strong axis of the column.

3. Steel-to-Wood connections were calculated as per the NDS 2018 using columns with Specific Gravity of 0.55, dry service conditions; if brackets are used in wet environment, tabulated shear, moment, and uplift values must be adjusted by the wet-service factor per the NDS.
4. The allowable loads in concrete have been divided by 1.6 to convert from LRFD to ASD.
5. Concrete design numbers are based on a minimum concrete compressive strength of 3000 psi
6. All rebar is weldable A706, Grade 60, #4 for 4, 5 and 6 series brackets and #5 for 8 series brackets
7. Bolts are 1/2" diameter SAE J429 Grade 5 with hex nuts
8. Screws are 1/4" diameter x 3" Strong Drive Screws (SDS) by Simpson Strong Tie, or alternative screws approved by Perma-Column
9. Minimum concrete cover for rebar shall be 3" when cast against and permanently exposed to earth, or 2" when exposed to earth or weather
10. Wood column above the bracket and concrete foundation below the bracket to be designed by others
11. Install all fasteners as per the manufacturer’s specifications and these notes
12. Final bracket design should include a complete building analysis performed by a design professional

16. Summary and Conclusion

Sturdi-Wall anchor brackets are designed to be used in a post-frame building application to connect wood columns to a concrete foundation. The standard Sturdi-Wall brackets are designed to transfer uplift, shear and downward forces from the wood column into the concrete foundation and can be installed with a wet-set or post-installed (drilled) anchors. The standard Sturdi-Wall anchor brackets do not have an ability to transfer any measurable moments from the wood column into the concrete foundation. Sturdi-Wall Plus anchor brackets, on the other hand, do provide a moment-resisting semi-rigid connection between the wood column and the concrete foundation. The SWP brackets, however, are only suitable for a wet-set installation. The building must be designed to resist lateral loads through diaphragm action or other bracing means. SW and SWP brackets are not an alternative to this requirement and are not recommended for columns that do not have a lateral restraint at the top.

This design manual can be downloaded from www.permacolumn.com

STURDI-WALL &
STURDI-WALL PLUS

SW46, SW55, SW63, SW64, SW65, SW66, SW60, SW 60C, SW83, SW84, SW85, SW88, SW80, SWP46, SWP63, SWP64, SWP66, SWP83, SWP84, SWP85 and SWP88 models

CALCULATIONS

(Revision 7)

IBC 2018

ACI 318-14
ANSI/AISC 360-16
ANSI/AWC NDS 2015

TTE Project Number E060-18

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April 2, 2022
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Revision 1 Changes:
Models SW88 and SWP88 are added to the calculations.

Revision 2 Changes:
Model SW60C is added to the calculations

Revision 3 Changes:
Models SW46, SW55, SW65 and SWP46 are added to the calculations.

Revision 4 Changes:
Section 11 (Shear Strength Calculations for SWP models) is added to the calculations

Revision 5 Changes:
Ultimate tensile strength, F_u, of all SW and SWP steel brackets in Sections 2 and 5 is decreased from 60 ksi to 55 ksi per ASTM A1018. The bending yield strength of the bolts in Sections 1, 4, 8 and 10 is increased from 45 ksi, to 106 ksi for SAE J429 Grade 5 bolts per 2015 NDS, Appendix I.4.

Revision 6 Changes:
Duplicated Sections 1, 4, 5, 8 and 10 (sections that include wood screw calculations). The Rev. 6 calculations now include values based on the SDS Simpson screws and the stronger Perma-Column proprietary screws. Sections 8.1 and 8.2 have been updated to include shear strength of the bracket-to-wood connections (changes to the narrative at the beginning of the section and changes to Tables 8.1E and 8.2E). Section 2 has been updated to include the effects of twisting in the universal brackets.

Revision 7 Changes:
Added Tables 10.1H and 10.2H (summary of uplift design values with reductions to SWP 46, 63, 64 and 66 models per latest test which was limited by values at 1/8” displacement). Minor units correction in Sections 10.1 and 10.2.

1.1 STURDI-WALL: SHEAR STRENGTH OF STEEL-TO-WOOD CONNECTION*

***WITH SDS SCREWS BY SIMPSON STRONG TIE**

Each Sturdi-Wall (SW) steel bracket is fastened to the wood column with 0.242"x3" structural screws and 1/2" SAE J429 Grade 5 through bolts. There are two fastener groups, the top fastener group and the bottom fastener group. The centroids of the fastener groups are separated by the distance "s". To calculate the location of each centroid group, it is necessary to first determine the slip-modulus for the 0.242" structural screw and the 1/2" through bolt. Per the *Wood Handbook* (FPL, 2010, United States Department of Agriculture Forest Service) the fastener slip-modulus for dowels in single shear in steel-to-wood application can be calculated using the following expression: $k = 270{,}000\ D^{1.5}$, where k is the slip-modulus and D is the fastener diameter. The slip modulus equation, however, does not include slippage due to fastener-hole clearance: a fastener has the freedom to move laterally with respect to the steel plate until it comes in contact with the edge of the hole in the steel plate. The holes for the screws and the bolts are 5/16" and 5/8" respectively. IF the fasteners are installed precisely through the center of the hole in the steel plate, the clearance on either side of the screw and the bolt is approximately 1/32" and 1/16", respectively. The screws will be engaged and start transferring load before the bolt may come in contact with the edge of the hole in the steel plate. For this reason, the slip-modulus of the bolt is reduced proportionally to the ratio of clearances: (1/32") / (1/16") = 0.5, or 50%. The individual and collective slip-modulus for each fastener group is shown in Table 1A.

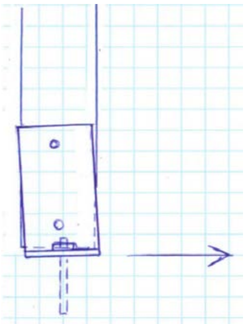


Figure 1A

From the free body diagram (Figure 1B), the shear force at the bottom of the bracket, between the bracket and the concrete surface, $V = F_B - F_T$, where F_B and F_T are horizontal forces at bottom and top fastener groups, respectively, and $F_B = V\,(s+y_B)\,/\,s$. From the same body diagram, it is evident that the force F_B is greater than F_T, and, since the type and quantity of fasteners in each group is the same, the shear strength of the SW bracket is controlled by the shear strength of the bottom fastener group: $V = s\,F_B\,/\,(s + y_B)$.

The load on each fastener type (screw, bolt) within the fastener group is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus of the entire fastener group: $N_s k_s/k_g$, $N_b k_b/k_g$, where N_s is the quantity of screws within the fastener group, N_b is the quantity of bolts in double shear within the fastener group, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of the entire fastener group (Table 1B). The slip-modulus of the screw fasteners does not equal the slip modulus of the bolt fastener(s): $N_s k_s \neq N_b k_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Table 2F). The allowable (ASD) and design (LRFD) shear strengths of each SW model as determined based on the steel-to-wood connection are provided in Table 1G.

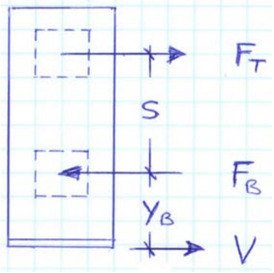


Figure 1B

The calculations are completed in Visual Analysis by IES and Microsoft Excel (2016) using the listed equations.

GOVERNING CODE:

National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

Allowable Lateral Strength of Screws	$Z'_{s,\ ASD} N_s = N_s\ Z\ C_D\ C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\ LRFD} N_s = \varphi\ N_s\ Z\ \lambda\ C_\Delta\ K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\ ASD} N_b = N_b\ Z\ C_D\ C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\ LRFD} N_b = \varphi\ N_b\ Z\ \lambda\ C_\Delta\ K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Slip Modulus for (1) screw, single shear	$k_s = 270{,}000\ D_s^{1.5}$	FPL, Chapter 8
Slip Modulus for (1) bolt, double shear	$k_b = 0.5\ [\ 2(270{,}000)\ D_b^{1.5}\ ]$	(see discussion above)
Slip Modulus for a Fastener Group	$k_g = N_s k_s + N_b k_b$	

N_s = quantity of screws in one fastener group

N_b = quantity of bolts in one fastener group

D_s = screw diameter

D_b = bolt diameter

Allowable Lateral Strength of Fastener Group	$Z'_{g,\ ASD} = \min\ [\ Z'_{s,\ ASD}\,(k_g/k_s)\ ,\ Z'_{b,\ ASD}\,(k_g/k_b)\ ]$
Design Lateral Strength of Fastener Group	$Z'_{g,\ LRFD} = \min\ [\ Z'_{s,\ LRFD}\,(k_g/k_s)\ ,\ Z'_{b,\ LRFD}\,(k_g/k_b)\ ]$

Allowable Shear Strength of Connection	$V_n/\Omega = s\ Z'_{g,\ ASD}\ /\ (s + y_B)$.
Design Shear Strength of Connection	$\phi V_n = s\ Z'_{g,\ LRFD}\ /\ (s + y_B)$.

s = distance between the centroids of the top and bottom fastener groups

CALCULATIONS:

TABLE 1.1A: SLIP MODLULUS OF FASTENERS

Model	D_s (in)	D_b (in)	k_s (lb/in)	k_b (lb/in)	N_s	N_b	k_g (lb/in)
SW 46	0.242	0.50	32143	95459	2	1	159745
SW 55	0.242	0.50	32143	95459	2	1	159745
SW 63	0.242	0.50	32143	95459	2	1	159745
SW 64	0.242	0.50	32143	95459	2	1	159745
SW 65	0.242	0.50	32143	95459	2	1	159745
SW 66	0.242	0.50	32143	95459	2	1	159745
SW 83	0.242	0.50	32143	95459	4	1	224032
SW 84	0.242	0.50	32143	95459	4	1	224032
SW 85	0.242	0.50	32143	95459	4	1	224032
SW 88	0.242	0.50	32143	95459	4	1	224032
SW 60*	0.242	0.50	32143	95459	2	1	159745
SW 80*	0.242	0.50	32143	95459	4	1	224032
SW 60C*	0.242	n/a	32143	n/a	3	n/a	96429

* Pair

TABLE 1.1B: LOCATION OF AND DISTANCE BETWEEN THE CENTROIDS OF THE TOP AND BOTTOM FASTENER GROUPS

	k_s	k_b	Elevation (in)							**y_B**	**y_T**	**s**
Model	(lb/in)	(lb/in)	Base	Bolt 1	Screw 1	Screw 2	Screw 3	Screw 4	Bolt 2	**(in)**	**(in)**	**(in)**
SW 46	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 55	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 63	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 64	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 65	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 66	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 83	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 84	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 85	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 88	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 60	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 80	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 60C	32143	32143	0	3.875	4.375	5.375	11.125	11.625	12.625	**4.542**	**11.79**	**7.25**

TABLE 1.1C: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A (Yield Limit Equations)

		SDS	F_{yb}	164000	$1+R_e$	1.1			θ	90
Screw Diameter (in)	D	0.242	$F_{em,par}$	5526	$2+R_e$	2.1			I_m	1259.3
Screw Length (in)	L	3	$F_{em,perp}$	5526	k_1	0.408			I_s	1280.4
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.536			II	522.4
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	6.944			III_m	572.7
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es,par}$	61800			III_s	380.5
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.920	$F_{es,perp}$	61800			IV	472.3
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800			D_r	0.242
Lateral Design Value (lbs)	Z	380		LRFD resistance factor			ϕ	0.65		
ASD Load Duration Factor	C_D	1.6		LRFD time effect factor			λ	1		
Geometry Factor	C_Δ	1		ASD to LRFD format conversion factor			K_F	3.32		
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s,ASD}$**	**609**		**LRFD Adjusted Lateral Design Value (lbs)**			**$Z'_{s,LRFD}$**	**821**		

TABLE 1.1D: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A (Yield Limit Equations)

Bolt Diameter (in)	D	0.5	$F_{em,par}$	6160	K_θ	1.250			I_m	1631
Main Member Thickness (in)	$t_{m,min}$	4.5	$F_{em,perp}$	3626	$1+R_e$	1.042			III_s	1494
Side Member Thickness (in)	t_s	0.25	F_{em}	3626	$2+R_e$	2.042			IV	1960
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.042	k_3	13.463				
Bolt Yield Strength (psi)	F_{yb}	106000								
Max Angle Load to Grain (deg)	θ	90								
Specific Gravity	G	0.55								
Reference Lateral Design Value (Z)	Z	1494		LRFD resistance factor			ϕ	0.65		
ASD Load Duration Factor	C_D	1.6		LRFD time effect factor			λ	1		
Geometry Factor	C_Δ	1		ASD to LRFD format conversion factor			K_F	3.32		
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b,ASD}$**	**2391**		**LRFD Adjusted Lateral Design Value (lbs)**			**$Z'_{b,LRFD}$**	**3224**		

TABLE 1.1E: LATERAL (SHEAR) STRENGTH OF EACH FASTENER GROUP

Model	k_s (lb/in)	k_b (lb/in)	k_g (lb/in)	$Z'_{s,\,ASD}$ (k_g/k_s) (lb)	$Z'_{b,\,ASD}$ (k_g/k_b) (lb)	$Z'_{s,\,LRFD}$ (k_g/k_s) (lb)	$Z'_{b,\,LRFD}$ (k_g/k_b) (lb)	**$Z'_{g,\,ASD}$ (lb)**	**$Z'_{g,\,LRFD}$ (lb)**
SW 46	32143	95459	159745	3026	4001	4081	5396	**3026**	**4081**
SW 55	32143	95459	159745	3026	4001	4081	5396	**3026**	**4081**
SW 63	32143	95459	159745	3026	4001	4081	5396	**3026**	**4081**
SW 64	32143	95459	159745	3026	4001	4081	5396	**3026**	**4081**
SW 65	32143	95459	159745	3026	4001	4081	5396	**3026**	**4081**
SW 66	32143	95459	159745	3026	4001	4081	5396	**3026**	**4081**
SW 83	32143	95459	224032	4243	5611	5723	7567	**4243**	**5723**
SW 84	32143	95459	224032	4243	5611	5723	7567	**4243**	**5723**
SW 85	32143	95459	224032	4243	5611	5723	7567	**4243**	**5723**
SW 88	32143	95459	224032	4243	5611	5723	7567	**4243**	**5723**
SW 60*	32143	95459	159745	3026	4001	4081	5396	**3026**	**4081**
SW 80*	32143	95459	224032	4243	5611	5723	7567	**4243**	**5723**
SW 60C	32143	n/a	96429	1826	n/a	2463	n/a	**1826**	**2463**

*Pair

TABLE 1.1F: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO

	N_s	N_b	Load Distribution		Load / Strength	
Model			Screws	Bolts	Screws	Bolts
SW 46	2	1	40.2%	59.8%	100.0%	76%
SW 55	2	1	40.2%	59.8%	100.0%	76%
SW 63	2	1	40.2%	59.8%	100.0%	76%
SW 64	2	1	40.2%	59.8%	100.0%	76%
SW 65	2	1	40.2%	59.8%	100.0%	76%
SW 66	2	1	40.2%	59.8%	100.0%	76%
SW 83	4	1	57.4%	42.6%	100.0%	76%
SW 84	4	1	57.4%	42.6%	100.0%	76%
SW 85	4	1	57.4%	42.6%	100.0%	76%
SW 88	4	1	57.4%	42.6%	100.0%	76%
SW 60*	2	1	40.2%	59.8%	100.0%	76%
SW 80*	4	1	57.4%	42.6%	100.0%	76%
SW 60C*	3	0	100.0%	0.0%	100.0%	0%

*Pair

TABLE 1.1G: SHEAR STRENGTH OF STEEL-TO-WOOD CONNECTION

Model	**ASD** **V_n/Ω** **(lb)**	**LRFD** **ϕV_n** **(lb)**
SW 46	**2100**	**2830**
SW 55	**2100**	**2830**
SW 63	**2100**	**2830**
SW 64	**2100**	**2830**
SW 65	**2100**	**2830**
SW 66	**2100**	**2830**
SW 83	**3030**	**4080**
SW 84	**3030**	**4080**
SW 85	**3030**	**4080**
SW 88	**3030**	**4080**
SW 60*	**2100**	**2830**
SW 80*	**3030**	**4080**
SW 60C	**1120**	**1510**

*Pair

NOTE: SW 60C bracket is a corner post bracket that is intended to be used alone (not in pairs). These calculations assume that wall girts and other secondary will provide torsional stability to the corner column. This bracket may not be used with columns that do not have sufficient girt/secondary framing to resist torsional loads.

1.2 STURDI-WALL: SHEAR STRENGTH OF STEEL-TO-WOOD CONNECTION*

***WITH PROPRIETARY SCREWS**

Each Sturdi-Wall (SW) steel bracket is fastened to the wood column with 0.242"x3" structural screws and 1/2" SAE J429 Grade 5 through bolts. There are two fastener groups, the top fastener group and the bottom fastener group. The centroids of the fastener groups are separated by the distance "s". To calculate the location of each centroid group, it is necessary to first determine the slip-modulus for the 0.242" structural screw and the 1/2" through bolt. Per the *Wood Handbook* (FPL, 2010, United States Department of Agriculture Forest Service) the fastener slip-modulus for dowels in single shear in steel-to-wood application can be calculated using the following expression: $k = 270{,}000\ D^{1.5}$, where k is the slip-modulus and D is the fastener diameter. The slip modulus equation, however, does not include slippage due to fastener-hole clearance: a fastener has the freedom to move laterally with respect to the steel plate until it comes in contact with the edge of the hole in the steel plate. The holes for the screws and the bolts are 5/16" and 5/8" respectively. IF the fasteners are installed precisely through the center of the hole in the steel plate, the clearance on either side of the screw and the bolt is approximately 1/32" and 1/16", respectively. The screws will be engaged and start transferring load before the bolt may come in contact with the edge of the hole in the steel plate. For this reason, the slip-modulus of the bolt is reduced proportionally to the ratio of clearances: (1/32") / (1/16") = 0.5, or 50%. The individual and collective slip-modulus for each fastener group is shown in Table 1A.

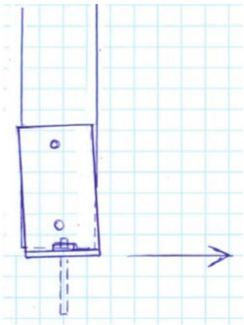


Figure 1A

From the free body diagram (Figure 1B), the shear force at the bottom of the bracket, between the bracket and the concrete surface, $V = F_B - F_T$, where F_B and F_T are horizontal forces at bottom and top fastener groups, respectively, and $F_B = V\ (s+y_B)\ /\ s$. From the same body diagram, it is evident that the force F_B is greater than F_T, and, since the type and quantity of fasteners in each group is the same, the shear strength of the SW bracket is controlled by the shear strength of the bottom fastener group: $V = s\ F_B\ /\ (s + y_B)$.

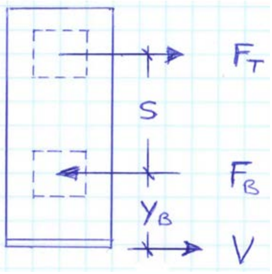


Figure 1B

The load on each fastener type (screw, bolt) within the fastener group is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus of the entire fastener group: N_sk_s/k_g, N_bk_b/k_g, where N_s is the quantity of screws within the fastener group, N_b is the quantity of bolts in double shear within the fastener group, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of the entire fastener group (Table 1B). The slip-modulus of the screw fasteners does not equal the slip modulus of the bolt fastener(s): $N_sk_s \neq N_bk_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Table 2F). The allowable (ASD) and design (LRFD) shear strengths of each SW model as determined based on the steel-to-wood connection are provided in Table 1G.

The calculations are completed in Visual Analysis by IES and Microsoft Excel (2016) using the listed equations.

GOVERNING CODE:

National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

Allowable Lateral Strength of Screws	$Z'_{s,\ ASD}\, N_s = N_s\, Z\, C_D\, C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\ LRFD}\, N_s = \varphi\, N_s\, Z\, \lambda\, C_\Delta\, K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\ ASD}\, N_b = N_b\, Z\, C_D\, C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\ LRFD}\, N_b = \varphi\, N_b\, Z\, \lambda\, C_\Delta\, K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Slip Modulus for (1) screw, single shear	**$k_s = 270{,}000\ D_s^{1.5}$**	FPL, Chapter 8
Slip Modulus for (1) bolt, double shear	**$k_b = 0.5\ [\ 2(270{,}000)\ D_b^{1.5}\ ]$**	(see discussion above)
Slip Modulus for a Fastener Group	**$k_g = N_s k_s + N_b k_b$**	

N_s = quantity of screws in one fastener group

N_b = quantity of bolts in one fastener group

D_s = screw diameter

D_b = bolt diameter

Allowable Lateral Strength of Fastener Group	**$Z'_{g,\,ASD} = \min\ [\ Z'_{s,\,ASD}\,(k_g/k_s)\ ,\ Z'_{b,\,ASD}\,(k_g/k_b)\ ]$**
Design Lateral Strength of Fastener Group	**$Z'_{g,\,LRFD} = \min\ [\ Z'_{s,\,LRFD}\,(k_g/k_s)\ ,\ Z'_{b,\,LRFD}\,(k_g/k_b)\ ]$**

Allowable Shear Strength of Connection	**$V_n/\Omega = s\ Z'_{g,\,ASD}\ /\ (s + y_B)$.**
Design Shear Strength of Connection	**$\phi V_n = s\ Z'_{g,\,LRFD}\ /\ (s + y_B)$.**

s = distance between the centroids of the top and bottom fastener groups

CALCULATIONS:

TABLE 1.2A: SLIP MODLULUS OF FASTENERS

Model	D_s (in)	D_b (in)	k_s (lb/in)	k_b (lb/in)	N_s	N_b	k_g (lb/in)
SW 46	0.242	0.50	32143	95459	2	1	159745
SW 55	0.242	0.50	32143	95459	2	1	159745
SW 63	0.242	0.50	32143	95459	2	1	159745
SW 64	0.242	0.50	32143	95459	2	1	159745
SW 65	0.242	0.50	32143	95459	2	1	159745
SW 66	0.242	0.50	32143	95459	2	1	159745
SW 83	0.242	0.50	32143	95459	4	1	224032
SW 84	0.242	0.50	32143	95459	4	1	224032
SW 85	0.242	0.50	32143	95459	4	1	224032
SW 88	0.242	0.50	32143	95459	4	1	224032
SW 60*	0.242	0.50	32143	95459	2	1	159745
SW 80*	0.242	0.50	32143	95459	4	1	224032
SW 60C*	0.242	n/a	32143	n/a	3	n/a	96429

* Pair

TABLE 1.2B: LOCATION OF AND DISTANCE BETWEEN THE CENTROIDS OF THE TOP AND BOTTOM FASTENER GROUPS

	k_s	k_b	Elevation (in)							**y_B**	**y_T**	**s**
Model	(lb/in)	(lb/in)	Base	Bolt 1	Screw 1	Screw 2	Screw 3	Screw 4	Bolt 2	**(in)**	**(in)**	**(in)**
SW 46	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 55	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 63	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 64	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 65	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 66	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 83	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 84	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 85	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 88	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 60	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	**3.627**	**11.87**	**8.25**
SW 80	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	**4.680**	**16.32**	**11.64**
SW 60C	32143	32143	0	3.875	4.375	5.375	11.125	11.625	12.625	**4.542**	**11.79**	**7.25**

TABLE 1.2C: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A (Yield Limit Equations)

			F_{yb}	199000	$1+R_e$	1.1	θ	90
Screw Diameter (in)	D	0.243	$F_{em, par}$	5526	$2+R_e$	2.1	I_m	1260.2
Screw Length (in)	L	3	$F_{em, perp}$	5526	k_1	0.408	I_s	1281.3
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.549	II	522.7
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	7.471	III_m	587.1
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es, par}$	61800	III_s	409.7
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.930	$F_{es, perp}$	61800	IV	522.8
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800	D_r	0.243
Lateral Design Value (lbs)	Z	410	LRFD resistance factor				ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor				λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor				K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s, ASD}$**	**655**	**LRFD Adjusted Lateral Design Value (lbs)**				**$Z'_{s, LRFD}$**	**884**

TABLE 1.2D: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A (Yield Limit Equations)

Bolt Diameter (in)	D	0.5	$F_{em, par}$	6160	K_θ	1.250	I_m	1631
Main Member Thickness (in)	$t_{m, min}$	4.5	$F_{em, perp}$	3626	$1+R_e$	1.042	III_s	1494
Side Member Thickness (in)	t_s	0.25	F_{em}	3626	$2+R_e$	2.042	IV	1960
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.042	k_3	13.463		
Bolt Yield Strength (psi)	F_{yb}	106000						
Max Angle Load to Grain (deg)	θ	90						
Specific Gravity	G	0.55						
Reference Lateral Design Value (Z)	Z	1494	LRFD resistance factor				ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor				λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor				K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b, ASD}$**	**2391**	**LRFD Adjusted Lateral Design Value (lbs)**				**$Z'_{b, LRFD}$**	**3224**

TABLE 1.2E: LATERAL (SHEAR) STRENGTH OF EACH FASTENER GROUP

Model	k_s (lb/in)	k_b (lb/in)	k_g (lb/in)	$Z'_{s,\,ASD}$ (k_g/k_s) (lb)	$Z'_{b,\,ASD}$ (k_g/k_b) (lb)	$Z'_{s,\,LRFD}$ (k_g/k_s) (lb)	$Z'_{b,\,LRFD}$ (k_g/k_b) (lb)	**$Z'_{g,\,ASD}$ (lb)**	**$Z'_{g,\,LRFD}$ (lb)**
SW 46	32143	95459	159745	3258	4001	4394	5396	**3258**	**4394**
SW 55	32143	95459	159745	3258	4001	4394	5396	**3258**	**4394**
SW 63	32143	95459	159745	3258	4001	4394	5396	**3258**	**4394**
SW 64	32143	95459	159745	3258	4001	4394	5396	**3258**	**4394**
SW 65	32143	95459	159745	3258	4001	4394	5396	**3258**	**4394**
SW 66	32143	95459	159745	3258	4001	4394	5396	**3258**	**4394**
SW 83	32143	95459	224032	4569	5611	6162	7567	**4569**	**6162**
SW 84	32143	95459	224032	4569	5611	6162	7567	**4569**	**6162**
SW 85	32143	95459	224032	4569	5611	6162	7567	**4569**	**6162**
SW 88	32143	95459	224032	4569	5611	6162	7567	**4569**	**6162**
SW 60*	32143	95459	159745	3258	4001	4394	5396	**3258**	**4394**
SW 80*	32143	95459	224032	4569	5611	6162	7567	**4569**	**6162**
SW 60C	32143	n/a	96429	1966	n/a	2652	n/a	**1966**	**2652**

*Pair

TABLE 1.2F: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO

	N_s	N_b	Load Distribution		Load / Strength	
Model			Screws	Bolts	Screws	Bolts
SW 46	2	1	40.2%	59.8%	100.0%	81%
SW 55	2	1	40.2%	59.8%	100.0%	81%
SW 63	2	1	40.2%	59.8%	100.0%	81%
SW 64	2	1	40.2%	59.8%	100.0%	81%
SW 65	2	1	40.2%	59.8%	100.0%	81%
SW 66	2	1	40.2%	59.8%	100.0%	81%
SW 83	4	1	57.4%	42.6%	100.0%	81%
SW 84	4	1	57.4%	42.6%	100.0%	81%
SW 85	4	1	57.4%	42.6%	100.0%	81%
SW 88	4	1	57.4%	42.6%	100.0%	81%
SW 60*	2	1	40.2%	59.8%	100.0%	81%
SW 80*	4	1	57.4%	42.6%	100.0%	81%
SW 60C*	3	0	100.0%	0.0%	100.0%	0%

*Pair

TABLE 1.2G: SHEAR STRENGTH OF STEEL-TO-WOOD CONNECTION

Model	**ASD** **V_n/Ω** **(lb)**	**LRFD** **ϕV_n** **(lb)**
SW 46	**2260**	**3050**
SW 55	**2260**	**3050**
SW 63	**2260**	**3050**
SW 64	**2260**	**3050**
SW 65	**2260**	**3050**
SW 66	**2260**	**3050**
SW 83	**3260**	**4390**
SW 84	**3260**	**4390**
SW 85	**3260**	**4390**
SW 88	**3260**	**4390**
SW 60*	**2260**	**3050**
SW 80*	**3260**	**4390**
SW 60C	**1210**	**1630**

*Pair

NOTE: SW 60C bracket is a corner post bracket that is intended to be used alone (not in pairs). These calculations assume that wall girts and other secondary will provide torsional stability to the corner column. This bracket may not be used with columns that do not have sufficient girt/secondary framing to resist torsional loads.

2. STURDI-WALL: SHEAR STRENGTH OF STEEL BRACKET

The calculations are presented in both the LRFD and ASD formats in accordance with the provisions of the governing code (AISC 360-16). The calculations are completed in Microsoft Excel (2016) using the listed equations.

The design (LRFD) and allowable (ASD) shear strength calculations are for vertical steel plates, horizontal steel plate, and welds.

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-16

GOVERNING EQUATIONS:

Design Shear Strength	$\varphi V_n = \varphi\, 0.6\, F_y\, A_g$	**(shear yielding)**	$\phi = 1.0$	(J4-4)
Design Shear Strength	$\varphi V_n = \varphi\, 0.6\, F_u\, A_{nv}$	**(shear rupture)**	$\phi = 0.75$	(J4-4)
Allowable Shear Strength	$V_n / \Omega = 0.6\, F_y\, A_g / \Omega$	**(shear yielding)**	$\Omega = 1.5$	(J4-4)
Allowable Shear Strength	$V_n / \Omega = 0.6\, F_u\, A_{nv} / \Omega$	**(shear rupture)**	$\Omega = 2$	(J4-4)

- **WELDS: AISC 360, SECTION J2**

Design Strength	$\phi R_n = \phi F_w A_w$	$\phi = 0.75$	(J2-3)
Allowable Strength	$R_n / \Omega = F_w A_w / \Omega$	$\Omega = 2.00$	(J2-3)
	$F_w = 0.60 F_{EXX}$		(T. J2.5)

CALCULATIONS:

STEEL BRACKET PROPERTIES		
Minimum Tensile Strength, F_u	55	ksi
Minimum Yield Strength, Fy	40	ksi
Thickness of steel, t	0.250	in

WELD PROPERTIES		
Fillet Weld Leg Size	0.25	in
Effective Weld Thickness (throat) , t_e	0.18	in
Electrode Classification Number	70	ksi
Nominal Strength of Weld Metal, F_w	42	ksi

TABLE 2A: SHEAR STRENGTH OF STEEL PLATES AND WELDS

	Weld Strength			Shear Strength of Steel Plates					
				Yielding			Rupture		
Model ID	A_w (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)	A_g (in^2)	LRFD ϕV_n (lb)	ASD V_n / Ω (lb)	A_e (in^2)	LRFD ϕV_n (lb)	ASD V_n / Ω (lb)
SW 46	3.54	111353	74235	2.50	60000	40000	2.19	54203	36135
SW 55	3.54	111510	74340	2.50	60000	40000	2.19	54203	36135
SW 63	3.54	111510	74340	2.50	60000	40000	2.19	54203	36135
SW 64	3.54	111353	74235	2.50	60000	40000	2.19	54203	36135
SW 65	3.54	111510	74340	2.50	60000	40000	2.19	54203	36135
SW 66	3.54	111353	74235	2.50	60000	40000	2.19	54203	36135
SW 83	4.95	155894	103929	3.50	84000	56000	3.19	78953	52635
SW 84	4.95	155894	103929	3.50	84000	56000	3.19	78953	52635
SW 85	4.95	155894	103929	3.50	84000	56000	3.19	78953	52635
SW 88	4.95	155894	103929	3.50	84000	56000	3.19	78953	52635
SW 60*	n/a	n/a	n/a	2.50	60000	40000	2.19	54203	36135
SW 80*	n/a	n/a	n/a	3.50	84000	56000	3.19	78953	52635
SW 60C*	n/a	n/a	n/a	1.00	24000	16000	0.92	22770	15180

*Pair

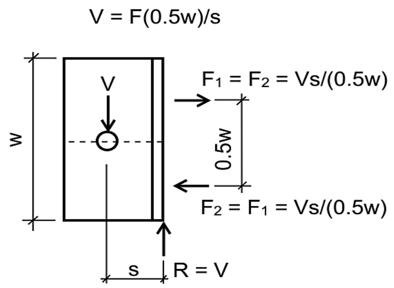
TABLE 2B: SHEAR STRENGTH BASED ON TWISTING STRENGTH OF THE ANGLES

Model ID	t (in)	w (in)	ϕM_n (in-lb)	M_n / Ω (in-lb)	y (in)	s (in)	LRFD ϕV_n (lb) pair	ASD V_n / Ω (lb) pair
SW 60	0.250	5.00	1406	936	3.0000	2.38	1970	1310
SW 80	0.250	7.00	1969	1310	3.5000	2.38	3320	2210
SW 60C	0.250	4.00	1125	749	3.5000	2.38	1080	720

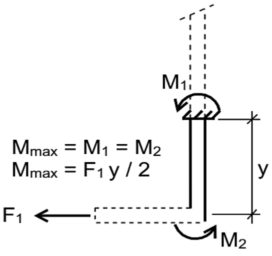
y = moment arm: distance from center of 1st bolt to center of short steel leg

φM_n = design bending strength of 1/2 of the steel angle

M_n / Ω = allowable bending strength of 1/2 of the steel angle



TOP VIEW



SIDE VIEW

3. STURDI-WALL: SHEAR STRENGTH OF ANCHORS

The calculations are presented in both the LRFD and ASD formats in accordance with the provisions of the governing codes (AISC 360-16 and ACI 318-14). The calculations include (1) shear strength of steel anchors and (2) concrete prying or breakout forces. The shear strength of steel anchors is covered in both ACI 318 and AISC 360 standards. It is desirable to present the results in terms of ASD and LRFD design, therefore, AISC 360, which includes both methods, is used for steel anchor design calculations. The breakout calculations are completed using the LRFD method per ACI 318 and are converted to the ASD levels using the conversion factor, α = 1.6. The calculations are completed in Microsoft Excel (2016) using the listed equations.

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-16
Building Code Requirements for Structural Concrete ACI 318-14

GOVERNING EQUATIONS:

- **BOLTS (ANCHORS): AISC 360, SECTION J3**

Design Strength	$\phi R_n N_b = \phi N_b F_{nv} A_b$	$\phi = 0.75$	(J3-1)
Allowable Strength	$R_n N_b / \Omega = N_b F_{nv} A_b / \Omega$	$\Omega = 2.00$	(J3-1)

N_b = number of bolts

- **ANCHORS (CONCRETE BREAKOUT): ACI 318, 17.5.2.1**

Design Strength	$\varphi V_{cbg} = \varphi(A_{Vc} / A_{Vco})\ \Psi_{ec,V}\ \Psi_{ed,V}\ \Psi_{c,V}\ \Psi_{h,V}\ V_b$	$\phi = 0.70$	(17.5.2.1b)
Allowable Strength	$V_{cbg}/\Omega = \varphi V_{cbg} / \alpha$	$\alpha = 1.6$	

CALCULATIONS:

ANCHOR PROPERTIES			CONCRETE PROPERTIES		
Nominal Shear Stress, F_{nv}	24 (or greater)	ksi	Compressive Strength, f'_c	3 (or greater)	ksi

TABLE 3A: SHEAR STRENGTH OF ANCHORS

Model ID	N_b	1/2" A_b (in^2)	5/8" A_b (in^2)	Shear Strength of Anchors 1/2" LRFD $\phi R_n N_b$ (lbf)	1/2" ASD $R_n N_b / \Omega$ (lbf)	5/8" LRFD $\phi R_n N_b$ (lbf)	5/8" ASD $R_n N_b / \Omega$ (lbf)
SW 46	2	0.20	0.31	7069	4712	11045	7363
SW 55	2	0.20	0.31	7069	4712	11045	7363
SW 63	2	0.20	0.31	7069	4712	11045	7363
SW 64	2	0.20	0.31	7069	4712	11045	7363
SW 65	2	0.20	0.31	7069	4712	11045	7363
SW 66	2	0.20	0.31	7069	4712	11045	7363
SW 83	2	0.20	0.31	7069	4712	11045	7363
SW 84	2	0.20	0.31	7069	4712	11045	7363
SW 85	2	0.20	0.31	7069	4712	11045	7363
SW 88	2	0.20	0.31	7069	4712	11045	7363
SW 60*	2	0.20	0.31	7069	4712	11045	7363
SW 80*	2	0.20	0.31	7069	4712	11045	7363
SW 60C*	2	0.20	0.31	7069	4712	11045	7363

*Pair

TABLE 3B: SHEAR STRENGTH OF ANCHORS: CONCRETE BREAKOUT

							1/2"	5/8"	1/2"		5/8"	
									LRFD	**ASD**	**LRFD**	**ASD**
Model ID	c_{a1} (in)	S_1 (in)	A_{Vc} (in^2)	A_{Vco} (in^2)	Ψ	h_{ef} (in)	V_b (lb)	V_b (lb)	φV_{cbg} (lb)	V_{cbg}/Ω (lb)	φV_{cbg} (lb)	V_{cbg}/Ω (lb)
SW 46	4.0	9.375	128.3	72.0	1.0	4.3	3335	3566	4159	2599	4447	2779
SW 55	4.0	9.375	128.3	72.0	1.0	4.3	3335	3566	4159	2599	4447	2779
SW 63	4.0	9.375	128.3	72.0	1.0	4.3	3335	3566	4159	2599	4447	2779
SW 64	4.0	10.875	137.3	72.0	1.0	4.3	3335	3566	4451	2782	4759	2974
SW 65	4.0	12.375	144.0	72.0	1.0	4.3	3335	3566	4669	2918	4993	3120
SW 66	4.0	10.875	137.3	72.0	1.0	4.3	3335	3566	4451	2782	4759	2974
SW 83	5.0	9.375	182.8	112.5	1.0	4.3	4661	4984	5302	3314	5669	3543
SW 84	5.0	10.875	194.1	112.5	1.0	4.3	4661	4984	5628	3518	6018	3761
SW 85	5.0	12.375	205.3	112.5	1.0	4.3	4661	4984	5955	3722	6367	3979
SW 88	5.0	12.875	209.1	112.5	1.0	4.3	4661	4984	6063	3790	6483	4052
SW 60	4.0	9.375	128.3	72.0	1.0	4.3	3335	3566	4159	2599	4447	2779
SW 80	5.0	9.375	182.8	112.5	1.0	4.3	4661	4984	5302	3314	5669	3543
SW 60C	4.0	0.000	72.0	72.0	1.0	4.3	3335	3566	2335	1459	2496	1560

(1) SW 60, SW 60C and SW 80 angles are to be used in pairs

(2) c_{a1} = distance from center of anchor to edge of concrete

(3) S_1 = spacing between anchors

(4) $\Psi_{ec,V}$ $\Psi_{ed,V}$ $\Psi_{c,V}$ $\Psi_{h,V}$ = 1.0, higher values possible if concrete is properly reinforced around the anchors

(5) h_{ef} is based on 6 in anchor, 5.75" embedment into the concrete

(6) c_{a1}, S_1, A_{Vc}, A_{Vco}, Ψ, hef, V_b, V_{cbg} as defined in ACI 318.

4.1. STURDI-WALL: UPLIFT (TENSION) STRENGTH OF THE STEEL-TO-WOOD CONNECTION*

***WITH SDS SCREWS BY SIMPSON STRONG TIE**

The calculations are presented in both the LRFD and ASD formats in accordance with provisions of the governing code (NDS 2015). The calculations are completed in Microsoft Excel (2016) using the listed equations.

The load on each fastener type (screw, bolt) is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus all fasteners: $N_s k_s/k_g$, $N_b k_b/k_g$, where N_s is the quantity of screws per bracket, N_b is the quantity of bolts in double shear per bracket, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of all fasteners (Tables 4C). The slip-modulus of screw fasteners does not equal the slip modulus of bolt fastener: $N_s k_s \neq N_b k_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Table 4E). The discussion and calculations for slip-modulus are provided in Section 1.

Table 4D shows the uplift strength of the SW steel brackets based on the lateral (shear) strength of the steel-to-wood connection.

GOVERNING CODE:

National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

Allowable Lateral Strength of Screws	$Z'_{s,\,ASD}\,N_s = N_s\,Z\,C_D\,C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\,LRFD}\,N_s = \varphi\,N_s\,Z\,\lambda\,C_\Delta\,K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\,ASD}\,N_b = N_b\,Z\,C_D\,C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\,LRFD}\,N_b = \varphi\,N_b\,Z\,\lambda\,C_\Delta\,K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Allowable Uplift Strength of Steel-to-Wood Connection	$T_n/\Omega = \min\ [\,Z'_{s,\,ASD}\,(k_g/k_s)\ ,\ Z'_{b,\,ASD}\,(k_g/k_b)\,]$
Design Uplift Strength of Steel-to-Wood Connection	$\phi T_n = \min\ [\,Z'_{s,\,LRFD}\,(k_g/k_s)\ ,\ Z'_{b,\,LRFD}\,(k_g/k_b)\,]$

CALCULATIONS:

TABLE 4.1A: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A

		SDS	F_{yb}	164000	$1+R_e$	1.1	θ	0	
Screw Diameter (in)	D	0.242	$F_{em, par}$	5526	$2+R_e$	2.1	I_m	1259.3	
Screw Length (in)	L	3	$F_{em, perp}$	5526	k_1	0.408	I_s	1280.4	
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.536	II	522.4	
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	6.944	III_m	572.7	
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es, par}$	61800	III_s	380.5	
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.920	$F_{es, perp}$	61800	IV	472.3	
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800	D_r	0.242	
Lateral Design Value (lbs)	Z	380	LRFD resistance factor					φ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor					λ	1
Geometry Factor	$C_Δ$	1	ASD to LRFD format conversion factor					K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s, ASD}$**	**609**	**LRFD Adjusted Lateral Design Value (lbs)**					**$Z'_{s, LRFD}$**	**821**

TABLE 4.1B: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A

Bolt Diameter (in)	D	0.5	$F_{em, par}$	6160	$K_θ$	1.000	I_m	3465	
Main Member Thickness (in)	$t_{m, min}$	4.5	$F_{em, perp}$	3626	$1+R_e$	1.071	III_s	2369	
Side Member Thickness (in)	t_s	0.25	F_{em}	6160	$2+R_e$	2.071	IV	3150	
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.071	k_3	10.192			
Bolt Yield Strength (psi)	F_{yb}	106000							
Max Angle Load to Grain (deg)	θ	0							
Specific Gravity	G	0.55							
Reference Lateral Design Value (Z)	Z	2369	LRFD resistance factor					φ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor					λ	1
Geometry Factor	$C_Δ$	1	ASD to LRFD format conversion factor					K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b, ASD}$**	**3790**	**LRFD Adjusted Lateral Design Value (lbs)**					**$Z'_{b, LRFD}$**	**5112**

TABLE 4.1C: FASTENER SLIP-MODULUS

Model	k_s (lb/in)	k_b (lb/in)	N_s	N_b	k_g (lb/in)
SW 46	32143	95459	4	2	319491
SW 55	32143	95459	4	2	319491
SW 63	32143	95459	4	2	319491
SW 64	32143	95459	4	2	319491
SW 65	32143	95459	4	2	319491
SW 66	32143	95459	4	2	319491
SW 83	32143	95459	8	2	448063
SW 84	32143	95459	8	2	448063
SW 85	32143	95459	8	2	448063
SW 88	32143	95459	8	2	448063
SW 60*	32143	95459	4	2	319491
SW 80*	32143	95459	8	2	448063
SW 60C*	32143	n/a	12	0	385716

*Pair

TABLE 4.1D: TENSILE STRENGTH BASED ON STEEL-TO-WOOD CONNECTION

Model	$Z'_{s,\ ASD}$ (k_g/k_s) (lb)	$Z'_{b,\ ASD}$ (k_g/k_b) (lb)	$Z'_{s,\ LRFD}$ (k_g/k_s) (lb)	$Z'_{b,\ LRFD}$ (k_g/k_b) (lb)	**ASD** **T_n/Ω** **(lb)**	**LRFD** **ϕT_n** **(lb)**
SW 46	6051	12684	8161	17108	**6050**	**8160**
SW 55	6051	12684	8161	17108	**6050**	**8160**
SW 63	6051	12684	8161	17108	**6050**	**8160**
SW 64	6051	12684	8161	17108	**6050**	**8160**
SW 65	6051	12684	8161	17108	**6050**	**8160**
SW 66	6051	12684	8161	17108	**6050**	**8160**
SW 83	8486	17789	11446	23993	**8490**	**11450**
SW 84	8486	17789	11446	23993	**8490**	**11450**
SW 85	8486	17789	11446	23993	**8490**	**11450**
SW 88	8486	17789	11446	23993	**8490**	**11450**
SW 60*	6051	12684	8161	17108	**6050**	**8160**
SW 80*	8486	17789	11446	23993	**8490**	**11450**
SW 60C*	7305	n/a	9853	n/a	**7310**	**9850**

*Pair

TABLE 4.1E: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO

	Load Distribution		Load / Strength	
Model	Screws	Bolts	Screws	Bolts
SW 46	40.2%	59.8%	100%	48%
SW 55	40.2%	59.8%	100%	48%
SW 63	40.2%	59.8%	100%	48%
SW 64	40.2%	59.8%	100%	48%
SW 65	40.2%	59.8%	100%	48%
SW 66	40.2%	59.8%	100%	48%
SW 83	57.4%	42.6%	100%	48%
SW 84	57.4%	42.6%	100%	48%
SW 85	57.4%	42.6%	100%	48%
SW 88	57.4%	42.6%	100%	48%
SW 60*	40.2%	59.8%	100%	48%
SW 80*	57.4%	42.6%	100%	48%
SW 60C*	100.0%	0.0%	100%	0%

*Pair

4.2. STURDI-WALL: UPLIFT (TENSION) STRENGTH OF THE STEEL-TO-WOOD CONNECTION*

***WITH PROPRIETARY SCREWS**

The calculations are presented in both the LRFD and ASD formats in accordance with provisions of the governing code (NDS 2015). The calculations are completed in Microsoft Excel (2016) using the listed equations.

The load on each fastener type (screw, bolt) is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus all fasteners: $N_s k_s/k_g$, $N_b k_b/k_g$, where N_s is the quantity of screws per bracket, N_b is the quantity of bolts in double shear per bracket, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of all fasteners (Tables 4C). The slip-modulus of screw fasteners does not equal the slip modulus of bolt fastener: $N_s k_s \neq N_b k_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Table 4E). The discussion and calculations for slip-modulus are provided in Section 1.

Table 4D shows the uplift strength of the SW steel brackets based on the lateral (shear) strength of the steel-to-wood connection.

GOVERNING CODE:

National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

Allowable Lateral Strength of Screws	$Z'_{s,\,ASD}\,N_s = N_s\,Z\,C_D\,C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\,LRFD}\,N_s = \varphi\,N_s\,Z\,\lambda\,C_\Delta\,K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\,ASD}\,N_b = N_b\,Z\,C_D\,C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\,LRFD}\,N_b = \varphi\,N_b\,Z\,\lambda\,C_\Delta\,K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Allowable Uplift Strength of Steel-to-Wood Connection	$T_n/\Omega = \min\ [\, Z'_{s,\,ASD}\,(k_g/k_s)\ ,\ Z'_{b,\,ASD}\,(k_g/k_b)\,]$
Design Uplift Strength of Steel-to-Wood Connection	$\phi T_n = \min\ [\, Z'_{s,\,LRFD}\,(k_g/k_s)\ ,\ Z'_{b,\,LRFD}\,(k_g/k_b)\,]$

CALCULATIONS:

TABLE 4.2A: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A

			F_{yb}	199000	$1+R_e$	1.1	θ	0	
Screw Diameter (in)	D	0.243	$F_{em, par}$	5526	$2+R_e$	2.1	I_m	1260.2	
Screw Length (in)	L	3	$F_{em, perp}$	5526	k_1	0.408	I_s	1281.3	
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.549	II	522.7	
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	7.471	III_m	587.1	
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es, par}$	61800	III_s	409.7	
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.930	$F_{es, perp}$	61800	IV	522.8	
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800	D_r	0.243	
Lateral Design Value (lbs)	Z	410	LRFD resistance factor					ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor					λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor					K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s, ASD}$**	**655**	**LRFD Adjusted Lateral Design Value (lbs)**					**$Z'_{s, LRFD}$**	**884**

TABLE 4.2B: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A

Bolt Diameter (in)	D	0.5	$F_{em, par}$	6160	K_θ	1.000	I_m	3465	
Main Member Thickness (in)	$t_{m, min}$	4.5	$F_{em, perp}$	3626	$1+R_e$	1.071	III_s	2369	
Side Member Thickness (in)	t_s	0.25	F_{em}	6160	$2+R_e$	2.071	IV	3150	
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.071	k_3	10.192			
Bolt Yield Strength (psi)	F_{yb}	106000							
Max Angle Load to Grain (deg)	θ	0							
Specific Gravity	G	0.55							
Reference Lateral Design Value (Z)	Z	2369	LRFD resistance factor					ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor					λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor					K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b, ASD}$**	**3790**	**LRFD Adjusted Lateral Design Value (lbs)**					**$Z'_{b, LRFD}$**	**5112**

TABLE 4.2C: FASTENER SLIP-MODULUS

Model	k_s (lb/in)	k_b (lb/in)	N_s	N_b	k_g (lb/in)
SW 46	32143	95459	4	2	319491
SW 55	32143	95459	4	2	319491
SW 63	32143	95459	4	2	319491
SW 64	32143	95459	4	2	319491
SW 65	32143	95459	4	2	319491
SW 66	32143	95459	4	2	319491
SW 83	32143	95459	8	2	448063
SW 84	32143	95459	8	2	448063
SW 85	32143	95459	8	2	448063
SW 88	32143	95459	8	2	448063
SW 60*	32143	95459	4	2	319491
SW 80*	32143	95459	8	2	448063
SW 60C*	32143	n/a	12	0	385716

*Pair

TABLE 4.2D: TENSILE STRENGTH BASED ON STEEL-TO-WOOD CONNECTION

Model	$Z'_{s, ASD}$ (k_g/k_s) (lb)	$Z'_{b, ASD}$ (k_g/k_b) (lb)	$Z'_{s, LRFD}$ (k_g/k_s) (lb)	$Z'_{b, LRFD}$ (k_g/k_b) (lb)	**ASD T_n/Ω (lb)**	**LRFD ϕT_n (lb)**
SW 46	6515	12684	8787	17108	**6520**	**8790**
SW 55	6515	12684	8787	17108	**6520**	**8790**
SW 63	6515	12684	8787	17108	**6520**	**8790**
SW 64	6515	12684	8787	17108	**6520**	**8790**
SW 65	6515	12684	8787	17108	**6520**	**8790**
SW 66	6515	12684	8787	17108	**6520**	**8790**
SW 83	9137	17789	12324	23993	**9140**	**12320**
SW 84	9137	17789	12324	23993	**9140**	**12320**
SW 85	9137	17789	12324	23993	**9140**	**12320**
SW 88	9137	17789	12324	23993	**9140**	**12320**
SW 60*	6515	12684	8787	17108	**6520**	**8790**
SW 80*	9137	17789	12324	23993	**9140**	**12320**
SW 60C*	7866	n/a	10609	n/a	**7870**	**10610**

*Pair

TABLE 4.2E: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO

	Load Distribution		Load / Strength	
Model	Screws	Bolts	Screws	Bolts
SW 46	40.2%	59.8%	100%	51%
SW 55	40.2%	59.8%	100%	51%
SW 63	40.2%	59.8%	100%	51%
SW 64	40.2%	59.8%	100%	51%
SW 65	40.2%	59.8%	100%	51%
SW 66	40.2%	59.8%	100%	51%
SW 83	57.4%	42.6%	100%	51%
SW 84	57.4%	42.6%	100%	51%
SW 85	57.4%	42.6%	100%	51%
SW 88	57.4%	42.6%	100%	51%
SW 60*	40.2%	59.8%	100%	51%
SW 80*	57.4%	42.6%	100%	51%
SW 60C*	100.0%	0.0%	100%	0%

*Pair

5. STURDI-WALL: UPLIFT STRENGTH BASED ON TENSILE AND BENDING STRENGTH OF STEEL BRACKET

The calculations are presented in both the LRFD and ASD formats in accordance with the provisions of the governing code (AISC 360-16). The calculations are completed in Microsoft Excel (2016) using the listed equations. The distribution of internal moments in the steel bracket is determined using a two-dimensional computer model in Visual Analysis by IES, see footnotes for Table 4D.

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-1

GOVERNING EQUATIONS:

- **STEEL BRACKET: AISC 360, SECTION D2**

Design Tensile Strength	$\phi P_n = \phi F_y A_g$	**(tensile yielding)**	$\phi = 0.90$	(D2-1)
	$\phi P_n = \phi F_u A_e$	**(tensile rupture)**	$\phi = 0.75$	(D2-2)
Allowable Tensile Strength	$P_n / \Omega = F_y A_g / \Omega$	**(tensile yielding)**	$\Omega = 1.67$	(D2-1)
	$P_n / \Omega = F_u A_e / \Omega$	**(tensile rupture)**	$\Omega = 2.00$	(D2-2)

- **WELDS: AISC 360, SECTION J2**

Design Strength	$\phi R_n = \phi F_w A_w$	$\phi = 0.75$	(J2-3)
Allowable Strength	$R_n / \Omega = F_w A_w / \Omega$	$\Omega = 2.00$	(J2-3)
	$F_w = 0.60 F_{EXX}$		(T. J2.5)

- **BENDING: AISC 360, SECTIONS F1 & F11**

Design Bending Strength	$\phi M_n = \phi F_y Z$	$\phi = 0.90$	(F1, F11)
Allowable Bending Strength	$M_n / \Omega = M_n Z / \Omega$	$\Omega = 1.67$	(F1, F11)

CALCULATIONS:

STEEL BRACKET PROPERTIES		
Minimum Tensile Strength, F_u	55	ksi
Minimum Yield Strength, Fy	40	ksi
Thickness of steel, t	0.250	in

WELD PROPERTIES		
Fillet Weld Leg Size	0.25	in
Effective Weld Thickness (throat) , t_e	0.18	in
Electrode Classification Number	70	ksi
Nominal Strength of Weld Metal, F_w	42	ksi

TABLE 5A: DESIGN TENSILE STRENGTH AND ALLOWABLE TENSILE STRENGTH (WELDS AND VERTICAL STEEL PLATES)

	Strength of Welds			Tensile Strength of Vertical Plates					
				Yielding			Rupture		
Model ID	A_w (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)	A_g (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)	A_e (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)
SW 46	3.54	111353	74235	2.5	90000	59880	2.19	90338	60225
SW 55	3.54	111353	74235	2.5	90000	59880	2.19	90338	60225
SW 63	3.54	111353	74235	2.5	90000	59880	2.19	90338	60225
SW 64	3.54	111353	74235	2.5	90000	59880	2.19	90338	60225
SW 65	3.54	111353	74235	2.5	90000	59880	2.19	90338	60225
SW 66	3.54	111353	74235	2.5	90000	59880	2.19	90338	60225
SW 83	4.95	155894	103929	3.5	126000	83832	3.19	131588	87725
SW 84	4.95	155894	103929	3.5	126000	83832	3.19	131588	87725
SW 85	4.95	155894	103929	3.5	126000	83832	3.19	131588	87725
SW 88	4.95	155894	103929	3.5	126000	83832	3.19	131588	87725
SW 60*	n/a	n/a	n/a	2.5	90000	59880	2.19	90338	60225
SW 80*	n/a	n/a	n/a	3.5	126000	83832	3.19	131588	87725
SW 60C*	n/a	n/a	n/a	1.0	36000	23952	0.92	37950	25300

*Pair

TABLE 5B: DESIGN UPLIFT STRENGTH AND ALLOWABLE UPLIFT STRENGTH AS DEFINED BY THE BENDING STRENGTH OF THE STEEL PLATES (ANYWHERE)

Model ID	**t (in)**	**w (in)**	**F_y (ksi)**	**Z (in^3)**	**ϕM_n (in-lb)**	**M_n / Ω (in-lb)**	**k**	**LRFD ϕT_n (lb)**	**ASD T_n / Ω (lb)**
SW 46	0.250	5.00	40	0.078	2813	1871	0.6300	4460	2970
SW 55	0.250	5.00	40	0.078	2813	1871	0.5611	5010	3330
SW 63	0.250	5.00	40	0.078	2813	1871	0.5084	5530	3680
SW 64	0.250	5.00	40	0.078	2813	1871	0.5164	5450	3620
SW 65	0.250	5.00	40	0.078	2813	1871	0.5440	5170	3440
SW 66	0.250	5.00	40	0.078	2813	1871	0.5141	5470	3640
SW 83	0.250	7.00	40	0.109	3938	2620	0.5265	7480	4980
SW 84	0.250	7.00	40	0.109	3938	2620	0.5369	7330	4880
SW 85	0.250	7.00	40	0.109	3938	2620	0.5440	7240	4820
SW 88	0.250	7.00	40	0.109	3938	2620	0.5459	7210	4800
SW 60	0.250	5.00	40	0.078	2813	1871	0.5504	5110	3400
SW 80	0.250	7.00	40	0.109	3938	2620	0.5825	6760	4500
SW 60C	0.250	4.00	40	0.063	2250	1497	1.1510	1950	1300

(1) SW 60, SW 60C and SW 80 brackets are used in pairs

(2) t = thickness of steel plate

(3) w = width of steel plate

(3) Z is plastic section modulus = $w t^2 / 4$

(4) Factor "k" represents the maximum moment found anywhere in the steel bracket under 1 pound of tension force. This factor was determined using a two dimensional computer model for each SW model and equals to the maximum internal moment divided by the total applied downward force of 1 lb, k = M/F.

(5) Tension strength, as defined by the bending strength of the steel bracket, is determined using the following expressions$\phi T_n = \phi M_n/k$, $T_n / \Omega = (M_n/k) / \Omega$

6. STURDI-WALL: UPLIFT STRENGTH OF ANCHORS

The calculations are presented in both the LRFD and ASD formats according to provisions of the governing code (ACI 318-14). The uplift strength calculations consider the effects of prying forces SW base plate exerts on the anchors. As the bracket is loaded in uplift, the ends of the steel base plate are prevented from moving down by the concrete surface (Figure 6B). The uplift force on the anchors is the sum of the uplift force on the bracket from the wood column and the vertical reaction forces at the ends of the steel base plate: $R_{y1} + R_{y2} = T + P_1 + P_2$ (Figure 6A). The load on the anchors is calculated using the magnification factor of $k = (R_{y1} + R_{y2}) / T$, where T = 1 lb of force applied to a two-dimensional computer model in Visual Analysis by IES, and R_{y1} and R_{y2} are the reactions at the location of the anchors.

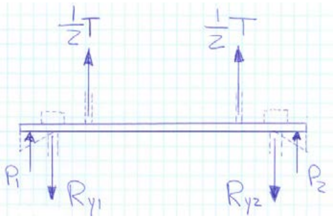


Figure 6A

Table 6A shows design and allowable uplift strength of the steel bracket based on tensile strength of anchor bolts. Table 6B shows design and allowable uplift strength of the steel bracket based on concrete breakout. It is desirable to present the results in terms of ASD and LRFD design, therefore, AISC 360, which include both methods, is used for steel anchor design calculations. The breakout calculations are completed using the LRFD method per ACI 318 and are converted to the ASD levels using the conversion factor, α = 1.6. The calculations are completed in Microsoft Excel (2016) using the listed equations.

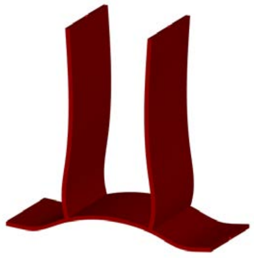


Figure 6B

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-16
Building Code Requirements for Structural Concrete ACI 318-14

GOVERNING EQUATIONS:

- **BOLTS (ANCHORS): AISC 360, SECTION J3**

Design Tensile Strength	$\phi R_n = \phi N_b F_{nt} A_b$	$\phi = 0.75$	(J3-1)
Allowable Tensilie Strength	$R_n / \Omega = N_b F_{nt} A_b / \Omega$	$\Omega = 2.00$	(J3-1)

N_b = number of bolts

- **ANCHORS (CONCRETE BREAKOUT): ACI 318, 17.4.2.1**

Design Strength	$\varphi N_{cbg} = \varphi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$	$\phi = 0.65$	(17.4.2.1b)
Allowable Strength	$N_{cbg} / \Omega = \varphi N_{cbg} / \alpha$	$\alpha = 1.6$	

CALCULATIONS:

ANCHOR PROPERTIES		
Nominal Tensile Stress, F_{nt}	45 (or greater)	ksi

CONCRETE PROPERTIES		
Compressive Strength, f'_c	3 (or greater)	ksi

TABLE 6A: UPLIFT STRENGTH OF SW BRACKET BASED ON TENSILE STRENGTH OF ANCHORS

		Tensile Strength of Anchors (A307)						Uplift Strength				
		1/2"	5/8"	1/2"		5/8"			1/2"		5/8"	
Model ID	N_b	A_b (in^2)	A_b (in^2)	LRFD ϕR_n (lbf)	ASD R_n / Ω (lbf)	LRFD ϕR_n (lbf)	ASD R_n / Ω (lbf)	k	LRFD ϕT_n (lbf)	ASD T_n / Ω (lbf)	LRFD ϕT_n (lbf)	ASD T_n / Ω (lbf)
SW 46	2	0.20	0.31	13254	8836	20709	13806	2.04	**6487**	**4325**	**10136**	**6758**
SW 55	2	0.20	0.31	13254	8836	20709	13806	1.94	**6849**	**4566**	**10702**	**7135**
SW 63	2	0.20	0.31	13254	8836	20709	13806	1.84	**7195**	**4797**	**11243**	**7495**
SW 64	2	0.20	0.31	13254	8836	20709	13806	1.86	**7145**	**4763**	**11164**	**7442**
SW 65	2	0.20	0.31	13254	8836	20709	13806	1.90	**6976**	**4650**	**10899**	**7266**
SW 66	2	0.20	0.31	13254	8836	20709	13806	1.85	**7160**	**4773**	**11188**	**7459**
SW 83	2	0.20	0.31	13254	8836	20709	13806	1.87	**7080**	**4720**	**11062**	**7375**
SW 84	2	0.20	0.31	13254	8836	20709	13806	1.89	**7016**	**4677**	**10963**	**7309**
SW 85	2	0.20	0.31	13254	8836	20709	13806	1.90	**6976**	**4650**	**10899**	**7266**
SW 88	2	0.20	0.31	13254	8836	20709	13806	1.90	**6976**	**4650**	**10899**	**7266**
SW 60*	2	0.20	0.31	13254	8836	20709	13806	1.91	**6935**	**4624**	**10837**	**7224**
SW 80*	2	0.20	0.31	13254	8836	20709	13806	1.96	**6748**	**4499**	**10544**	**7029**
SW 60C*	1	0.20	0.31	6627	4418	10354	6903	1.95	**3395**	**2263**	**5304**	**3536**

(1) SW 60 and SW 80 brackets to be used in pairs

(2) $\varphi T_n = \varphi R_n / k$

(3) $T_n / \Omega = (R_n / \Omega) / k$

(4) $k = (R_{y1} + R_{y2}) / T$, $T = 1$ lb, R_{y1} and R_{y2} are anchor reaction forces in Visual Analysis

TABLE 6B: UPLIFT (TENSILE) STRENGTH OF ANCHORS: CONCRETE BREAKOUT

8" CONCRETE WALL, ANCHORS CENTERED IN WALL

Model ID	c_{a1} (in)	S_1 (in)	A_{Nc} (in^2)	A_{Nco} (in^2)	Ψ	h_{ef} (in)	N_b (lb)	LRFD φN_{cbg} (lb)	ASD N_{cbg}/Ω (lb)	k	LRFD φT_n (lb)	ASD T_n/Ω (lb)
SW 46	4.0	9.375	178.2	166.4	1.0	4.3	8303	5779	3612	2.04	2829	1768
SW 55	4.0	9.375	178.2	166.4	1.0	4.3	8303	5779	3612	1.94	2987	1867
SW 63	4.0	9.375	178.2	166.4	1.0	4.3	8303	5779	3612	1.84	3137	1961
SW 64	4.0	10.875	190.2	166.4	1.0	4.3	8303	6168	3855	1.86	3325	2078
SW 65	4.0	12.375	199.2	166.4	1.0	4.3	8303	6460	4038	1.90	3400	2125
SW 66	4.0	10.875	190.2	166.4	1.0	4.3	8303	6168	3855	1.85	3332	2083
SW 83	4.0	9.375	178.2	166.4	1.0	4.3	8303	5779	3612	1.87	3087	1929
SW 84	4.0	10.875	190.2	166.4	1.0	4.3	8303	6168	3855	1.89	3265	2041
SW 85	4.0	12.375	199.2	166.4	1.0	4.3	8303	6460	4038	1.90	3400	2125
SW 88	4.0	12.875	199.2	166.4	1.0	4.3	8303	6460	4038	1.90	3400	2125
SW 60	4.0	9.375	178.2	166.4	1.0	4.3	8303	5779	3612	1.91	3024	1890
SW 80	4.0	9.375	178.2	166.4	1.0	4.3	8303	5779	3612	1.96	2942	1839
SW 60C	4.0	0	103.2	166.4	1.0	4.3	8303	3347	2092	1.95	1715	1072

10" CONCRETE WALL, ANCHORS CENTERED IN WALL

Model ID	c_{a1} (in)	S_1 (in)	A_{Nc} (in^2)	A_{Nco} (in^2)	Ψ	h_{ef} (in)	N_b (lb)	LRFD φN_{cbg} (lb)	ASD N_{cbg}/Ω (lb)	k	LRFD φT_n (lb)	ASD T_n/Ω (lb)
SW 46	5.0	9.375	222.8	166.4	1.0	4.3	8303	7224	4515	2.04	3536	2210
SW 55	5.0	9.375	222.8	166.4	1.0	4.3	8303	7224	4515	1.94	3733	2333
SW 63	5.0	9.375	222.8	166.4	1.0	4.3	8303	7224	4515	1.84	3922	2451
SW 64	5.0	10.875	237.8	166.4	1.0	4.3	8303	7710	4819	1.86	4156	2598
SW 65	5.0	12.375	252.8	166.4	1.0	4.3	8303	8197	5123	1.90	4314	2696
SW 66	5.0	10.875	237.8	166.4	1.0	4.3	8303	7710	4819	1.85	4165	2603
SW 83	5.0	9.375	222.8	166.4	1.0	4.3	8303	7224	4515	1.87	3859	2412
SW 84	5.0	10.875	237.8	166.4	1.0	4.3	8303	7710	4819	1.89	4082	2551
SW 85	5.0	12.375	252.8	166.4	1.0	4.3	8303	8197	5123	1.90	4314	2696
SW 88	5.0	12.875	257.8	166.4	1.0	4.3	8303	8359	5224	1.90	4399	2750
SW 60	5.0	9.375	222.8	166.4	1.0	4.3	8303	7224	4515	1.91	3780	2363
SW 80	5.0	9.375	222.8	166.4	1.0	4.3	8303	7224	4515	1.96	3678	2299
SW 60C	5.0	0	129.0	166.4	1.0	4.3	8303	4183	2615	1.95	2143	1339

12" CONCRETE WALL, ANCHORS CENTERED IN WALL

Model ID	c_{a1} (in)	S_1 (in)	A_{Nc} (in^2)	A_{Nco} (in^2)	Ψ	h_{ef} (in)	N_b (lb)	LRFD φN_{cbg} (lb)	ASD N_{cbg}/Ω (lb)	k	LRFD φT_n (lb)	ASD T_n/Ω (lb)
SW 46	6.0	9.375	267.3	166.4	1.0	4.3	8303	8669	5418	2.04	4243	2652
SW 55	6.0	9.375	267.3	166.4	1.0	4.3	8303	8669	5418	1.94	4480	2800
SW 63	6.0	9.375	267.3	166.4	1.0	4.3	8303	8669	5418	1.84	4706	2941
SW 64	6.0	10.875	285.3	166.4	1.0	4.3	8303	9252	5783	1.86	4988	3117
SW 65	6.0	12.375	303.3	166.4	1.0	4.3	8303	9836	6148	1.90	5177	3236
SW 66	6.0	10.875	285.3	166.4	1.0	4.3	8303	9252	5783	1.85	4999	3124
SW 83	6.0	9.375	267.3	166.4	1.0	4.3	8303	8669	5418	1.87	4631	2894
SW 84	6.0	10.875	285.3	166.4	1.0	4.3	8303	9252	5783	1.89	4898	3061
SW 85	6.0	12.375	303.3	166.4	1.0	4.3	8303	9836	6148	1.90	5177	3236
SW 88	6.0	12.875	309.3	166.4	1.0	4.3	8303	10031	6269	1.90	5279	3300
SW 60	6.0	9.375	267.3	166.4	1.0	4.3	8303	8669	5418	1.91	4536	2835
SW 80	6.0	9.375	267.3	166.4	1.0	4.3	8303	8669	5418	1.96	4414	2759
SW 60C	6.0	0	154.8	166.4	1.0	4.3	8303	5020	3138	1.95	2572	1607

EDGE DISTANCE OF 7.5" AND GREATER (EACH SIDE OF WALL)

Model ID	c_{a1} (in)	S_1 (in)	A_{Nc} (in^2)	A_{Nco} (in^2)	Ψ	h_{ef} (in)	N_b (lb)	LRFD φN_{cbg} (lb)	ASD N_{cbg}/Ω (lb)	k	LRFD φT_n (lb)	ASD T_n/Ω (lb)
SW 46	7.5	9.375	332.8	166.4	1.0	4.3	8303	10793	6746	2.04	5283	3302
SW 55	7.5	9.375	332.8	166.4	1.0	4.3	8303	10793	6746	1.94	5578	3486
SW 63	7.5	9.375	332.8	166.4	1.0	4.3	8303	10793	6746	1.84	5860	3662
SW 64	7.5	10.875	332.8	166.4	1.0	4.3	8303	10793	6746	1.86	5819	3637
SW 65	7.5	12.375	332.8	166.4	1.0	4.3	8303	10793	6746	1.90	5681	3550
SW 66	7.5	10.875	332.8	166.4	1.0	4.3	8303	10793	6746	1.85	5831	3644
SW 83	7.5	9.375	332.8	166.4	1.0	4.3	8303	10793	6746	1.87	5766	3604
SW 84	7.5	10.875	332.8	166.4	1.0	4.3	8303	10793	6746	1.89	5714	3571
SW 85	7.5	12.375	332.8	166.4	1.0	4.3	8303	10793	6746	1.90	5681	3550
SW 88	7.5	12.875	332.8	166.4	1.0	4.3	8303	10793	6746	1.90	5681	3550
SW 60	7.5	9.375	332.8	166.4	1.0	4.3	8303	10793	6746	1.91	5648	3530
SW 80	7.5	9.375	332.8	166.4	1.0	4.3	8303	10793	6746	1.96	5496	3435
SW 60C	7.5	0	166.4	166.4	1.0	4.3	8303	5397	3373	1.95	2765	1728

(1) SW 60, SW 60C and SW 80 angles are to be used in pairs
(2) c_{a1} = distance from center of anchor to edge of concrete, assume the anchors are installed in center of wall
(3) S_1 = spacing between anchors
(4) $A_{Nc} = \min [(c_{a1} + c_{a1})(1.5h_{ef} + S_1 + 1.5h_{ef}), N_b A_{Nco}]$
(5) $A_{Nco} = 9 h_{ef}^2$
(6) $\Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} = 1.0$
(7) h_{ef} is based on 5.75 in anchor embedment into the concrete, $h_{ef} = (3/4)5.75$
(8) $N_b = k_c \sqrt{(f'_c)}\, h_{ef}^{1.5}$, $k_c = 17$
(9) $k = (R_{y1} + R_{y2}) / T$, T = 1 lb, R_{y1} and R_{y2} are anchor reaction forces in Visual Analysis

7. STURDI-WALL PLUS: ROTATIONAL STIFFNESS

The effective rotational stiffness of the SWP steel bracket consists of three parts, three rotational springs arranged in series:

(1) $(M/\theta)_f$, the rotational stiffness of the steel-to-wood connection (slip-modulus of the dowel fasteners)

(2) $(M/\theta)_s$, the rotational stiffness of the steel saddle (3d finite element analysis in a structural design computer program)

(3) $(M/\theta)_r$, the rotational stiffness resulting from the axial deformation in the tension rebar

Each SWP steel bracket is fastened to wood column with 0.242"x3" structural screws and 1/2" SAE J429 Grade 5 through bolts. There are two fastener groups, the top fastener group and the bottom fastener group. The centroids of the fastener groups are separated by the distance "s" (Table 7A). To calculate the rotational stiffness of the steel-to-wood connection, it is necessary to first determine the slip-modulus for the 0.242" structural screw and the 1/2" through bolt. Per the *Wood Handbook (FPL, 2010, United States Department of Agriculture Forest Service)* the fastener slip-modulus for dowels loaded in single shear in steel-to-wood application can be calculated using the following expression: $k = 270{,}000\ D^{1.5}$, where k is the slip-modulus and D is the fastener diameter. The slip modulus equation, however, does not include slippage due to fastener-hole clearance: a fastener has the freedom to move laterally with respect to the steel plate until it comes in contact with the edge of the hole in the steel plate. The holes for the screws and the bolts are 5/16" and 5/8" respectively. If the fasteners are installed precisely through the center of the holes in the steel plate, the clearance on either side the screw and the bolt is approximately 1/32" and 1/16", respectively. The screws will be engaged and start transferring load before the bolt may come in contact with the edge of the hole in the steel plate. For this reason, the slip-modulus of the bolt is reduced proportionally to the ratio of clearances: (1/32") / (1/16") = 0.5, or 50%. The slip-modulus for each fastener group and the resulting rotational stiffness for each model is shown in Table 7B.

The rotational stiffness of the steel bracket below the top of the concrete pier is attributed mostly to the axial deformation of the tension rebar. Since the axial forces in the rebar are linearly decreasing from maximum to zero along the rebar development length L_d, or rebar length of 18", whichever is less, the effective length used in calculating axial rebar stiffness is equal to the lesser of L_d /2 or 9". The rotational stiffness of the steel saddle, $(M/\theta)_s$, and the rebar $(M/\theta)_r$, is analyzed jointly using a finite element analysis in Visual Analysis by IES, and the effective rotational stiffness is designated as $(M/\theta)_{s,r}$. Figure 7 shows a sketch of the finite element analysis model with supports. All springs have infinite stiffness and are set to only provide resistance to compression forces. The 1000 in-lb moment is applied to all models via the F_T and F_B forces, which are equal in magnitude and opposite in direction, $F_T = F_B = M/s$. The horizontal displacement Δ at force F_T (top fastener group) is divided by y to obtain the angle of rotation, θ, in radians, $\theta = \Delta / y$. The rotational stiffness $(M/\theta)_{s,r} = M / \theta$. The results of the analysis for each model are summarized in Table 7C.

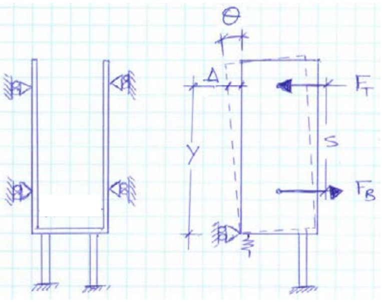


Figure 7

The effective rotational stiffness for each model, consisting of the steel-to-wood element and, the steel saddle and the rebar, is shown in Table 7D. The calculations are completed in Visual Analysis by IES and Microsoft Excel (2016) using the listed equations.

GOVERNING EQUATIONS:

Effective Rotational Stiffness	$(M/\theta)_e = [1 / (M/\theta)_f + 1 / (M/\theta)_{s,r}]^{-1}$	
Rotational Stiffness of Steel-to-Wood	$(M/\theta)_f = k\ s^2 / 2$	
Rotational Stiffness of Saddle and Rebar	$(M/\theta)_{s,r}$ **= determined from finite element analysis**	
Slip Modulus for (1) screw, single shear	$k_s = 270{,}000\ D_s^{1.5}$	FPL, Chapter 8
Slip Modulus for (1) bolt, double shear	$k_b = 0.5\ [\ 2(270{,}000)\ D_b^{1.5}\ ]$	(see discussion above)
Slip Modulus for a Fastener Group	$k_g = N_s k_s + N_b k_b$	
Rebar Development Length	$L_d = [(3/40)(f_y/\sqrt{f'_c})\ (\Psi_t\ \Psi_e\ \Psi_s) / c_b\ ]\ d_b^2$	(ACI 318-14, Eq. 25.4.2.3a)

s = distance between the centroids of the top and bottom fastener groups
N_s = quantity of screws in one fastener group
N_b = quantity of bolts in one fastener group
D_s = screw diameter
D_b = bolt diameter

CALCULATIONS:

TABLE 7A: LOCATION OF AND DISTANCE BETWEEN THE CENTROIDS OF THE TOP AND BOTTOM FASTENER GROUPS

Model	k_s (lb/in)	k_b (lb/in)	Elevation (in) Base	Bolt 1	Screw 1	Screw 2	Screw 3	Screw 4	Bolt 2	Bottom	Top	**s (in)**
SWP 46	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	3.627	11.87	**8.25**
SWP 63	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	3.627	11.87	**8.25**
SWP 64	32143	95459	0	3.375	4.375	n/a	n/a	16.125	17.125	3.627	16.87	**13.25**
SWP 66	32143	95459	0	3.375	4.375	n/a	n/a	11.125	12.125	3.627	11.87	**8.25**
SWP 83	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	4.680	16.32	**11.64**
SWP 84	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	4.680	16.32	**11.64**
SWP 85	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	4.680	16.32	**11.64**
SWP 88	32143	95459	0	3.875	4.875	6.875	14.125	16.125	17.125	4.680	16.32	**11.64**

TABLE 7B: ROTATIONAL STIFFNESS OF STEEL-TO-WOOD CONNECTION, $(M/\theta)_f$

Model	D_s (in)	D_b (in)	k_s (lb/in)	k_b (lb/in)	N_s	N_b	k_g (lb/in)	s (in)	**$(M/\theta)_f$ (in-kip/rad)**	**$(M/\theta)_f$ (in-kip/deg)**
SWP 46	0.242	0.50	32143	95459	2	1	159745	8.25	**5,000**	**94.8**
SWP 63	0.242	0.50	32143	95459	2	1	159745	8.25	**5,000**	**94.8**
SWP 64	0.242	0.50	32143	95459	2	1	159745	13.25	**14,000**	**245**
SWP 66	0.242	0.50	32143	95459	2	1	159745	8.25	**5,000**	**94.8**
SWP 83	0.242	0.50	32143	95459	4	1	224032	11.64	**15,000**	**265**
SWP 84	0.242	0.50	32143	95459	4	1	224032	11.64	**15,000**	**265**
SWP 85	0.242	0.50	32143	95459	4	1	224032	11.64	**15,000**	**265**
SWP 88	0.242	0.50	32143	95459	4	1	224032	11.64	**15,000**	**265**

TABLE 7C: ROTATIONAL STIFFNESS OF THE STEEL SADDLE AND REBAR, $(M/\theta)_{s,r}$

Model	L_d / 2* (in)	M (in-lb)	θ (rad)	**$(M/\theta)_{s,r}$ (in-kip/rad)**	**$(M/\theta)_{s,r}$ (in-kip/deg)**
SWP 46	9.0	1000	0.000317	**3150**	**55.1**
SWP 63	9.0	1000	0.000317	**3150**	**55.1**
SWP 64	5.1	1000	0.000313	**3150**	**55.8**
SWP 66	7.3	1000	0.000306	**3250**	**57.0**
SWP 83	9.0	1000	0.000144	**6900**	**121**
SWP 84	8.2	1000	0.000153	**6500**	**114**
SWP 85	7.7	1000	0.000157	**6350**	**111**
SWP 88	7.7	1000	0.000158	**6300**	**110**

* see Table 4C

TABLE 7D: EFFECTIVE ROTATIONAL STIFFNESS OF SWP, $(M/\theta)_e$

Model	**$(M/\theta)_e$ (in-kip/rad)**	**$(M/\theta)_e$ (in-kip/deg)**
SWP 46	**1900**	**34.8**
SWP 63	**1900**	**34.8**
SWP 64	**2550**	**45.4**
SWP 66	**1950**	**35.6**
SWP 83	**4700**	**83.2**
SWP 84	**4500**	**79.7**
SWP 85	**4450**	**78.3**
SWP 88	**4400**	**78.0**

8.1 STURDI-WALL PLUS: BENDING AND SHEAR STRENGTH OF STEEL-TO-WOOD CONNECTION*

***WITH SDS SCREWS BY SIMPSON STRONG TIE**

The shear and bending forces are transferred from the wood column into the steel bracket via 0.242"x3" structural screws and 1/2" through bolts. The calculations below are for wood columns with specific gravity, SG, of 0.55 and higher. The calculations assume a rotationally rigid concrete foundation to ensure that moment reversal (location of zero moment) occurs above the bracket, not below (Figure 8.1A). The distance between the centroids of the top and bottom fastener groups, s, and the distance from the bottom of column to the centroid of the bottom fastener group, a, are specified in tables below. SWP brackets are designed to transfer shear, V and bending moment, M, forces as measured at the bottom of the bracket. There are four load cases to consider, see Figure 8.1B. Load Case 1 defines maximum shear strength, V_{max}, of the column-to-bracket connection in absence of moment forces. Load Case 2 defines the maximum moment strength, M_{max}, of the column-to-bracket connection in absence of shear forces. Load Case 3 is a combination of Load Case 1 and Load Case 2 where a maximum moment and a maximum shear force are applied to the bracket simultaneously. In all load cases, maximum shear strength V_{max}, and maximum moment strength, M_{max}, are defined such that the magnitude of the resulting forces F_T (force at the topo fastener group) and F_B (force at the bottom fastener group) does not exceed the latearal strength of each respective fastener group.

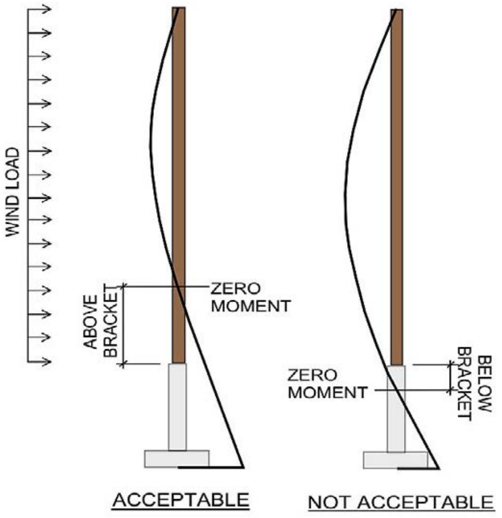
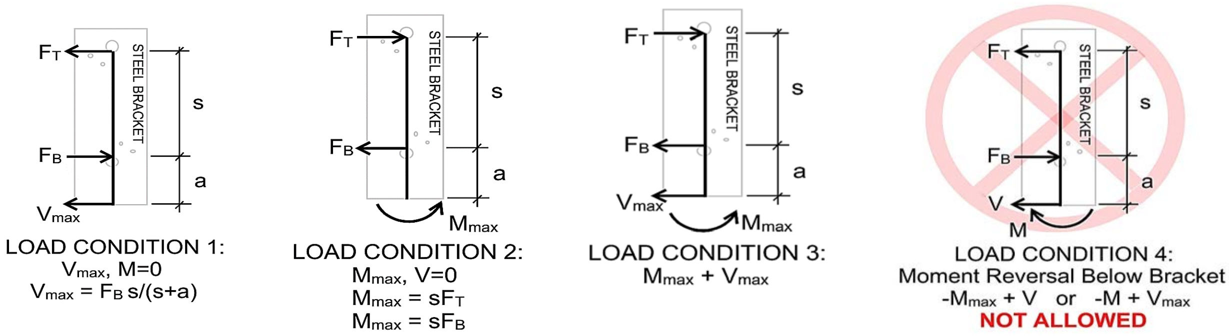


FIGURE 8.1A

The resulting forces F_T and F_B in Load Case 1 are acting in opposite direction from the resulting forces F_T and F_B in Load Case 2. This means that adding a shear load to the connection that is loaded with the maximum moment force will result in reduction in forces F_T and F_B. Similarly, adding a moment force to the connection that is loaded with the maximum shear force will result in reduction in forces F_T and F_B. Therefore, V_{max} and M_{max} loading may be applied to the bracket simultaneously without any reduction in strength. Load Case 4 represents the condition in which the moment reversal occurs below the bracket. In this load condition, M_{max}, as determined by Load Condition 2, cannot be used in combination with a shear force of any magnitude and V_{max}, as determined by Load Condition 1, cannot be used in combination with moment force of any magnitude. As shear force increases moment strength decreases, and as moment force increases shear strength decreases. This condition is rare and should not occur when foundation is properly designed.



LOAD CONDITION 1:
V_{max}, M=0
$V_{max} = F_B\, s/(s+a)$

LOAD CONDITION 2:
M_{max}, V=0
$M_{max} = sF_T$
$M_{max} = sF_B$

LOAD CONDITION 3:
$M_{max} + V_{max}$

LOAD CONDITION 4:
Moment Reversal Below Bracket
$-M_{max} + V$ or $-M + V_{max}$
NOT ALLOWED

FIGURE 8.1B

The load on each fastener type (screw, bolt) within the fastener group is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus of the entire fastener group: N_sk_s/k_g, N_bk_b/k_g, where N_s is the quantity of screws within the fastener group, N_b is the quantity of bolts in double shear within the fastener group, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of the entire fastener group (Table 7.1B). The slip-modulus of the screw fasteners does not equal the slip modulus of the bolt fastener(s): $N_sk_s \neq N_bk_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Tables 8.1C and 8.1D).

The allowable bending and shear strength (ASD) and the design bending strength and shear strength (LRFD) of the steel-to-wood connection for each model is shown in Table 8.1E. The calculations are completed in Microsoft Excel (2016) using the listed equations.

GOVERNING CODE:

National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

Allowable Lateral Strength of Screws	$Z'_{s,\ ASD} N_s = N_s Z C_D C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\ LRFD} N_s = \varphi N_s Z \lambda C_\Delta K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\ ASD} N_b = N_b Z C_D C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\ LRFD} N_b = \varphi N_b Z \lambda C_\Delta K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Allowable Lateral Strength of Fastener Group	$V_n/\Omega = \min\ [\ Z'_{s,\ ASD} (k_g/k_s)\ ,\ Z'_{b,\ ASD} (k_g/k_b)\ ]$
Design Lateral Strength of Fastener Group	$\varphi V_n = \min\ [\ Z'_{s,\ LRFD} (k_g/k_s)\ ,\ Z'_{b,\ LRFD} (k_g/k_b)\ ]$

Allowable Bending Strength of Connection	$M_n/\Omega = s\ NZ'_{ASD}$
Design Bending Strength of Connection	$\phi M_n = s\ NZ'_{LRFD}$
Allowable Shear Strength of Connection	$V_n/\Omega = NZ'_{ASD}\ s/(s+a)$
Design Shear Strength of Connection	$\phi V_n = NZ'_{LRFD}\ s/(s+a)$

s = distance between the centroids of the fastener groups

a = distance from bottom of bracket to centroid of the bottom fastener group

CALCULATIONS:

TABLE 8.1A: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A (Yield Limit Equations)

			F_{yb}	164000	$1{+}R_e$	1.1	θ	90
Screw Diameter (in)	D	0.242	$F_{em,\ par}$	5526	$2{+}R_e$	2.1	I_m	1259.3
Screw Length (in)	L	3	$F_{em,\ perp}$	5526	k_1	0.408	I_s	1280.4
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.536	II	522.4
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	6.944	III_m	572.7
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es,\ par}$	61800	III_s	380.5
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.920	$F_{es,\ perp}$	61800	IV	472.3
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800	D_r	0.242
Lateral Design Value (lbs)	Z	380	LRFD resistance factor		ϕ	0.65		
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor		λ	1		
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor		K_F	3.32		
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s,\ ASD}$**	**609**	**LRFD Adjusted Lateral Design Value (lbs)**		**$Z'_{s,\ LRFD}$**	**821**		

TABLE 8.1B: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A (Yield Limit Equations)

Bolt Diameter (in)	D	0.5	$F_{em,\ par}$	6160	K_θ	1.250	I_m	1631
Main Member Thickness (in)	$t_{m,\ min}$	4.5	$F_{em,\ perp}$	3626	$1{+}R_e$	1.042	III_s	1494
Side Member Thickness (in)	t_s	0.25	F_{em}	3626	$2{+}R_e$	2.042	IV	1960
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.042	k_3	13.463		
Bolt Yield Strength (psi)	F_{yb}	106000						
Max Angle Load to Grain (deg)	θ	90						
Specific Gravity	G	0.55						
Reference Lateral Design Value (Z)	Z	1494	LRFD resistance factor		ϕ	0.65		
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor		λ	1		
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor		K_F	3.32		
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b,\ ASD}$**	**2391**	**LRFD Adjusted Lateral Design Value (lbs)**		**$Z'_{b,\ LRFD}$**	**3224**		

TABLE 8.1C: LATERAL (SHEAR) STRENGTH OF EACH FASTENER GROUP

Model	k_s (lb/in)	k_b (lb/in)	k_g (lb/in)	$Z'_{s,\ LRFD}$ (k_g/k_s) (lb)	$Z'_{b,\ LRFD}$ (k_g/k_b) (lb)	$Z'_{s,\ ASD}$ (k_g/k_s) (lb)	$Z'_{b,\ ASD}$ (k_g/k_b) (lb)	**LRFD φV (lb)**	**ASD V_n/Ω (lb)**
SWP 46	32143	95459	159745	4081	5396	3026	4001	**4081**	**3026**
SWP 63	32143	95459	159745	4081	5396	3026	4001	**4081**	**3026**
SWP 64	32143	95459	159745	4081	5396	3026	4001	**4081**	**3026**
SWP 66	32143	95459	159745	4081	5396	3026	4001	**4081**	**3026**
SWP 83	32143	95459	224032	5723	7567	4243	5611	**5723**	**4243**
SWP 84	32143	95459	224032	5723	7567	4243	5611	**5723**	**4243**
SWP 85	32143	95459	224032	5723	7567	4243	5611	**5723**	**4243**
SWP 88	32143	95459	224032	5723	7567	4243	5611	**5723**	**4243**

TABLE 8.1D: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO

Model	N_s	N_b	Load Distribution Screws	Load Distribution Bolts	Load / Strength Screws	Load / Strength Bolts
SWP 46	2	1	40.2%	59.8%	100.0%	76%
SWP 63	2	1	40.2%	59.8%	100.0%	76%
SWP 64	2	1	40.2%	59.8%	100.0%	76%
SWP 66	2	1	40.2%	59.8%	100.0%	76%
SWP 83	4	1	57.4%	42.6%	100.0%	76%
SWP 84	4	1	57.4%	42.6%	100.0%	76%
SWP 85	4	1	57.4%	42.6%	100.0%	76%
SWP 88	4	1	57.4%	42.6%	100.0%	76%

TABLE 8.1E: SHEAR AND BENDING STRENGTH OF STEEL-TO-WOOD CONNECTION

			LRFD		**ASD**	
Model	a (in)	s (in)	**ϕV_n (lb)**	**ϕM_n (lb-in)**	**V_n/Ω (lb)**	**M_n/Ω (lb-in)**
SWP 46	3.63	8.25	**2830**	**33670**	**2100**	**24960**
SWP 63	3.63	8.25	**2830**	**33670**	**2100**	**24960**
SWP 64	3.63	13.25	**3200**	**54070**	**2380**	**40090**
SWP 66	3.63	8.25	**2830**	**33670**	**2100**	**24960**
SWP 83	4.68	11.65	**4080**	**66670**	**3030**	**49430**
SWP 84	4.68	11.65	**4080**	**66670**	**3030**	**49430**
SWP 85	4.68	11.65	**4080**	**66670**	**3030**	**49430**
SWP 88	4.68	11.65	**4080**	**66670**	**3030**	**49430**

8.2 STURDI-WALL PLUS: BENDING AND SHEAR STRENGTH OF STEEL-TO-WOOD CONNECTION*

***WITH PROPRIATARY SCREWS**

The shear and bending forces are transferred from the wood column into the steel bracket via 0.242"x3" structural screws and 1/2" through bolts. The calculations below are for wood columns with specific gravity, SG, of 0.55 and higher. The calculations assume a rotationally rigid concrete foundation to ensure that moment reversal (location of zero moment) occurs above the bracket, not below (Figure 8.2A). The distance between the centroids of the top and bottom fastener groups, s, and the distance from the bottom of column to the centroid of the bottom fastener group, a, are specified in tables below. SWP brackets are designed to transfer shear, V and bending moment, M, forces as measured at the bottom of the bracket. There are four load cases to consider, see Figure 8.2B. Load Case 1 defines maximum shear strength, V_{max}, of the column-to-bracket connection in absence of moment forces. Load Case 2 defines the maximum moment strength, M_{max}, of the column-to-bracket connection in absence of shear forces. Load Case 3 is a combination of Load Case 1 and Load Case 2 where a maximum moment and a maximum shear force are applied to the bracket simultaneously. In all load cases, maximum shear strength V_{max}, and maximum moment strength, M_{max}, are defined such that the magnitude of the resulting forces F_T (force at the topo fastener group) and F_B (force at the bottom fastener group) does not exceed the latearal strength of each respective fastener group.

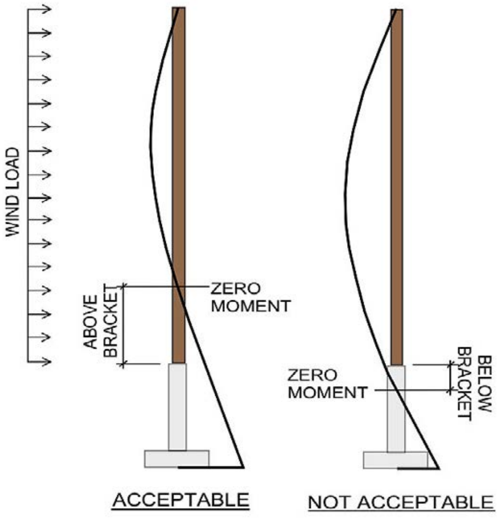
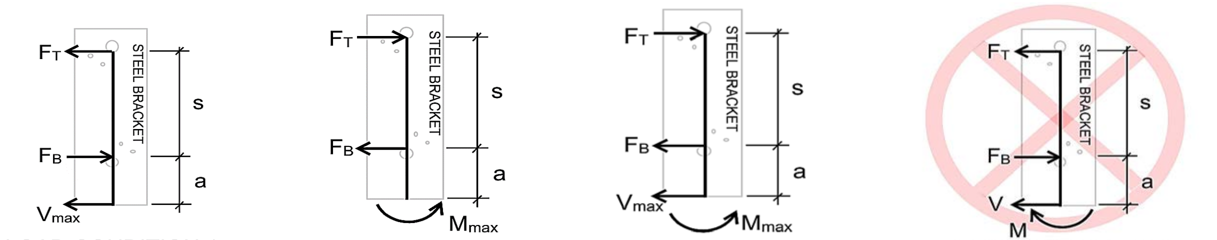


FIGURE 8.2A

The resulting forces F_T and F_B in Load Case 1 are acting in opposite direction from the resulting forces F_T and F_B in Load Case 2. This means that adding a shear load to the connection that is loaded with the maximum moment force will result in reduction in forces F_T and F_B. Similarly, adding a moment force to the connection that is loaded with the maximum shear force will result in reduction in forces F_T and F_B. Therefore, V_{max} and M_{max} loading may be applied to the bracket simultaneously without any reduction in strength. Load Case 4 represents the condition in which the moment reversal occurs below the bracket. In this load condition, M_{max}, as determined by Load Condition 2, cannot be used in combination with a shear force of any magnitude and V_{max}, as determined by Load Condition 1, cannot be used in combination with moment force of any magnitude. As shear force increases moment strength decreases, and as moment force increases shear strength decreases. This condition is rare and should not occur when foundation is properly designed.



LOAD CONDITION 1:
V_{max}, M=0
$V_{max} = F_B\, s/(s+a)$

LOAD CONDITION 2:
M_{max}, V=0
$M_{max} = sF_T$
$M_{max} = sF_B$

LOAD CONDITION 3:
$M_{max} + V_{max}$

LOAD CONDITION 4:
Moment Reversal Below Bracket
$-M_{max} + V$ or $-M + V_{max}$
NOT ALLOWED

FIGURE 8.2B

The load on each fastener type (screw, bolt) within the fastener group is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus of the entire fastener group: $N_s k_s/k_g$, $N_b k_b/k_g$, where N_s is the quantity of screws within the fastener group, N_b is the quantity of bolts in double shear within the fastener group, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of the entire fastener group (Table 7.1B). The slip-modulus of the screw fasteners does not equal the slip modulus of the bolt fastener(s): $N_s k_s \neq N_b k_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Tables 8.2C and 8.2D).

The allowable bending and shear strength (ASD) and the design bending strength and shear strength (LRFD) of the steel-to-wood connection for each model is shown in Table 8.2E. The calculations are completed in Microsoft Excel (2016) using the listed equations.

GOVERNING CODE:

National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

Allowable Lateral Strength of Screws	$Z'_{s,\ ASD}\ N_s = N_s\ Z\ C_D\ C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\ LRFD}\ N_s = \varphi\ N_s\ Z\ \lambda\ C_\Delta\ K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\ ASD}\ N_b = N_b\ Z\ C_D\ C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\ LRFD}\ N_b = \varphi\ N_b\ Z\ \lambda\ C_\Delta\ K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Allowable Lateral Strength of Fastener Group	$V_n/\Omega = \min\ [\ Z'_{s,\ ASD}\ (k_g/k_s)\ ,\ Z'_{b,\ ASD}\ (k_g/k_b)\ ]$
Design Lateral Strength of Fastener Group	$\varphi V_n = \min\ [\ Z'_{s,\ LRFD}\ (k_g/k_s)\ ,\ Z'_{b,\ LRFD}\ (k_g/k_b)\ ]$

Allowable Bending Strength of Connection	$M_n/\Omega = s\ NZ'_{ASD}$
Design Bending Strength of Connection	$\phi M_n = s\ NZ'_{LRFD}$
Allowable Shear Strength of Connection	$V_n/\Omega = NZ'_{ASD}\ s/(s+a)$
Design Shear Strength of Connection	$\phi V_n = NZ'_{LRFD}\ s/(s+a)$

s = distance between the centroids of the fastener groups
a = distance from bottom of bracket to centroid of the bottom fastener group

CALCULATIONS:

TABLE 8.2A: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A (Yield Limit Equations)

			F_{yb}	199000	$1+R_e$	1.1	θ	90
Screw Diameter (in)	D	0.243	$F_{em, par}$	5526	$2+R_e$	2.1	I_m	1260.2
Screw Length (in)	L	3	$F_{em, perp}$	5526	k_1	0.408	I_s	1281.3
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.549	II	522.7
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	7.471	III_m	587.1
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es, par}$	61800	III_s	409.7
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.930	$F_{es, perp}$	61800	IV	522.8
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800	D_r	0.243
Lateral Design Value (lbs)	Z	410	LRFD resistance factor		ϕ	0.65		
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor		λ	1		
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor		K_F	3.32		
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s, ASD}$**	**655**	**LRFD Adjusted Lateral Design Value (lbs)**		**$Z'_{s, LRFD}$**	**884**		

TABLE 8.2B: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A (Yield Limit Equations)

Bolt Diameter (in)	D	0.5	$F_{em, par}$	6160	K_θ	1.250	I_m	1631
Main Member Thickness (in)	$t_{m, min}$	4.5	$F_{em, perp}$	3626	$1+R_e$	1.042	III_s	1494
Side Member Thickness (in)	t_s	0.25	F_{em}	3626	$2+R_e$	2.042	IV	1960
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.042	k_3	13.463		
Bolt Yield Strength (psi)	F_{yb}	106000						
Max Angle Load to Grain (deg)	θ	90						
Specific Gravity	G	0.55						
Reference Lateral Design Value (Z)	Z	1494	LRFD resistance factor		ϕ	0.65		
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor		λ	1		
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor		K_F	3.32		
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b, ASD}$**	**2391**	**LRFD Adjusted Lateral Design Value (lbs)**		**$Z'_{b, LRFD}$**	**3224**		

TABLE 8.2C: LATERAL (SHEAR) STRENGTH OF EACH FASTENER GROUP

Model	k_s (lb/in)	k_b (lb/in)	k_g (lb/in)	$Z'_{s, LRFD}$ (k_g/k_s) (lb)	$Z'_{b, LRFD}$ (k_g/k_b) (lb)	$Z'_{s, ASD}$ (k_g/k_s) (lb)	$Z'_{b, ASD}$ (k_g/k_b) (lb)	**LRFD φV (lb)**	**ASD V_n/Ω (lb)**
SWP 46	32143	95459	159745	4394	5396	3258	4001	**4394**	**3258**
SWP 63	32143	95459	159745	4394	5396	3258	4001	**4394**	**3258**
SWP 64	32143	95459	159745	4394	5396	3258	4001	**4394**	**3258**
SWP 66	32143	95459	159745	4394	5396	3258	4001	**4394**	**3258**
SWP 83	32143	95459	224032	6162	7567	4569	5611	**6162**	**4569**
SWP 84	32143	95459	224032	6162	7567	4569	5611	**6162**	**4569**
SWP 85	32143	95459	224032	6162	7567	4569	5611	**6162**	**4569**
SWP 88	32143	95459	224032	6162	7567	4569	5611	**6162**	**4569**

TABLE 8.2D: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO						
	N_s	N_b	Load Distribution		Load / Strength	
Model			Screws	Bolts	Screws	Bolts
SWP 46	2	1	40.2%	59.8%	100.0%	81%
SWP 63	2	1	40.2%	59.8%	100.0%	81%
SWP 64	2	1	40.2%	59.8%	100.0%	81%
SWP 66	2	1	40.2%	59.8%	100.0%	81%
SWP 83	4	1	57.4%	42.6%	100.0%	81%
SWP 84	4	1	57.4%	42.6%	100.0%	81%
SWP 85	4	1	57.4%	42.6%	100.0%	81%
SWP 88	4	1	57.4%	42.6%	100.0%	81%

TABLE 8.2E: SHEAR AND BENDING STRENGTH OF STEEL-TO-WOOD CONNECTION						
			LRFD		**ASD**	
Model	a (in)	s (in)	**ϕV_n (lb)**	**ϕM_n (lb-in)**	**V_n/Ω (lb)**	**M_n/Ω (lb-in)**
SWP 46	3.63	8.25	**3050**	**36250**	**2260**	**26880**
SWP 63	3.63	8.25	**3050**	**36250**	**2260**	**26880**
SWP 64	3.63	13.25	**3450**	**58220**	**2560**	**43160**
SWP 66	3.63	8.25	**3050**	**36250**	**2260**	**26880**
SWP 83	4.68	11.65	**4400**	**71780**	**3260**	**53220**
SWP 84	4.68	11.65	**4400**	**71780**	**3260**	**53220**
SWP 85	4.68	11.65	**4400**	**71780**	**3260**	**53220**
SWP 88	4.68	11.65	**4400**	**71780**	**3260**	**53220**

9. STURDI-WALL PLUS: BENDING STRENGTH OF STEEL BRACKET (SADDLE AND REBAR)

The bending strength calculations for the Sturdi-Wall Plus bracket (saddle and rebar) are presented in both the LRFD and ASD formats in accordance with the provisions of the governing code (AISC 360-16). The calculations for the rebar development into the concrete pier are prepared using ACI 318-14. The calculations are completed using the finite element analysis in Visual Analysis by IES and Microsoft Excel (2016) using the listed equations.

In Visual Analysis, a 1000 lb-in moment is applied to each model in the form of horizontal forces, F_T and F_B, equal in magnitude and opposite in direction, applied at the centroid of the top and bottom fastener group, respectively. The restraint conditions for the finite element analysis models are described in Section 7. The resulting maximum internal bending moment, M_{max}, in units of (lb-in)/in, located anywhere in the steel saddle, is reported in Table 9B. Figure 9 shows the concentration of the bending stresses in the steel saddle. To determine the design (LRFD) and allowable (ASD) bending strengths of the steel saddle, the ratio (1000 / M_{max}) is multiplied by the design (LRFD) and allowable (ASD) strengths of the steel saddle plate (the plate design is based on the 1" wide segment to be consistent with internal moment units used in the Visual Analysis). This method ensures that, if a moment equal to the design (LRFD) or allowable (ASD) strength of the steel bracket is applied to the bracket, the resulting maximum internal bending moment located anywhere in the saddle is equal to the design (LRFD) and allowable (ASD) bending strength of the steel plate from which the saddle is made.

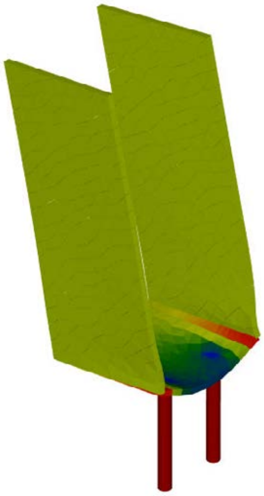


Figure 9: Visual Analysis Model

The design and allowable bending strengths for each model based on the tensile strength of rebar and weld connections are shown in Table 9A. The design and allowable bending strength of the steel bracket is controlled by the bending strength of the 1/4" thick steel saddle (Table 9B). Calculations for fastener bearing against the hole edges, calculations for shear strength and tension strength of steel plates, and calculations for block shear strength are not expected to control the design and are not provided. The minimum length required for the rebar to achieve full strength is provided in Table 9C. The rebar in models SWP 63 and SWP83 can achieve only 87% and 53% of full development, respectively. As a result, the design and allowable bending strength for this model in Table 9A is reduced accordingly.

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-16
Building Code Requirements for Structural Concrete, ACI 318-14

GOVERNING EQUATIONS:

- **REBAR TENSILE STRENGTH: AISC 360, SECTION D2**

Design Tensile Strength	$\phi P_n = \phi F_y A_g$	$\phi = 0.90$	(D2-1)
Allowable Tensile Strength	$P_n / \Omega = F_y A_g / \Omega$	$\Omega = 1.67$	(D2-1)

- **WELDS: AISC 360, SECTION J2**

Design Strength	$\phi R_n = \phi F_w A_w$	$\phi = 0.75$	(J2-3)
Allowable Strength	$R_n / \Omega = F_w A_w / \Omega$	$\Omega = 2.00$	(J2-3)
	$F_w = 0.60F_{EXX}$		(T. J2.5)
	$A_w = Lt_e$, where L = length of weld, t_e = effective weld thickness		

- **BENDING IN STEEL SADDLE: AISC 360, SECTIONS F1 & F11**

Design Bending Strength	$\phi M_n = \phi F_y Z$	$\phi = 0.90$	(F1, F11)
Allowable Bending Strength	$M_n / \Omega = M_n Z / \Omega$	$\Omega = 1.67$	(F1, F11)

- **REBAR DEVELOPMENT REQUIREMENTS, ACI 318, Equation 25.4.2.3a**

Development Length	$L_d = [(3/40)(f_y/\sqrt{f'_c})\ (\Psi_t\ \Psi_e\ \Psi_s) / c_b\ ]\ d_b^2$	(ACI 318, Eq. 25.4.2.3a)

CALCULATIONS:

REBAR PROPERTIES (ASTM A706)		
Rebar Yield Strength, F_y	60	ksi
#4 Rebar Section Area, A_s	0.20	in^2
#5 Rebar Section Area, A_s	0.31	in^2

STEEL PLATE PROPERTIES		
Minimum Yield Strength, F_y	40	ksi
Thickness of steel, t	0.25	in

WELD PROPERTIES		
Effective Weld Thickness (throat) , t_e	0.25	in
Total Weld Length, L, for #4 rebar	1.57	in/bar
Total Weld Length, L, for #5 rebar	1.96	in/bar
Effective Weld Area, $A_w = Lt_e$ for #4	0.39	in^2/bar
Effective Weld Area, $A_w = Lt_e$ for #5	0.49	in^2/bar
Electrode Classification Number	70	ksi
Nominal Strength of Weld Metal, F_w	42	ksi

Table 9A: BENDING STRENGTH BASED ON REBAR AND WELD STRENGTH

Model	N_TA_s (in^2)	ϕP_n (lbf)	P_n / Ω (lbf)	$N_TA_w$$^{(1)}$ (in^2)	ϕR_n (lbf)	R_n / Ω (lbf)	d (in)	LRFD ϕM_n (in-lb)	ASD M_n / Ω (in-lb)
SWP 46	0.40	21600	14371	0.79	24728	16485	3.1	58190	38720
SWP 63	0.40	21600	14371	0.79	24728	16485	3.1	58190	38720
SWP 64	0.40	21600	14371	0.79	24728	16485	3.1	66960	44550
SWP 66	0.40	21600	14371	0.79	24728	16485	3.1	66960	44550
SWP 83	0.62	33480	22275	0.98	30870	20580	4.9	79890	53260
SWP 84	0.62	33480	22275	0.98	30870	20580	4.9	151260	100840
SWP 85	0.62	33480	22275	0.98	30870	20580	4.9	151260	100840
SWP 85	0.62	33480	22275	0.98	30870	20580	4.9	151260	100840

A_s = area of (one) tension rebar

N_T = quantity of tension rebar

d = distance between compression force and tension rebar

$\phi M_n = \min(\varphi P_n, \varphi R_n)\, d$

$M_n / \Omega = \min(P_n/\Omega, R_n/\Omega)\, d$

Table 9B: BENDING STRENGTH BASED ON BENDING OF 1/4" SADDLE

	Bending Strength of 1" wide Plate Sample					Bending Strength of Steel Saddle			
Model ID	w (in)	t (in)	Z (in^3)	ϕM_n (in-lb)	M_n / Ω (in-lb)	M (in-lb)	M_{max} (in-lb/in)	LRFD ϕM_n (in-lb)	ASD M_n / Ω (in-lb)
SWP 46	1.00	0.50	0.0625	2250	1497	1000	48.00	46880	31190
SWP 63	1.00	0.50	0.0625	2250	1497	1000	48.00	46880	31190
SWP 64	1.00	0.50	0.0625	2250	1497	1000	48.00	46880	31190
SWP 66	1.00	0.50	0.0625	2250	1497	1000	48.00	46880	31190
SWP 83	1.00	0.50	0.0625	2250	1497	1000	28.00	80360	53460
SWP 84	1.00	0.50	0.0625	2250	1497	1000	28.00	80360	53460
SWP 85	1.00	0.50	0.0625	2250	1497	1000	28.00	80360	53460
SWP 88	1.00	0.50	0.0625	2250	1497	1000	28.00	80360	53460

w = width of plate sample

t = thickness of plate

$Z = w\, t^2 / 4$

Design Bending Strength of Steel Saddle, $\varphi M_n = (M / M_{max})$ (design bending strength of steel plate)

Allowable Bending Strength of Steel Saddle, $M_n / \Omega = (M / M_{max})$ (allowable bending strength of steel plate)

TABLE 9C: REBAR DEVELOPMENT LENGTH

	#	**d_b**	**f_y**	**f'_c**	**Ψ_t**	**Ψ_e**	**Ψ_s**	**$c_{b, cover}$**	**$c_{b, 1/2\ sp}$**	**L_d**	**L_r**	**Developed**
Model ID		**(in)**	**(ksi)**	**(ksi)**				**(in)**	**(in)**	**(in)**	**(in)**	**%**
SWP 46	4	0.5	60000	3000	1.0	1.5	0.8	2	1.19	**20.7**	18	**87%**
SWP 63	4	0.5	60000	3000	1.0	1.5	0.8	2	1.19	**20.7**	18	**87%**
SWP 64	4	0.5	60000	3000	1.0	1.2	0.8	2	1.94	**10.2**	18	**100%**
SWP 66	4	0.5	60000	3000	1.0	1.5	0.8	2	1.69	**14.6**	18	**100%**
SWP 83	5	0.625	60000	3000	1.0	1.5	0.8	2	1.13	**34.1**	18	**53%**
SWP 84	5	0.625	60000	3000	1.0	1.2	0.8	2	1.88	**16.4**	18	**100%**
SWP 85	5	0.625	60000	3000	1.0	1.2	0.8	2	2.59	**15.4**	18	**100%**
SWP 88	5	0.625	60000	3000	1.0	1.2	0.8	2	2.84	**15.4**	18	**100%**

10.1 STURDI-WALL PLUS: UPLIFT (TENSION) STRENGTH*

***WITH SDS SCREWS BY SIMPSON STRONG TIE**

The calculations are presented in both the LRFD and ASD formats according to provisions of the governing code (AISC 360-16 and NDS 2015). The calculations are completed in Microsoft Excel (2016) using the listed equations. The internal loads in the steel saddle bracket are determined using Visual Analysis (v.18) by IES, Inc.

The rebar in SWP83 model can achieve only 66% of full development. As a result, the design and allowable tensile strength for this model in Table 3A is reduced by a factor of 0.66.

The load on each fastener type (screw, bolt) is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus all fasteners: $N_s k_s/k_g$, $N_b k_b/k_g$, where N_s is the quantity of screws per bracket, N_b is the quantity of bolts in double shear per bracket, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of all fasteners (see Tables 10E and 10G). The slip-modulus of the screw fasteners does not equal the slip modulus of the bolt fastener(s): $N_s k_s \neq N_b k_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Table 10G). The discussion and calculations for slip-modulus are provided in Section 7.

Table 10A shows tensile strength of the SWP brackets based on tensile strength of rebar, weld strength, and tensile strength of vertical plates. Table 10B shows the tensile strength of the SWP brackets based on the bending strength of the steel saddle (controls the design). Table 10F shows the tensile strength of the SWP steel brackets based on the lateral (shear) strength of the steel-to-wood connection.

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-16
National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

- **REBAR AND STEEL SADDLE: AISC 360, SECTION D2**

Design Tensile Strength	$\phi P_n = \phi F_y A_g$	**(tensile yielding)**	$\phi = 0.90$	(D2-1)
	$\phi P_n = \phi F_u A_e$	**(tensile rupture)**	$\phi = 0.75$	(D2-2)
Allowable Tensile Strength	$P_n / \Omega = F_y A_g / \Omega$	**(tensile yielding)**	$\Omega = 1.67$	(D2-1)
	$P_n / \Omega = F_u A_e / \Omega$	**(tensile rupture)**	$\Omega = 2.00$	(D2-2)

- **WELDS: AISC 360, SECTION J2**

Design Strength	$\phi R_n = \phi F_w A_w$	$\phi = 0.75$	(J2-3)
Allowable Strength	$R_n / \Omega = F_w A_w / \Omega$	$\Omega = 2.00$	(J2-3)
	$F_w = 0.60 F_{EXX}$		(T. J2.5)

- **BENDING IN STEEL SADDLE: AISC 360, SECTIONS F1 & F11**

Design Bending Strength	$\phi M_n = \phi F_y Z$	$\phi = 0.90$	(F1, F11)
Allowable Bending Strength	$M_n / \Omega = M_n Z / \Omega$	$\Omega = 1.67$	(F1, F11)

- **STEEL-TO-WOOD CONNECTION (BOLT, SCREWS): NDS 2015**

Allowable Lateral Strength of Screws	$Z'_{s,\,ASD}\,N_s = N_s\,Z\,C_D\,C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\,LRFD}\,N_s = \varphi\,N_s\,Z\,\lambda\,C_\Delta\,K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\,ASD}\,N_b = N_b\,Z\,C_D\,C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\,LRFD}\,N_b = \varphi\,N_b\,Z\,\lambda\,C_\Delta\,K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Allowable Lateral Strength of Mixed Fasteners	$V_a = \min\ [\,Z'_{s,\,ASD}\,(k_g/k_s)\,,\ Z'_{b,\,ASD}\,(k_g/k_b)\,]$
Design Lateral Strength of Mixed Fasteners	$\varphi V = \min\ [\,Z'_{s,\,LRFD}\,(k_g/k_s)\,,\ Z'_{b,\,LRFD}\,(k_g/k_b)\,]$

CALCULATIONS:

REBAR PROPERTIES		
Rebar Yield Strength, F_y	60	ksi

STEEL SADDLE BRACKET PROPERTIES		
Minimum Tensile Strength, F_u	55	ksi
Minimum Yield Strength, Fy	40	ksi
Thickness of steel, t	0.250	in

WELD PROPERTIES		
Effective Weld Thickness (throat) , t_e	0.25	in
Electrode Classification Number	70	ksi
Nominal Strength of Weld Metal, F_w	42	ksi

TABLE 10.1A: DESIGN TENSILE STRENGTH AND ALLOWABLE TENSILE STRENGTH (REBAR, WELDS, AND VERTICAL STEEL PLATES)

	Tensile Strength of Rebar and Welds						Tensile Strength of Steel Saddle Vertical Plates					
	Rebar Tensile Strength			Weld Strength			Yielding			Rupture		
Model ID	A_s (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)	A_w (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)	A_g (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)	A_e (in^2)	LRFD ϕR_n (lb)	ASD R_n / Ω (lb)
SWP 46	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 63	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 64	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 66	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 83	1.24	44194	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725
SWP 84	1.24	66960	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725
SWP 85	1.24	66960	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725
SWP 88	1.24	66960	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725

TABLE 10.1B: DESIGN TENSILE STRENGTH AND ALLOWABLE TENSILE STRENGTH AS DEFINED BY THE BENDING STRENGTH OF THE STEEL SADDLE

Model ID	t (in)	w (in)	F_y (ksi)	Z (in^3)	ϕM_n (in-lb)	M_n / Ω (in-lb)	k (lb-in)	LRFD ϕT_n (lb)	ASD T_n / Ω (lb)
SWP 46	0.250	5.00	40	0.078	2813	1871	0.2725	10320	6870
SWP 63	0.250	5.00	40	0.078	2813	1871	0.2725	10320	6870
SWP 64	0.250	5.00	40	0.078	2813	1871	0.3102	9070	6030
SWP 66	0.250	5.00	40	0.078	2813	1871	0.3005	9360	6230
SWP 83	0.250	7.00	40	0.109	3938	2620	0.2507	15710	10450
SWP 84	0.250	7.00	40	0.109	3938	2620	0.2898	13590	9040
SWP 85	0.250	7.00	40	0.109	3938	2620	0.3191	12340	8210
SWP 88	0.250	7.00	40	0.109	3938	2620	0.3271	12040	8010

(1) t = thickness of steel plate (saddle)
(2) w = width of steel plate (saddle)
(3) Z is plastic section modulus = $w t^2 / 4$
(3) Factor "k" represents the maximum moment found anywhere in the steel saddle under 1 pound of tension force. This factor was determined using a two dimensional computer model for each SWP model and equals Moment divided by total applied downward force, k = M/F.
(4) Tension strength, as defined by the bending strength of the steel saddle bracket, is determined using the following expressions: $\phi T_n = \phi M_n/k$, $T_n / \Omega = (M_n/k) / \Omega$

TABLE 10.1C: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A

		SDS	F_{yb}	164000	$1+R_e$	1.1	θ	0
Screw Diameter (in)	D	0.242	$F_{em,\ par}$	5526	$2+R_e$	2.1	l_m	1259.3
Screw Length (in)	L	3	$F_{em,\ perp}$	5526	k_1	0.408	l_s	1280.4
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.536	II	522.4
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	6.944	III_m	572.7
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es,\ par}$	61800	III_s	380.5
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.920	$F_{es,\ perp}$	61800	IV	472.3
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800	D_r	0.242
Lateral Design Value (lbs)	Z	380	LRFD resistance factor				ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor				λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor				K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s,\ ASD}$**	**609**	**LRFD Adjusted Lateral Design Value (lbs)**				**$Z'_{s,\ LRFD}$**	**821**

TABLE 10.1D: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A

Bolt Diameter (in)	D	0.5	$F_{em,\ par}$	6160	K_θ	1.000	l_m	3465
Main Member Thickness (in)	$t_{m,\ min}$	4.5	$F_{em,\ perp}$	3626	$1+R_e$	1.071	III_s	2369
Side Member Thickness (in)	t_s	0.25	F_{em}	6160	$2+R_e$	2.071	IV	3150
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.071	k_3	10.192		
Bolt Yield Strength (psi)	F_{yb}	106000						
Max Angle Load to Grain (deg)	θ	0						
Specific Gravity	G	0.55						
Reference Lateral Design Value (Z)	Z	2369	LRFD resistance factor				ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor				λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor				K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b,\ ASD}$**	**3790**	**LRFD Adjusted Lateral Design Value (lbs)**				**$Z'_{b,\ LRFD}$**	**5112**

TABLE 10.1E: FASTENER SLIP-MODULUS

Model	k_s (lb/in)	k_b (lb/in)	N_s	N_b	k_g (lb/in)
SWP 46	32143	95459	4	2	319491
SWP 63	32143	95459	4	2	319491
SWP 64	32143	95459	4	2	319491
SWP 66	32143	95459	4	2	319491
SWP 83	32143	95459	8	2	448063
SWP 84	32143	95459	8	2	448063
SWP 85	32143	95459	8	2	448063
SWP 88	32143	95459	8	2	448063

TABLE 10.1F: TENSILE STRENGTH BASED ON STEEL-TO-WOOD SHEAR CONNECTION

Model	$Z'_{s,\ LRFD}$ (k_g/k_s) (lb)	$Z'_{b,\ LRFD}$ (k_g/k_b) (lb)	$Z'_{s,\ ASD}$ (k_g/k_s) (lb)	$Z'_{b,\ ASD}$ (k_g/k_b) (lb)	**LRFD** $\boldsymbol{\varphi V_n}$ **(lb)**	**ASD** $\boldsymbol{V_n/\Omega}$ **(lb)**
SWP 46	8161	17108	6051	12684	**8160**	**6050**
SWP 63	8161	17108	6051	12684	**8160**	**6050**
SWP 64	8161	17108	6051	12684	**8160**	**6050**
SWP 66	8161	17108	6051	12684	**8160**	**6050**
SWP 83	11446	23993	8486	17789	**11450**	**8490**
SWP 84	11446	23993	8486	17789	**11450**	**8490**
SWP 85	11446	23993	8486	17789	**11450**	**8490**
SWP 88	11446	23993	8486	17789	**11450**	**8490**

TABLE 10.1G: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO

	Load Distribution		Load / Strength	
Model	Screws	Bolts	Screws	Bolts
SWP 46	40.2%	59.8%	100%	48%
SWP 63	40.2%	59.8%	100%	48%
SWP 64	40.2%	59.8%	100%	48%
SWP 66	40.2%	59.8%	100%	48%
SWP 83	57.4%	42.6%	100%	48%
SWP 84	57.4%	42.6%	100%	48%
SWP 85	57.4%	42.6%	100%	48%
SWP 88	57.4%	42.6%	100%	48%

TALBE 10.1H: SUMMARY

Model	**LRFD** $\boldsymbol{\phi T_n}$ **(lb)**	**ASD** $\boldsymbol{T_n / \Omega}$ **(lb)**
SWP 46	6,515	4,835
SWP 63	6,515	4,835
SWP 64	6,515	4,835
SWP 66	6,515	4,835
SWP 83	11,450	8,490
SWP 84	11,450	8,490
SWP 85	11,450	8,210
SWP 88	11,450	8,010

NOTE: uplift values for SWP 46, 63, 64 and 66 models have been reduced per latest test (values are limited by 1/8" displacement criterion)

10.2 STURDI-WALL PLUS: UPLIFT (TENSION) STRENGTH*

***WITH PROPRIATARY SCREWS**

The calculations are presented in both the LRFD and ASD formats according to provisions of the governing code (AISC 360-16 and NDS 2015). The calculations are completed in Microsoft Excel (2016) using the listed equations. The internal loads in the steel saddle bracket are determined using Visual Analysis (v.18) by IES, Inc.

The rebar in SWP83 model can achieve only 66% of full development. As a result, the design and allowable tensile strength for this model in Table 3A is reduced by a factor of 0.66.

The load on each fastener type (screw, bolt) is proportional to the ratio of the slip-modulus of the fastener type to the cumulative slip-modulus all fasteners: $N_s k_s/k_g$, $N_b k_b/k_g$, where N_s is the quantity of screws per bracket, N_b is the quantity of bolts in double shear per bracket, k_s is the slip-modulus of one screw in single shear, k_b is the slip-modulus of one bolt in double shear, and k_g is the cumulative slip-modulus of all fasteners (see Tables 10E and 10G). The slip-modulus of the screw fasteners does not equal the slip modulus of the bolt fastener(s): $N_s k_s \neq N_b k_b$. As a result, one fastener type is loaded to the maximum allowable or design lateral strength, while the second fastener type receives the balance of the load which will not reach the fastener's maximum capacity (Table 10G). The discussion and calculations for slip-modulus are provided in Section 7.

Table 10A shows tensile strength of the SWP brackets based on tensile strength of rebar, weld strength, and tensile strength of vertical plates. Table 10B shows the tensile strength of the SWP brackets based on the bending strength of the steel saddle (controls the design). Table 10F shows the tensile strength of the SWP steel brackets based on the lateral (shear) strength of the steel-to-wood connection.

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-16
National Design Specification for Wood Construction, NDS (2015)

GOVERNING EQUATIONS:

- **REBAR AND STEEL SADDLE: AISC 360, SECTION D2**

Design Tensile Strength	$\phi P_n = \phi F_y A_g$	**(tensile yielding)**	$\phi = 0.90$	(D2-1)
	$\phi P_n = \phi F_u A_e$	**(tensile rupture)**	$\phi = 0.75$	(D2-2)
Allowable Tensile Strength	$P_n / \Omega = F_y A_g / \Omega$	**(tensile yielding)**	$\Omega = 1.67$	(D2-1)
	$P_n / \Omega = F_u A_e / \Omega$	**(tensile rupture)**	$\Omega = 2.00$	(D2-2)

- **WELDS: AISC 360, SECTION J2**

Design Strength	$\phi R_n = \phi F_w A_w$	$\phi = 0.75$	(J2-3)
Allowable Strength	$R_n / \Omega = F_w A_w / \Omega$	$\Omega = 2.00$	(J2-3)
	$F_w = 0.60F_{EXX}$		(T. J2.5)

- **BENDING IN STEEL SADDLE: AISC 360, SECTIONS F1 & F11**

Design Bending Strength	$\phi M_n = \phi F_y Z$	$\phi = 0.90$	(F1, F11)
Allowable Bending Strength	$M_n / \Omega = M_n Z / \Omega$	$\Omega = 1.67$	(F1, F11)

- **STEEL-TO-WOOD CONNECTION (BOLT, SCREWS): NDS 2015**

Allowable Lateral Strength of Screws	$Z'_{s,\ ASD} N_s = N_s Z C_D C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Screws	$Z'_{s,\ LRFD} N_s = \varphi N_s Z \lambda C_\Delta K_F$	NDS Table 11.3.1
Allowable Lateral Strength of Bolt(s)	$Z'_{b,\ ASD} N_b = N_b Z C_D C_\Delta$	NDS Table 11.3.1
Design Lateral Strength of Bolt(s)	$Z'_{b,\ LRFD} N_b = \varphi N_b Z \lambda C_\Delta K_F$	NDS Table 11.3.1

Z = Unadjusted reference lateral (shear) design value for one fastener	NDS Table 12.3.1A
Z' = Adjusted lateral design value for one fastener	NDS Table 11.3.1
C_D = ASD load duration factor	NDS Table 2.3.2
C_Δ = Geometry factor	NDS 12.5.1
N = total quantity of fasteners in the group	
ϕ = LRFD resistance factor	NDS Table N2
λ = LRFD time effect factor	NDS Table N3
K_F = ASD to LRFD format conversion factor	NDS Table N1
Subscript "s" = screws	
Subscript "b" = bolts	

Allowable Lateral Strength of Mixed Fasteners	$V_a = \min\ [\ Z'_{s,\ ASD} (k_g/k_s)\ ,\ Z'_{b,\ ASD} (k_g/k_b)\ ]$
Design Lateral Strength of Mixed Fasteners	$\varphi V = \min\ [\ Z'_{s,\ LRFD} (k_g/k_s)\ ,\ Z'_{b,\ LRFD} (k_g/k_b)\ ]$

CALCULATIONS:

REBAR PROPERTIES		
Rebar Yield Strength, F_y	60	ksi

STEEL SADDLE BRACKET PROPERTIES		
Minimum Tensile Strength, F_u	55	ksi
Minimum Yield Strength, Fy	40	ksi
Thickness of steel, t	0.250	in

WELD PROPERTIES		
Effective Weld Thickness (throat) , t_e	0.25	in
Electrode Classification Number	70	ksi
Nominal Strength of Weld Metal, F_w	42	ksi

TABLE 10.2A: DESIGN TENSILE STRENGTH AND ALLOWABLE TENSILE STRENGTH (REBAR, WELDS, AND VERTICAL STEEL PLATES)

	Tensile Strength of Rebar and Welds						**Tensile Strength of Steel Saddle Vertical Plates**					
	Rebar Tensile Strength			**Weld Strength**			**Yielding**			**Rupture**		
Model ID	**A_s (in^2)**	**LRFD ϕR_n (lb)**	**ASD R_n / Ω (lb)**	**A_w (in^2)**	**LRFD ϕR_n (lb)**	**ASD R_n / Ω (lb)**	**A_g (in^2)**	**LRFD ϕR_n (lb)**	**ASD R_n / Ω (lb)**	**A_e (in^2)**	**LRFD ϕR_n (lb)**	**ASD R_n / Ω (lb)**
SWP 46	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 63	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 64	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 66	0.80	43200	28740	1.57	49460	32970	2.5	90000	59880	2.19	90338	60225
SWP 83	1.24	44194	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725
SWP 84	1.24	66960	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725
SWP 85	1.24	66960	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725
SWP 88	1.24	66960	44550	1.96	61740	41160	3.5	126000	83832	3.19	131588	87725

TABLE 10.2B: DESIGN TENSILE STRENGTH AND ALLOWABLE TENSILE STRENGTH AS DEFINED BY THE BENDING STRENGTH OF THE STEEL SADDLE

Model ID	t (in)	w (in)	F_y (ksi)	Z (in^3)	ϕM_n (in-lb)	M_n / Ω (in-lb)	k (lb-in)	LRFD ϕT_n (lb)	ASD T_n / Ω (lb)
SWP 46	0.250	5.00	40	0.078	2813	1871	0.2725	10320	6870
SWP 63	0.250	5.00	40	0.078	2813	1871	0.2725	10320	6870
SWP 64	0.250	5.00	40	0.078	2813	1871	0.3102	9070	6030
SWP 66	0.250	5.00	40	0.078	2813	1871	0.3005	9360	6230
SWP 83	0.250	7.00	40	0.109	3938	2620	0.2507	15710	10450
SWP 84	0.250	7.00	40	0.109	3938	2620	0.2898	13590	9040
SWP 85	0.250	7.00	40	0.109	3938	2620	0.3191	12340	8210
SWP 88	0.250	7.00	40	0.109	3938	2620	0.3271	12040	8010

(1) t = thickness of steel plate (saddle)

(2) w = width of steel plate (saddle)

(3) Z is plastic section modulus = $w t^2 / 4$

(3) Factor "k" represents the maximum moment found anywhere in the steel saddle under 1 pound of tension force. This factor was determined using a two dimensional computer model for each SWP model and equals Moment divided by total applied downward force, k = M/F.

(4) Tension strength, as defined by the bending strength of the steel saddle bracket, is determined using the following expressions: $\phi T_n = \phi M_n/k$, $T_n / \Omega = (M_n/k) / \Omega$

TABLE 10.2C: ADJUSTED LATERAL DESIGN VALUE OF ONE SCREW: NDS Table 12.3.1A

			F_{yb}	199000	$1+R_e$	1.1	θ	0
Screw Diameter (in)	D	0.243	$F_{em, par}$	5526	$2+R_e$	2.1	I_m	1260.2
Screw Length (in)	L	3	$F_{em, perp}$	5526	k_1	0.408	I_s	1281.3
Thickness of Steel Plate Member (in)	l_s	0.25	F_{em}	5526	k_2	0.549	II	522.7
Thickness of Wood Member (in)	l_m	4.5	R_e	0.089	k_3	7.471	III_m	587.1
Screw Penetration into main member (in)	p	2.75	R_t	11.000	$F_{es, par}$	61800	III_s	409.7
Minimum Allowed Penetration, p_{min} = 6D	p_{min}	1.5	K_o	2.930	$F_{es, perp}$	61800	IV	522.8
Specific Gravity of Wood Member	G	0.55	p	2.8	F_{es}	61800	D_r	0.243
Lateral Design Value (lbs)	Z	410	LRFD resistance factor				ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor				λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor				K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{s, ASD}$**	**655**	**LRFD Adjusted Lateral Design Value (lbs)**				**$Z'_{s, LRFD}$**	**884**

TABLE 10.2D: ADJUSTED LATERAL DESIGN VALUE OF ONE BOLT (DOUBLE SHEAR): NDS Table 12.3.1A

Bolt Diameter (in)	D	0.5	$F_{em, par}$	6160	K_θ	1.000	I_m	3465
Main Member Thickness (in)	$t_{m, min}$	4.5	$F_{em, perp}$	3626	$1+R_e$	1.071	III_s	2369
Side Member Thickness (in)	t_s	0.25	F_{em}	6160	$2+R_e$	2.071	IV	3150
Dowel Bearing Strength (psi)	F_{es}	87000	R_e	0.071	k_3	10.192		
Bolt Yield Strength (psi)	F_{yb}	106000						
Max Angle Load to Grain (deg)	θ	0						
Specific Gravity	G	0.55						
Reference Lateral Design Value (Z)	Z	2369	LRFD resistance factor				ϕ	0.65
ASD Load Duration Factor	C_D	1.6	LRFD time effect factor				λ	1
Geometry Factor	C_Δ	1	ASD to LRFD format conversion factor				K_F	3.32
ASD Adjusted Lateral Design Value (lbs)	**$Z'_{b, ASD}$**	**3790**	**LRFD Adjusted Lateral Design Value (lbs)**				**$Z'_{b, LRFD}$**	**5112**

TABLE 10.2E: FASTENER SLIP-MODULUS

Model	k_s (lb/in)	k_b (lb/in)	N_s	N_b	k_g (lb/in)
SWP 46	32143	95459	4	2	319491
SWP 63	32143	95459	4	2	319491
SWP 64	32143	95459	4	2	319491
SWP 66	32143	95459	4	2	319491
SWP 83	32143	95459	8	2	448063
SWP 84	32143	95459	8	2	448063
SWP 85	32143	95459	8	2	448063
SWP 88	32143	95459	8	2	448063

TABLE 10.2F: TENSILE STRENGTH BASED ON STEEL-TO-WOOD SHEAR CONNECTION

Model	$Z'_{s, LRFD}$ (k_g/k_s) (lb)	$Z'_{b, LRFD}$ (k_g/k_b) (lb)	$Z'_{s, ASD}$ (k_g/k_s) (lb)	$Z'_{b, ASD}$ (k_g/k_b) (lb)	**LRFD φV_n (lb)**	**ASD V_n/Ω (lb)**
SWP 46	8787	17108	6515	12684	**8790**	**6520**
SWP 63	8787	17108	6515	12684	**8790**	**6520**
SWP 64	8787	17108	6515	12684	**8790**	**6520**
SWP 66	8787	17108	6515	12684	**8790**	**6520**
SWP 83	12324	23993	9137	17789	**12320**	**9140**
SWP 84	12324	23993	9137	17789	**12320**	**9140**
SWP 85	12324	23993	9137	17789	**12320**	**9140**
SWP 88	12324	23993	9137	17789	**12320**	**9140**

TABLE 10.2G: LOAD DISTRIBUTION RATIO AND LOAD-TO-STRENGTH RATIO

	Load Distribution		Load / Strength	
Model	Screws	Bolts	Screws	Bolts
SWP 46	40.2%	59.8%	100%	51%
SWP 63	40.2%	59.8%	100%	51%
SWP 64	40.2%	59.8%	100%	51%
SWP 66	40.2%	59.8%	100%	51%
SWP 83	57.4%	42.6%	100%	51%
SWP 84	57.4%	42.6%	100%	51%
SWP 85	57.4%	42.6%	100%	51%
SWP 88	57.4%	42.6%	100%	51%

TALBE 10.2H: SUMMARY

Model	**LRFD ϕT_n (lb)**	**ASD T_n / Ω (lb)**
SWP 46	6,515	4,835
SWP 63	6,515	4,835
SWP 64	6,515	4,835
SWP 66	6,515	4,835
SWP 83	12,320	9,140
SWP 84	12,320	9,040
SWP 85	12,320	8,210
SWP 88	12,040	8,010

NOTE: uplift values for SWP 46, 63, 64 and 66 models have been reduced per latest test (values are limited by 1/8" displacement criterion)

11. STURDI-WALL PLUS: SHEAR STRENGTH OF BRACKET-TO-CONCRETE CONNECTION

The calculations are presented in both the LRFD and ASD formats in accordance with the provisions of the governing codes (AISC 360-16 and ACI 318-14). The calculations include (1) shear strength of rebar and (2) concrete prying or breakout forces. The shear strength of rebar is covered in both ACI 318 and AISC 360 standards. It is desirable to present the results in terms of ASD and LRFD design, therefore, AISC 360, which includes both methods, is used for steel anchor design calculations. The breakout calculations are completed using the LRFD method per ACI 318 and are converted to the ASD levels using the conversion factor, α = 1.6. The calculations are completed in Microsoft Excel (2016) using the listed equations.

GOVERNING CODE:

Specification for Structural Steel Buildings ANSI/AISC 360-1
Building Code Requirements for Structural Concrete ACI 318-1

GOVERNING EQUATIONS:

- **WELDS: AISC 360, SECTION J2**

Design Strength	$\phi R_n = \phi F_w A_w$	$\phi = 0.75$	(J2-3)
Allowable Strength	$R_n / \Omega = F_w A_w / \Omega$	$\Omega = 2.00$	(J2-3)
	$F_w = 0.60 F_{EXX}$		(T. J2.5)

- **REBAR: AISC 360, SECTION J3**

Design Strength	$\phi R_n N_r = \phi N_r F_{nv} A_s$	$\phi = 0.75$	(J3-1)
Allowable Strength	$R_n N_r / \Omega = N_r F_{nv} A_s / \Omega$	$\Omega = 2.00$	(J3-1)

N_r = number of rebar

- **ANCHORS (CONCRETE BREAKOUT): ACI 318, 17.5.2.**

Design Strength	$\varphi V_{cbg} = \varphi(A_{Vc} / A_{Vco}) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$	$\phi = 0.70$	(17.5.2.1b)
Allowable Strength	$V_{cbg}/\Omega = \varphi V_{cbg} / \alpha$	$\alpha = 1.6$	

CALCULATIONS:

REBAR PROPERTIES		
Nominal Shear Stress, F_{nv}	32 (or greater)	ksi

CONCRETE PROPERTIES		
Compressive Strength, f'_c	3 (or greater)	ksi

WELD PROPERTIES		
Effective Weld Thickness (throat) , t_e	0.25	in
Electrode Classification Number	70	ksi
Nominal Strength of Weld Metal, F_w	42	ksi

TABLE 11A: DESIGN SHEAR STRENGTH AND ALLOWABLE SHEAR STRENGTH (REBAR AND WELDS)

	Rebar Shear Strength				Weld Strength		
			LRFD	ASD		LRFD	ASD
Model ID	N_r	A_s (in^2)	ϕR_n (lb)	R_n / Ω (lb)	A_w (in^2)	ϕR_n (lb)	R_n / Ω (lb)
SWP 46	4.00	0.20	19200	15329	1.57	49455	32970
SWP 63	4.00	0.20	19200	15329	1.57	49455	32970
SWP 64	4.00	0.20	19200	15329	1.57	49455	32970
SWP 66	4.00	0.20	19200	15329	1.57	49455	32970
SWP 83	4.00	0.31	29760	23760	1.96	61740	41160
SWP 84	4.00	0.31	29760	23760	1.96	61740	41160
SWP 85	4.00	0.31	29760	23760	1.96	61740	41160
SWP 88	4.00	0.31	29760	23760	1.96	61740	41160

TABLE 11B: SHEAR STRENGTH BASED ON CONCRETE BREAKOUT STRENGTH

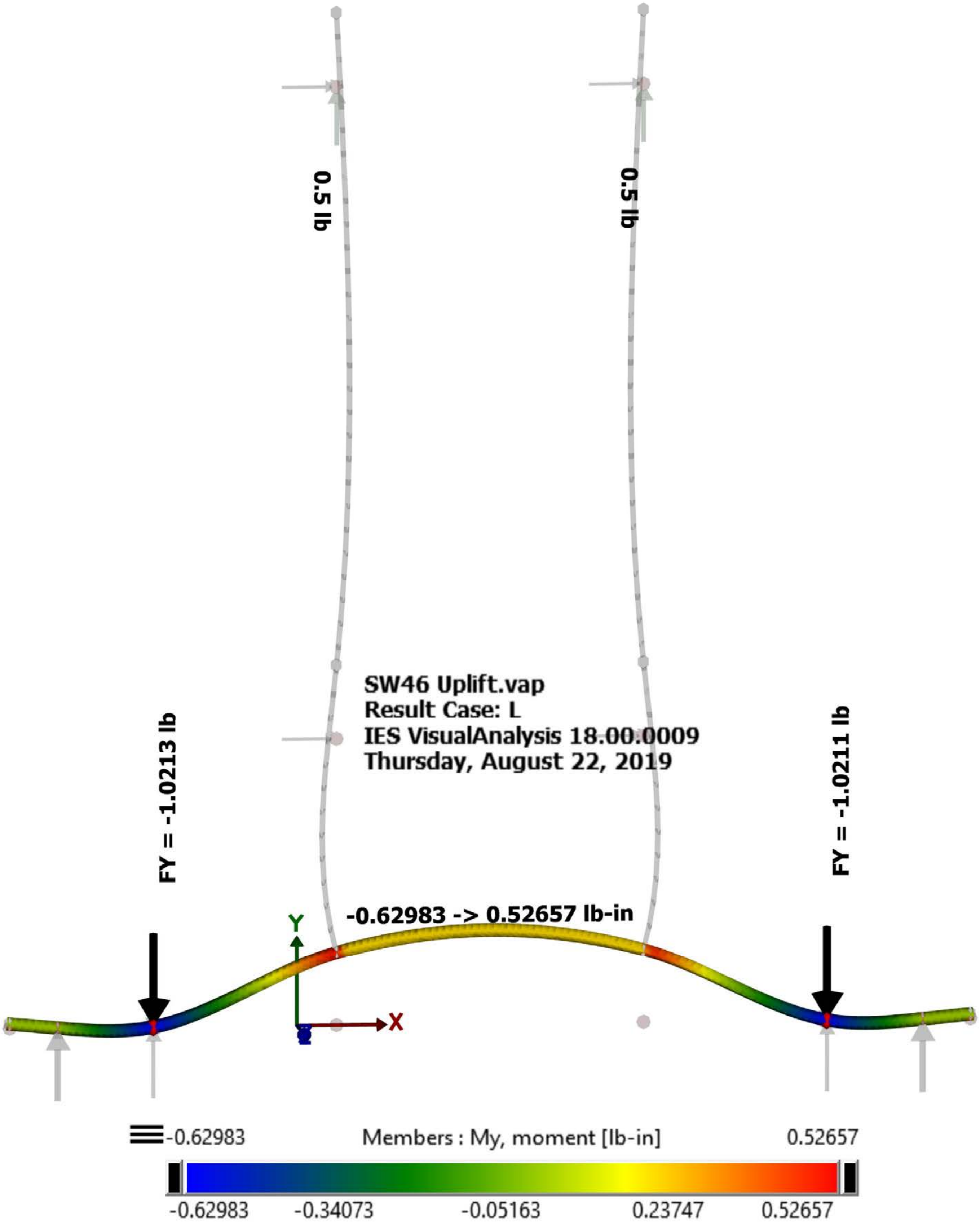
Model ID	**$c_{a1,2}$ (in)**	**S_1 (in)**	**A_{Vc} (in^2)**	**A_{Vco} (in^2)**	**Ψ**	**h_{ef} (in)**	**V_b (lb)**	**LRFD φV_{cbg} (lb)**	**ASD V_{cbg}/Ω (lb)**
SWP 46	4.75	2.250	117.6	101.5	1.0	18.0	5103	4140	2590
SWP 63	4.75	2.250	117.6	101.5	1.0	18.0	5103	4140	2590
SWP 64	4.75	3.750	128.3	101.5	1.0	18.0	5103	4510	2820
SWP 66	4.75	3.250	124.7	101.5	1.0	18.0	5103	4390	2740
SWP 83	6.38	2.250	204.4	182.9	1.0	18.0	7935	6210	3880
SWP 84	6.38	3.750	218.7	182.9	1.0	18.0	7935	6640	4150
SWP 85	6.38	5.250	233.1	182.9	1.0	18.0	7935	7080	4430
SWP 88	6.38	5.250	233.1	182.9	1.0	18.0	7935	7080	4430

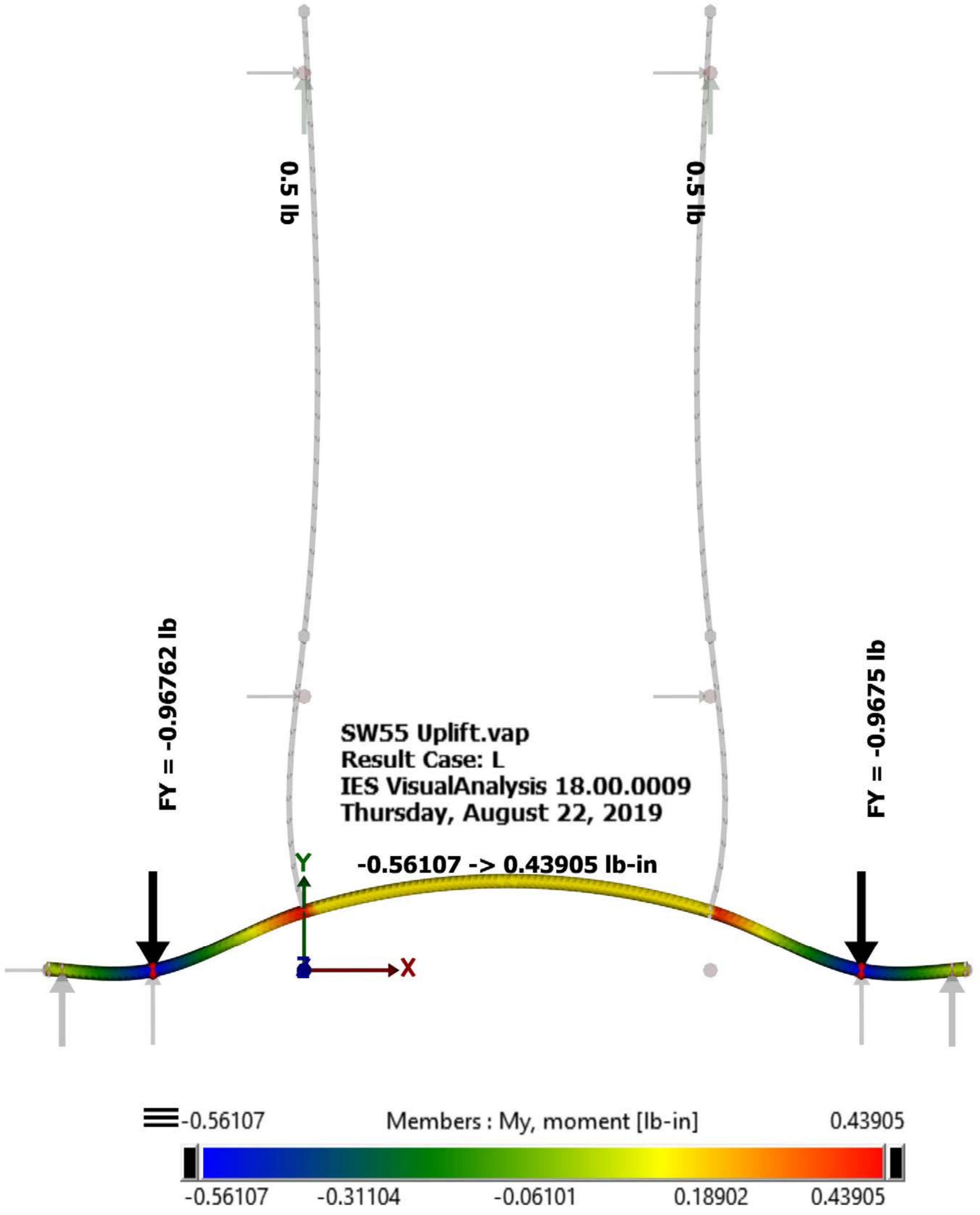
(1) c_{a1} = distance from center of anchor to edge of concrete
(2) S_1 = spacing between rebar
(3) $\Psi_{ec,V}$ $\Psi_{ed,V}$ $\Psi_{c,V}$ $\Psi_{h,V}$ = 1.0, higher values possible if concrete is properly reinforced around the anchors
(4) h_{ef} is effective rebar embedment into concrete
(5) Calculations are based on 2" concrete cover to nearest rebar in the direction of the loa
(6) $c_{a1,2}$, S_1, A_{Vc}, A_{Vco}, Ψ, V_b and V_{cbg} as defined in ACI 318

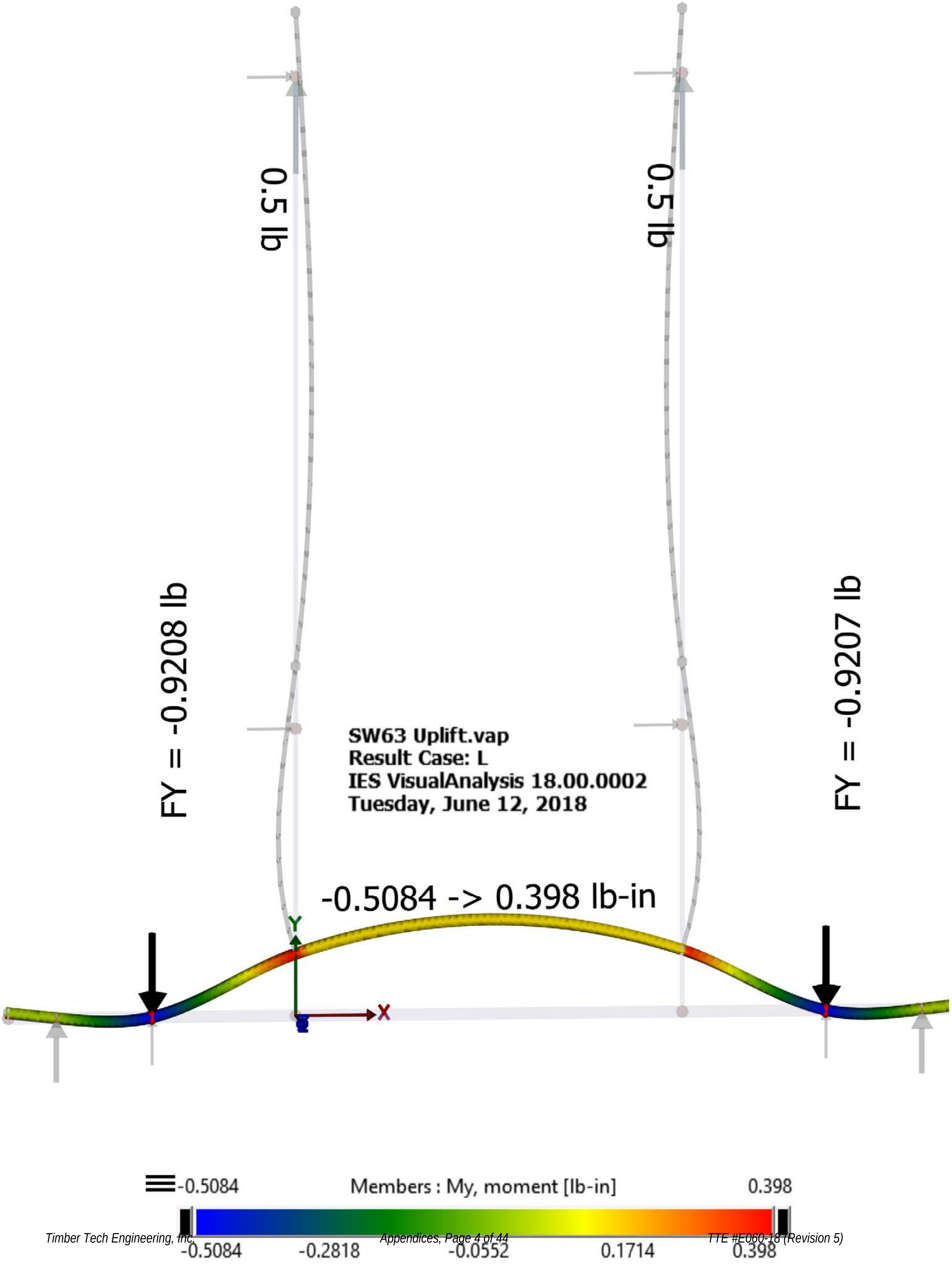
APPENDIX A

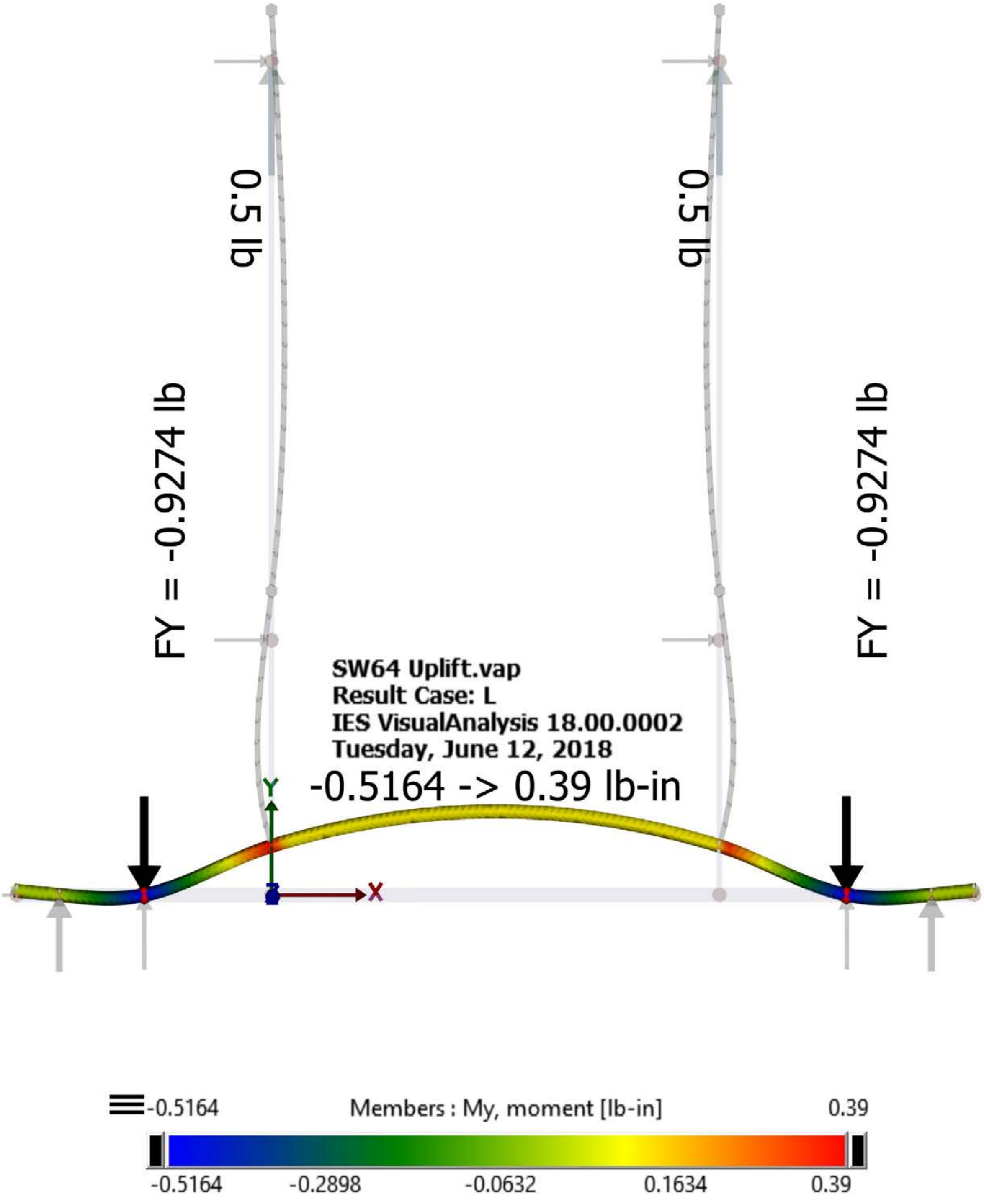
Structural Models of Sturdi-Wall Brackets To Determine the Maximum Bending Moment (Unity) In Steel Bracket and to Determine Uplift Forces (Unity) on Anchors

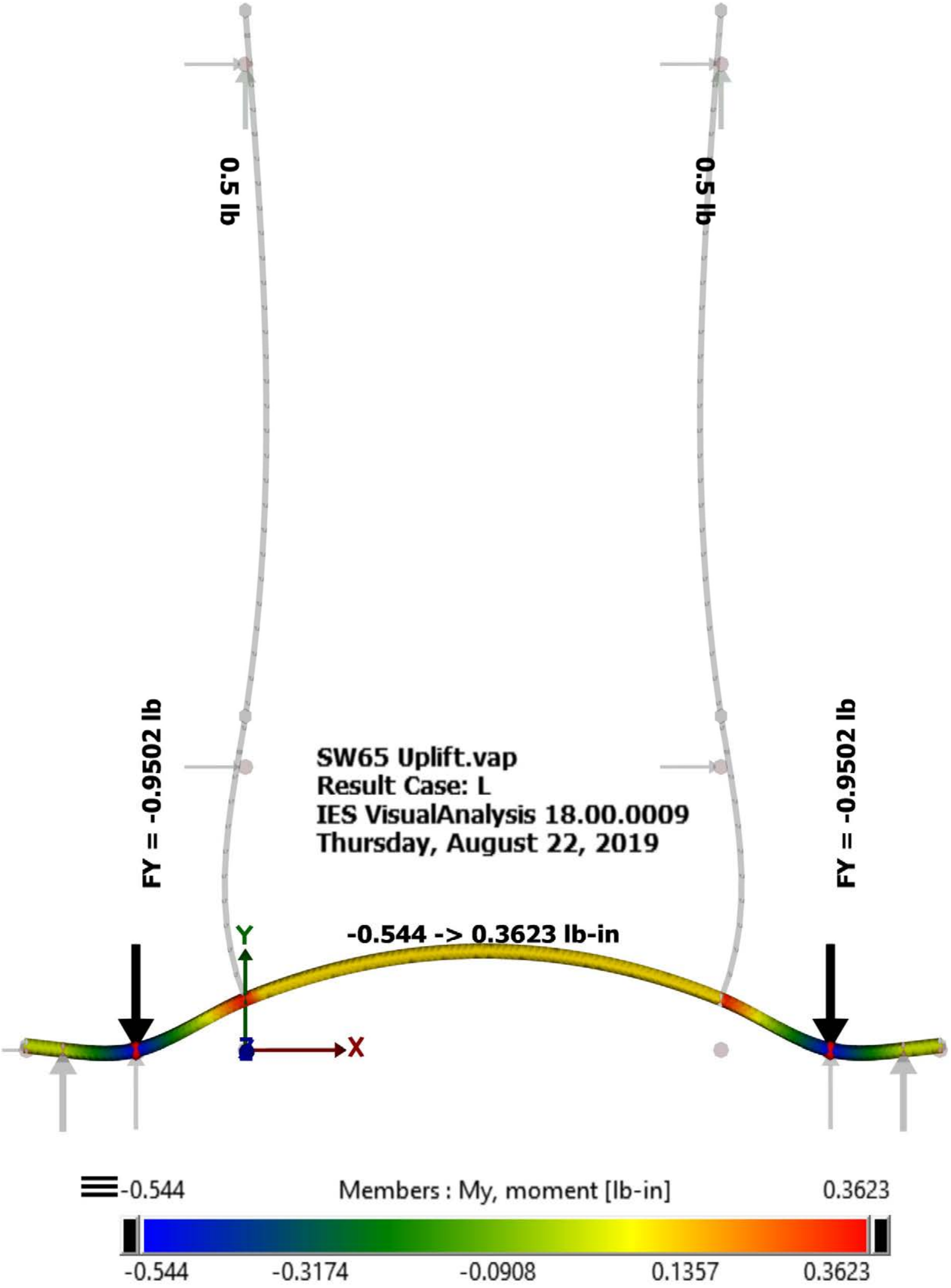
Visual Analysis by IES, Inc
Version 18

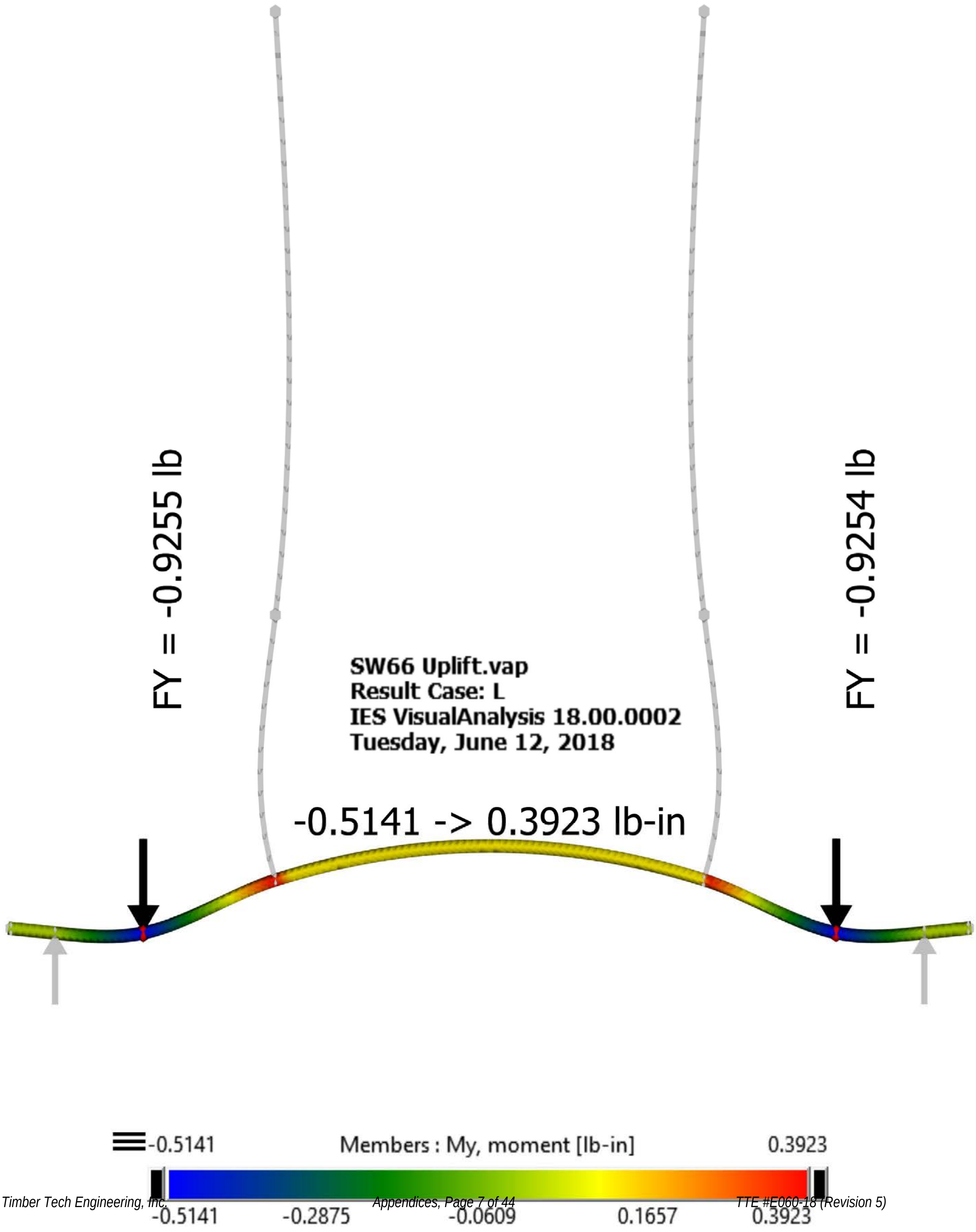

0.5 lb
0.5 lb
SW46 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Thursday, August 22, 2019
FY = -1.0213 lb
FY = -1.0211 lb
-0.62983 -> 0.52657 lb-in
Y
X
-0.62983
Members : My, moment [lb-in]
0.52657
-0.62983
-0.34073
-0.05163
0.23747
0.52657

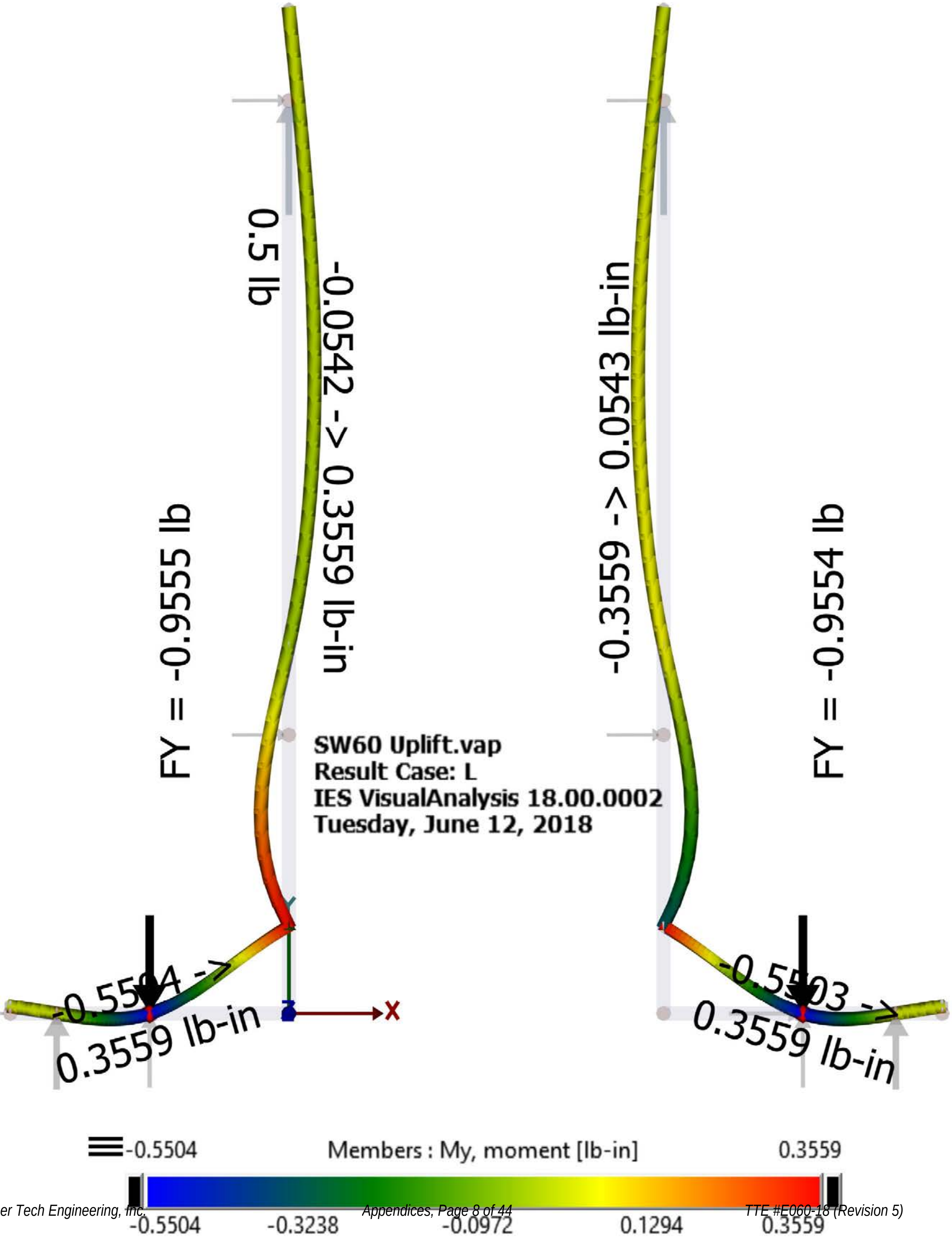

0.5 lb
0.5 lb
FY = -0.96762 lb
FY = -0.9675 lb
SW55 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Thursday, August 22, 2019
-0.56107 -> 0.43905 lb-in
Y
X
-0.56107
Members : My, moment [lb-in]
0.43905
-0.56107
-0.31104
-0.06101
0.18902
0.43905

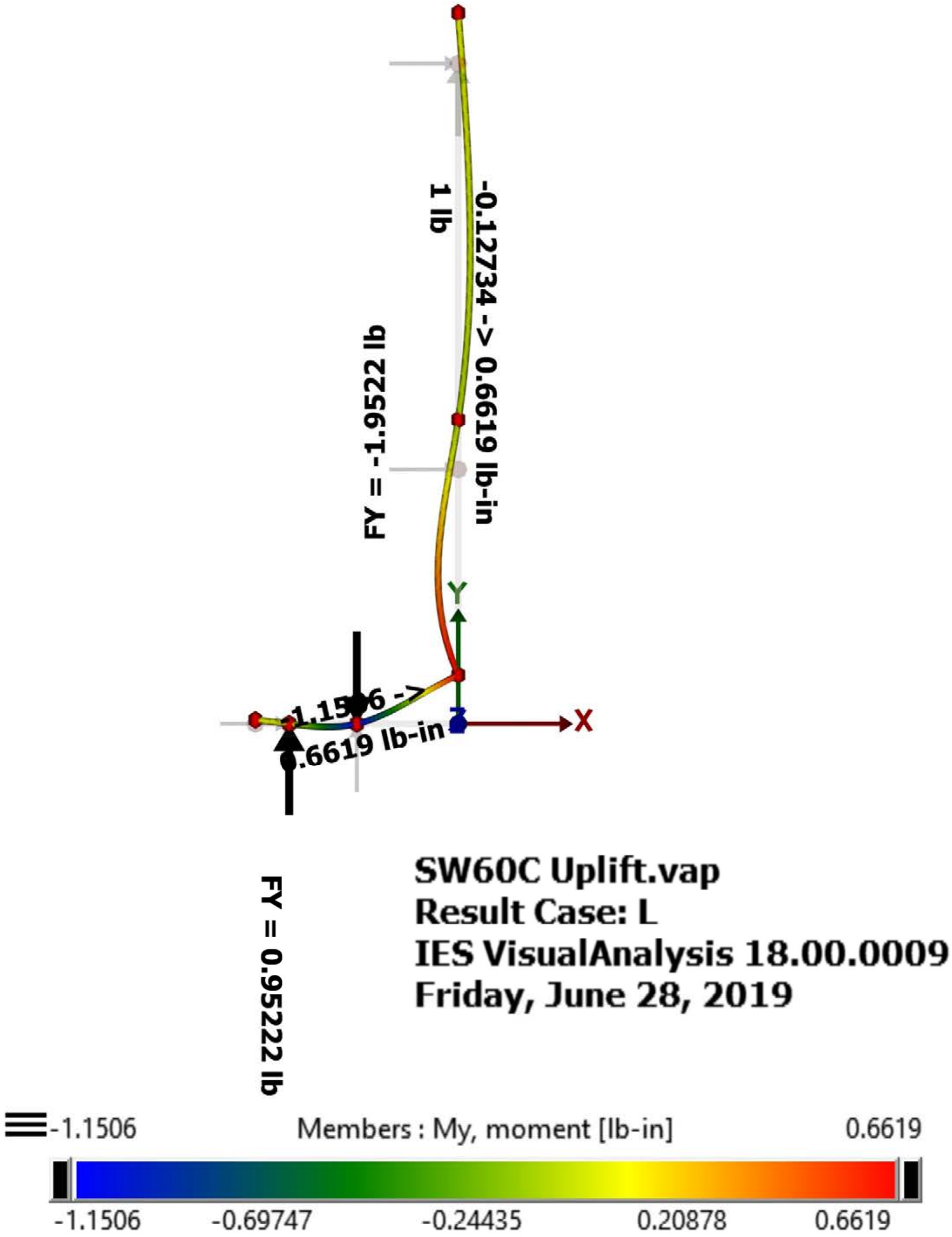

0.5 lb
0.5 lb
FY = -0.9208 lb
FY = -0.9207 lb
SW63 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
-0.5084 -> 0.398 lb-in
Y
X
-0.5084
Members : My, moment [lb-in]
0.398
-0.5084
-0.2818
-0.0552
0.1714
0.398

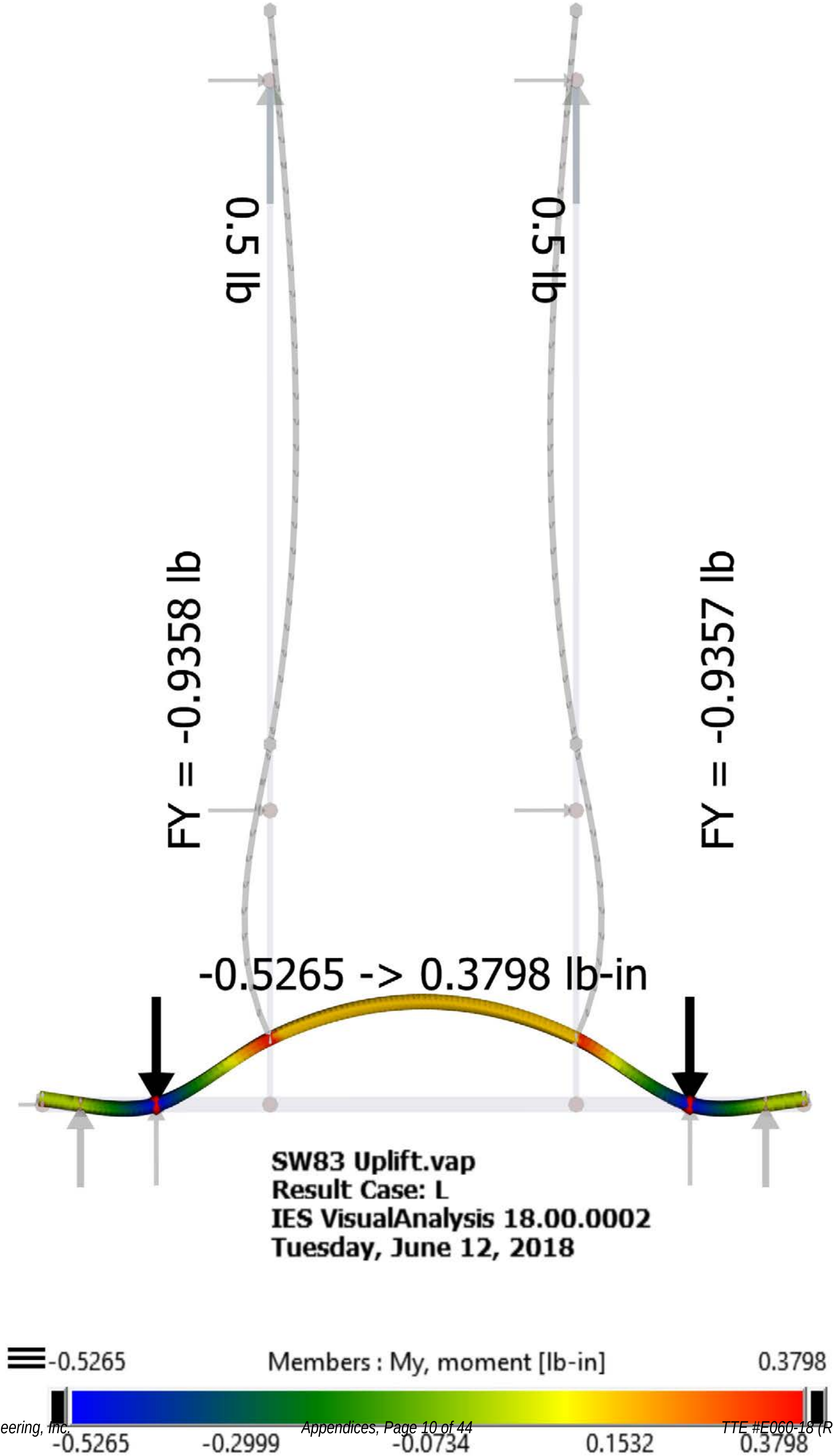

0.5 lb
0.5 lb
FY = -0.9274 lb
FY = -0.9274 lb
SW64 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
-0.5164 -> 0.39 lb-in
Y
X
-0.5164
Members : My, moment [lb-in]
0.39
-0.5164
-0.2898
-0.0632
0.1634
0.39

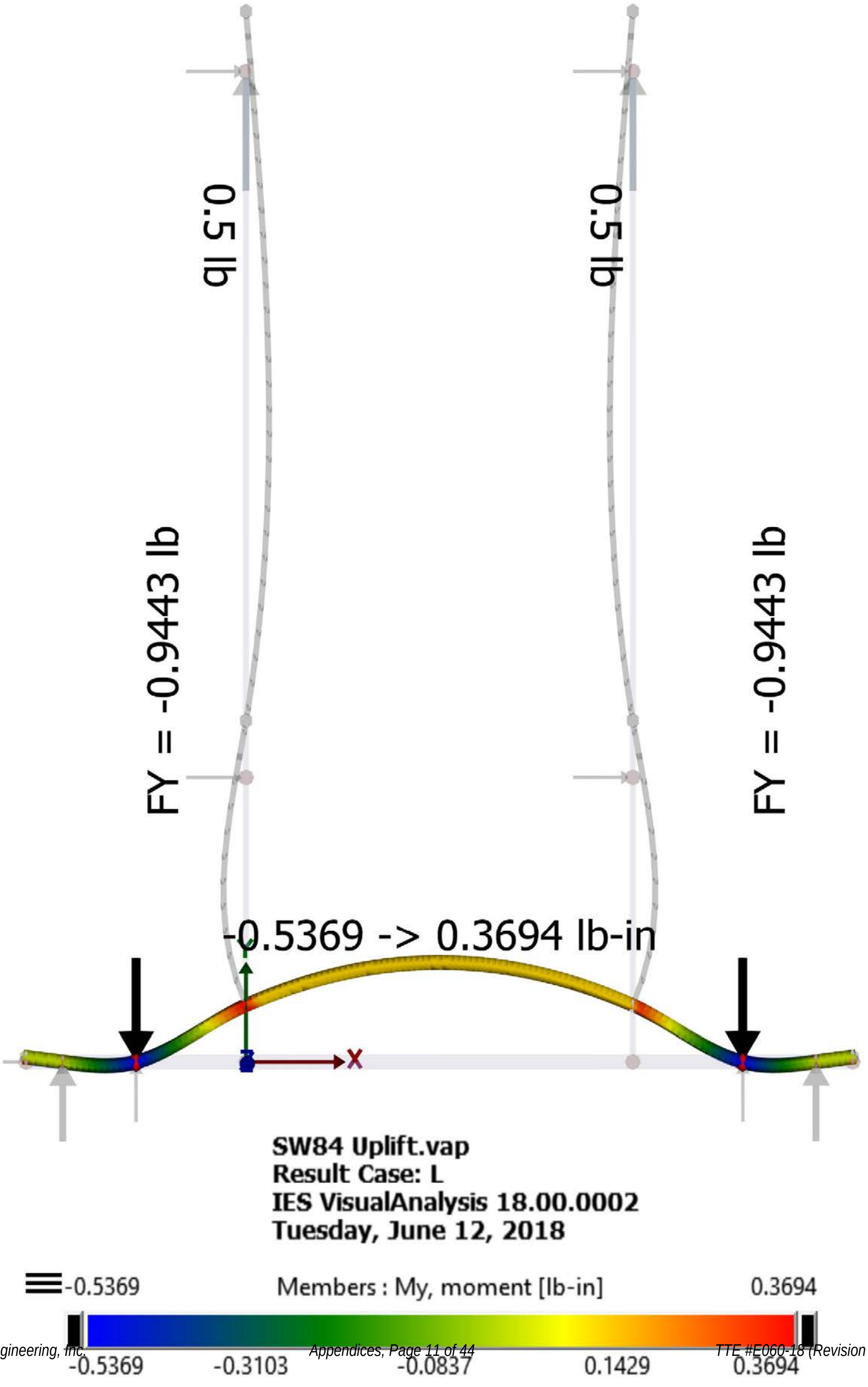

0.5 lb
0.5 lb
FY = -0.9502 lb
FY = -0.9502 lb
SW65 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Thursday, August 22, 2019
-0.544 -> 0.3623 lb-in
Y
Z
X
-0.544
Members : My, moment [lb-in]
0.3623
-0.544
-0.3174
-0.0908
0.1357
0.3623


FY = -0.9255 lb
FY = -0.9254 lb
SW66 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
-0.5141 -> 0.3923 lb-in
-0.5141
Members : My, moment [lb-in]
0.3923
-0.5141
-0.2875
-0.0609
0.1657
0.3923

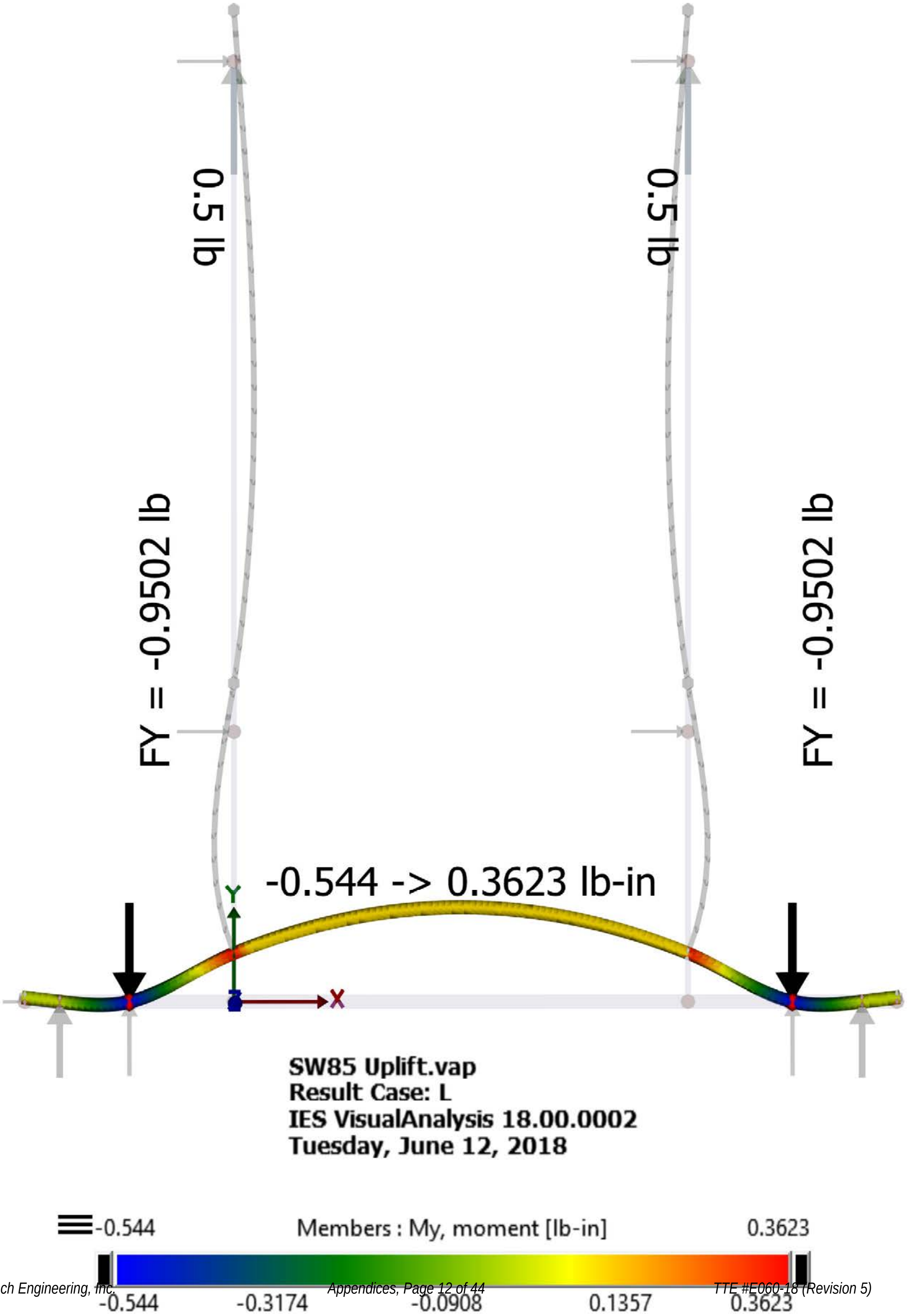

0.5 lb
-0.0542 -> 0.3559 lb-in
-0.3559 -> 0.0543 lb-in
FY = -0.9555 lb
FY = -0.9554 lb
SW60 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
-0.55 4 ->
0.3559 lb-in
-0.55 03 ->
0.3559 lb-in
Y
X
-0.5504
Members : My, moment [lb-in]
0.3559
-0.5504
-0.3238
-0.0972
0.1294
0.3559


1 lb
-0.12734 -> 0.6619 lb-in
FY = -1.9522 lb
Y
X
1.1506 ->
.6619 lb-in
FY = 0.95222 lb
SW60C Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Friday, June 28, 2019
-1.1506
Members : My, moment [lb-in]
0.6619
-1.1506
-0.69747
-0.24435
0.20878
0.6619

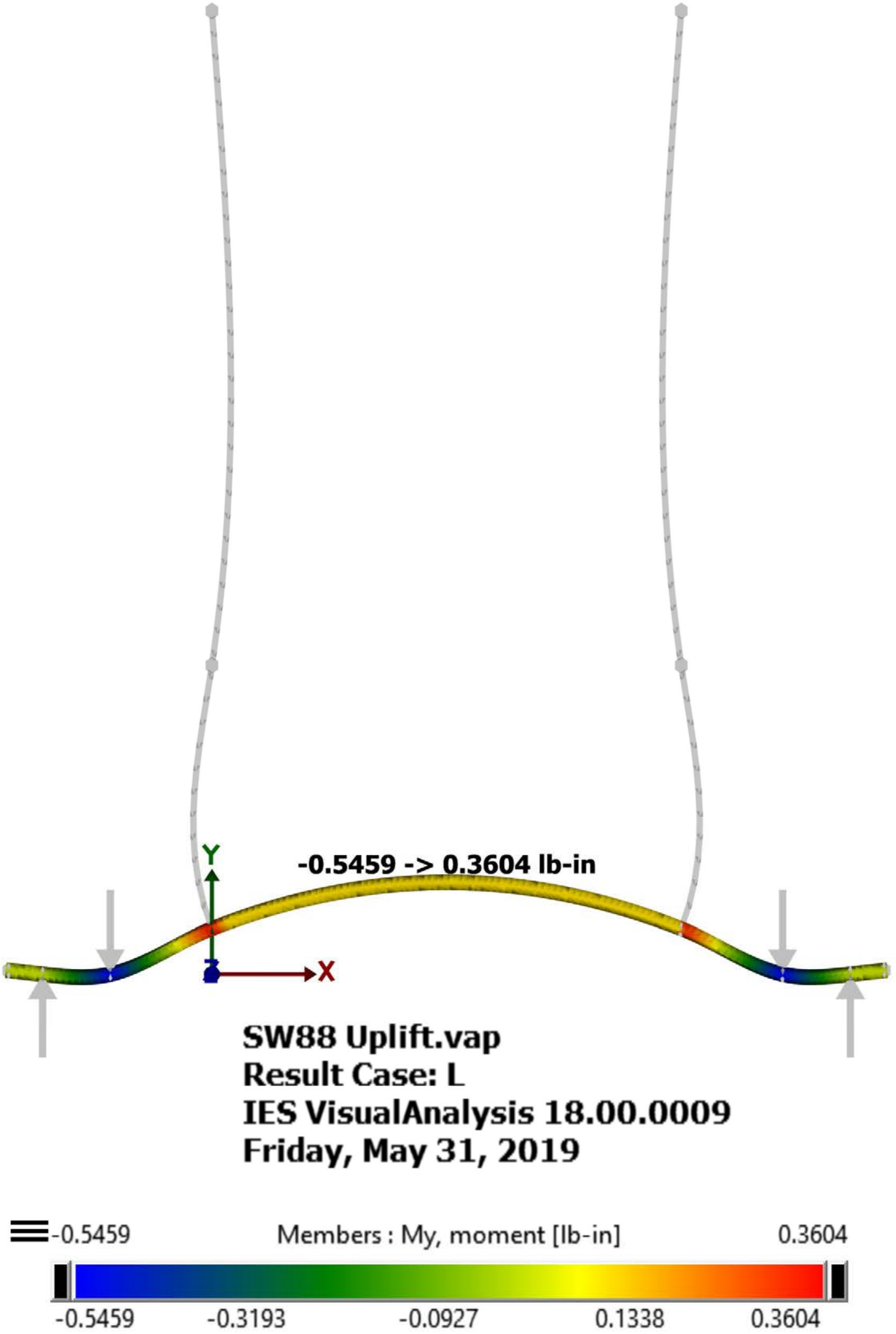

0.5 lb
0.5 lb
FY = -0.9358 lb
FY = -0.9357 lb
-0.5265 -> 0.3798 lb-in
SW83 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
-0.5265
Members : My, moment [lb-in]
0.3798
-0.5265
-0.2999
-0.0734
0.1532
0.3798

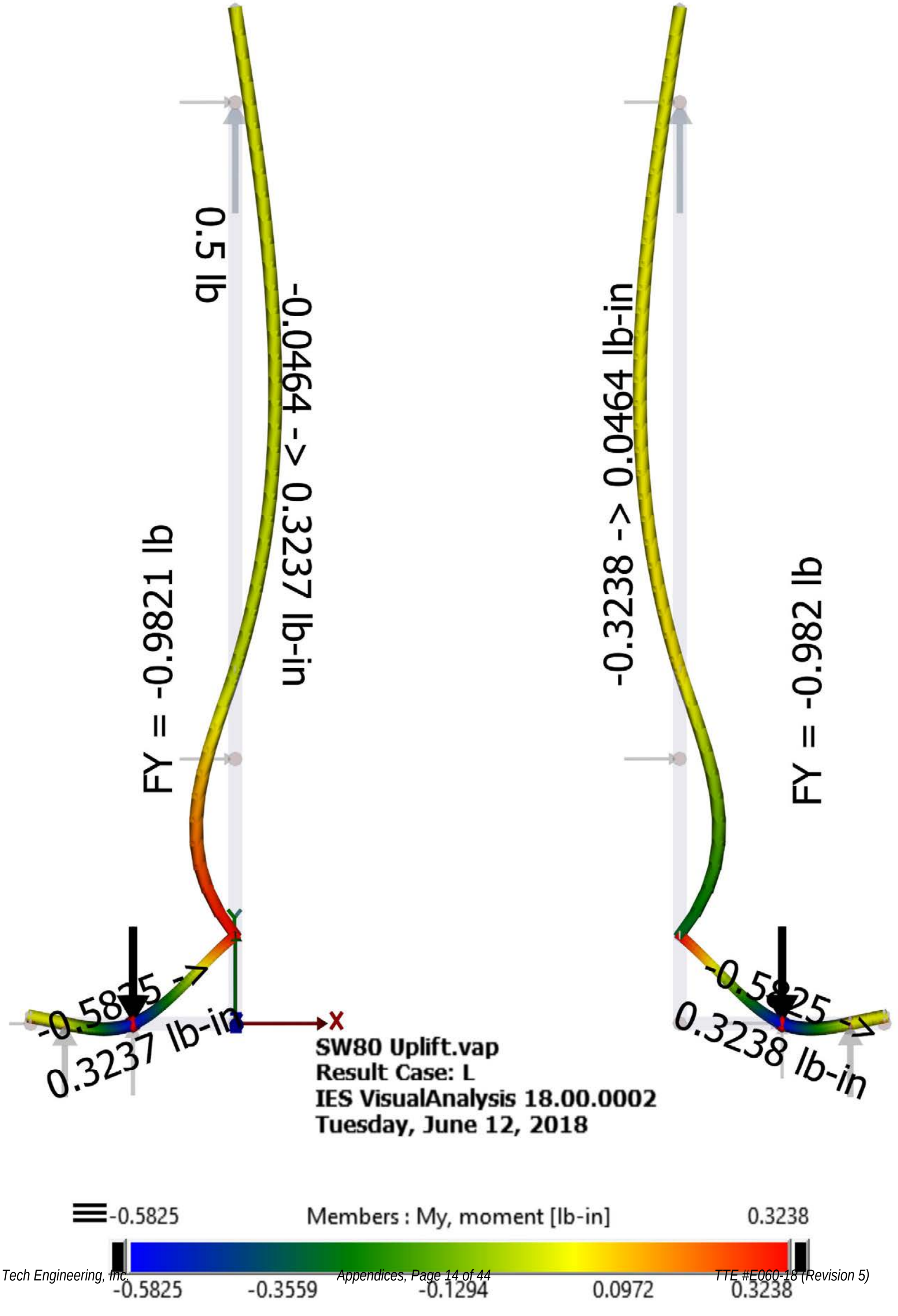


SW84 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



SW85 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018


Y
X
Z
-0.5459 -> 0.3604 lb-in
SW88 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Friday, May 31, 2019
-0.5459
Members : My, moment [lb-in]
0.3604
-0.5459
-0.3193
-0.0927
0.1338
0.3604

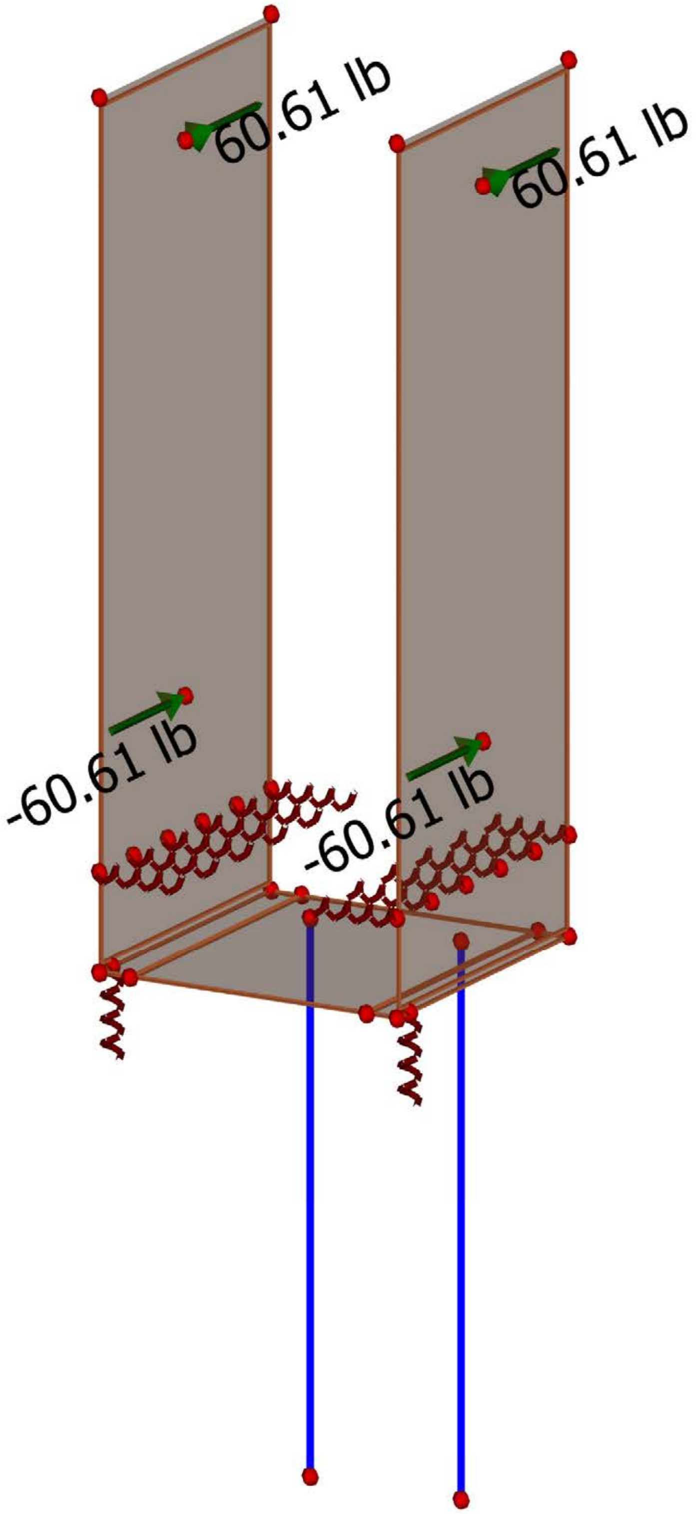


SW80 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018

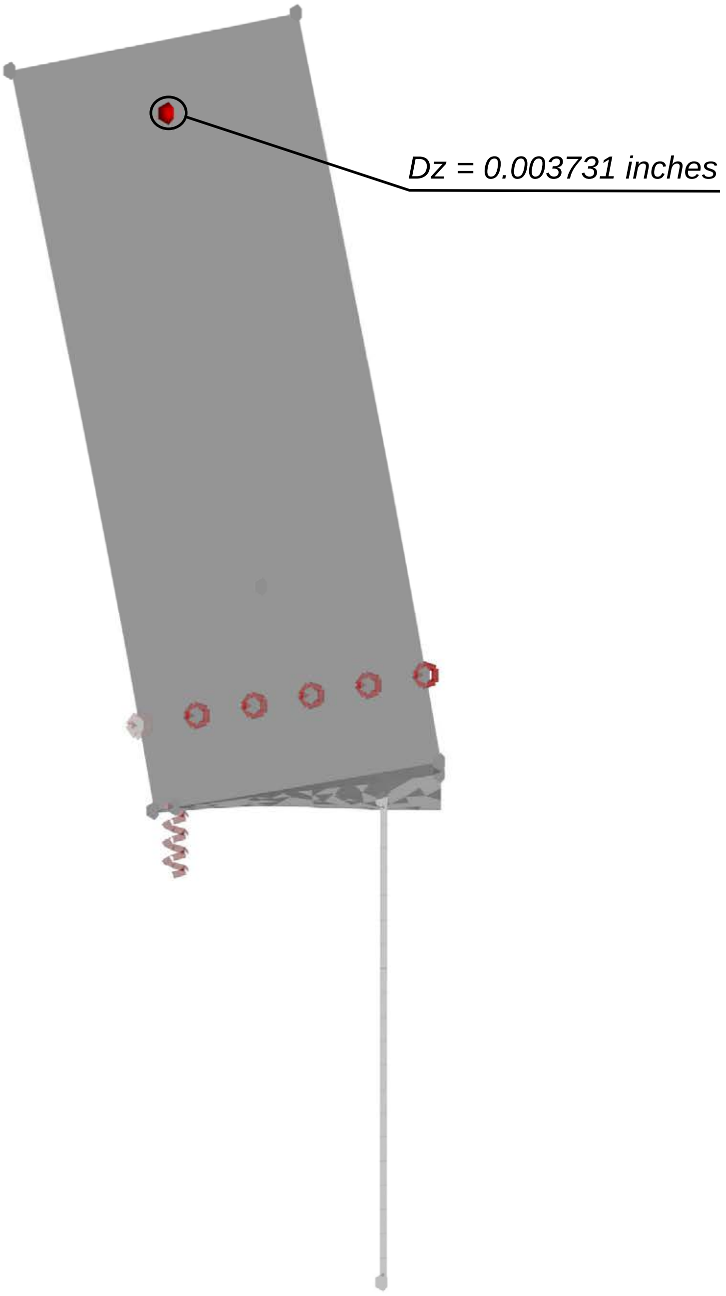
APPENDIX B

Structural Models of Sturdi-Wall Plus Brackets (Finite Element Analysis) To Determine Rotational Stiffness and the Maximum Bending Moment (Unity) In Steel Bracket

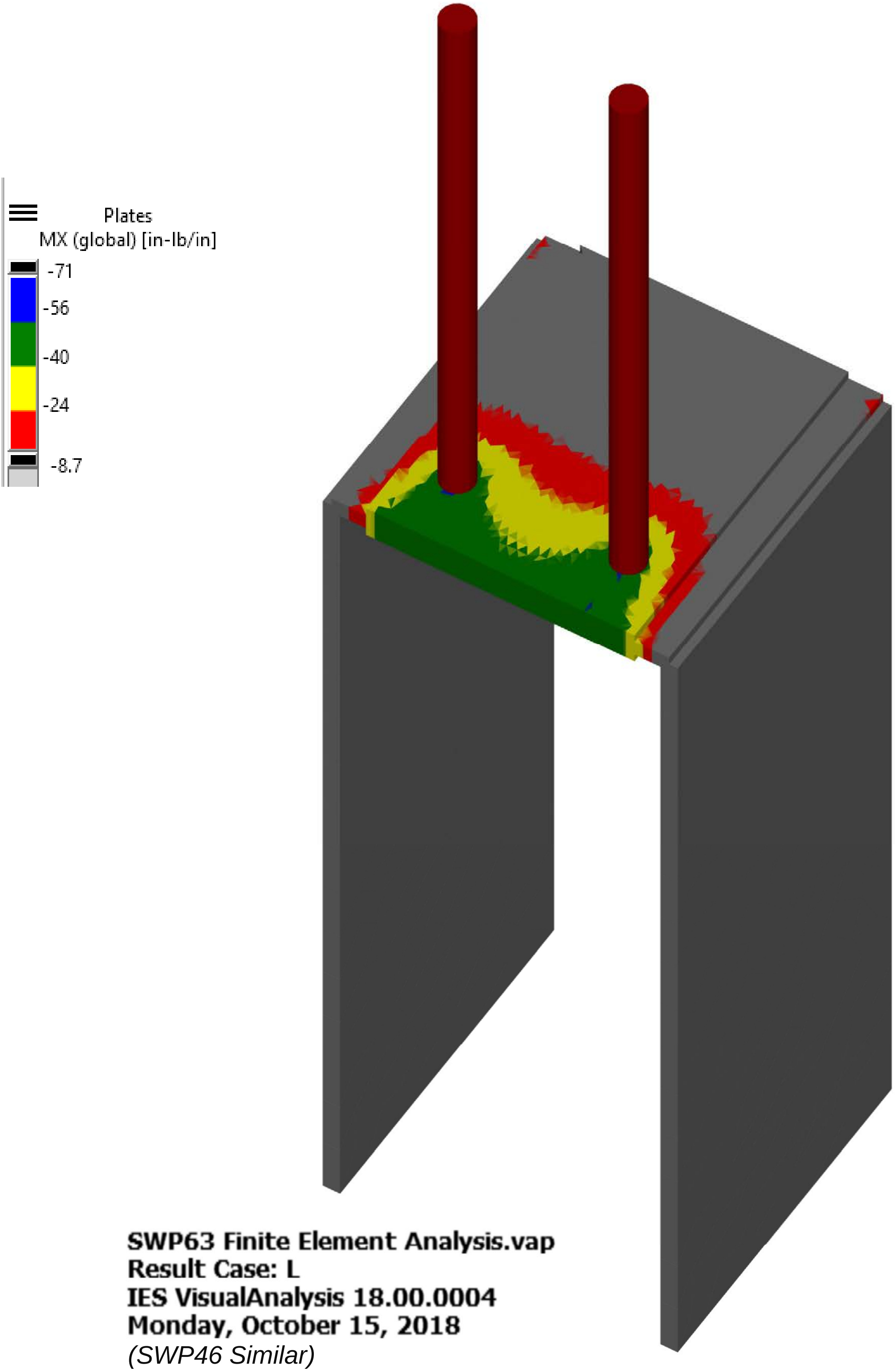
Visual Analysis by IES, Inc
Version 18



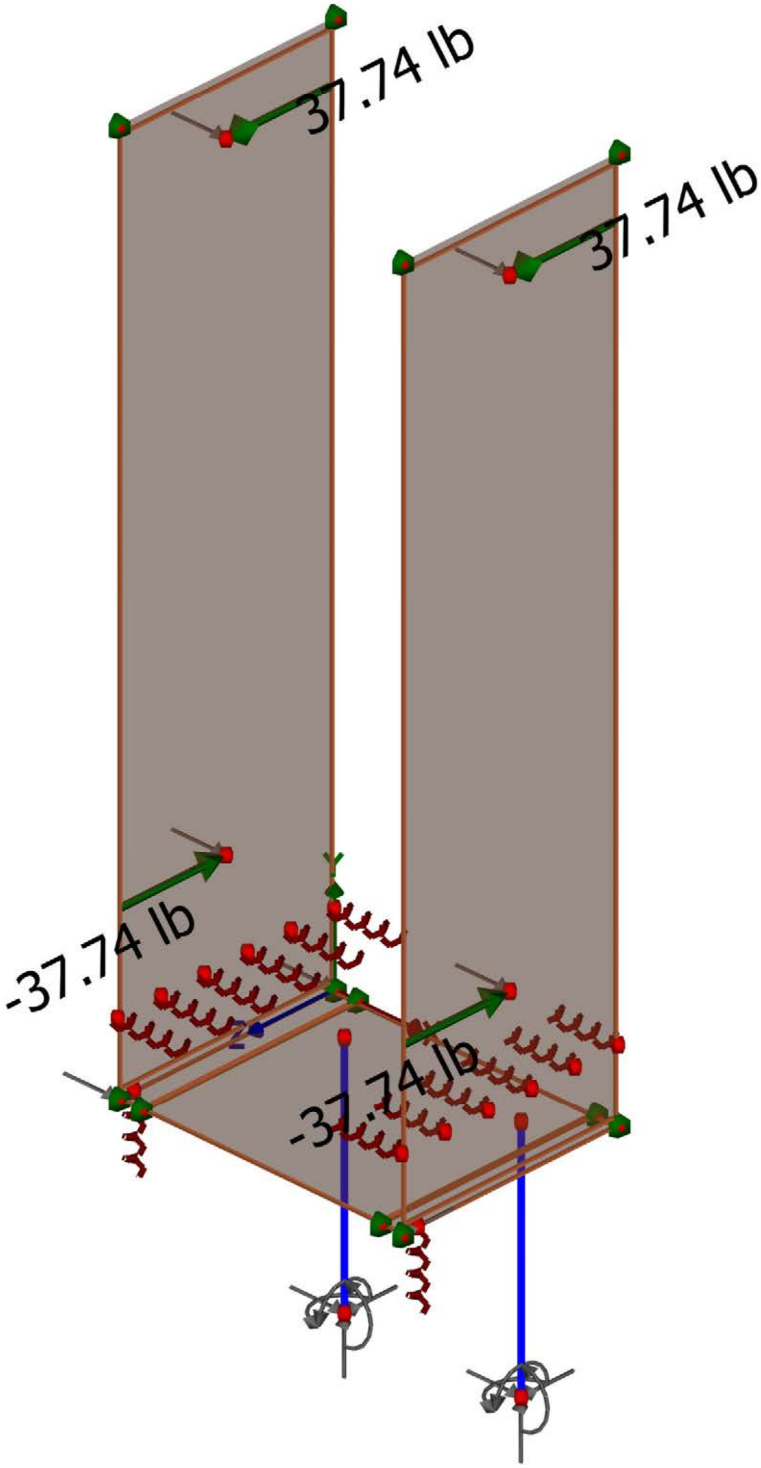
SWP63 Finite Element Analysis.vap
Service Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
(SWP46 Similar)



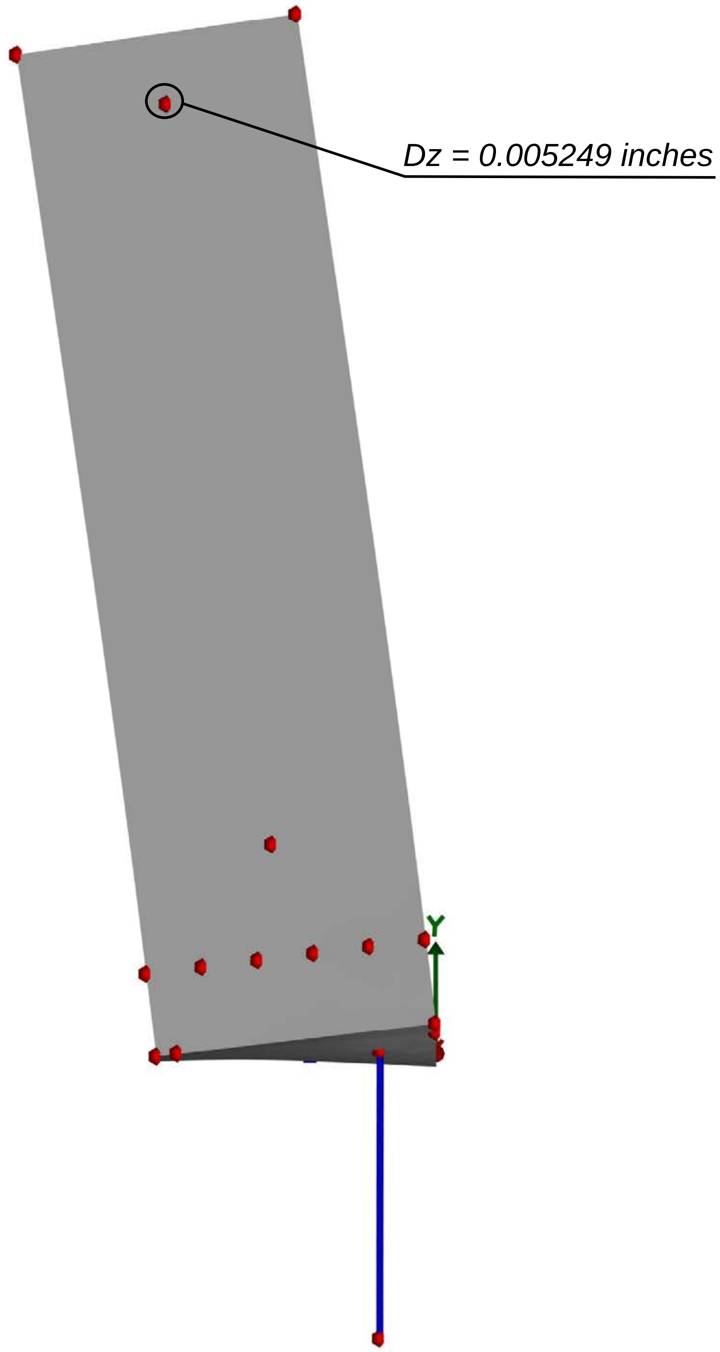
SWP63 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
(SWP46 Similar)



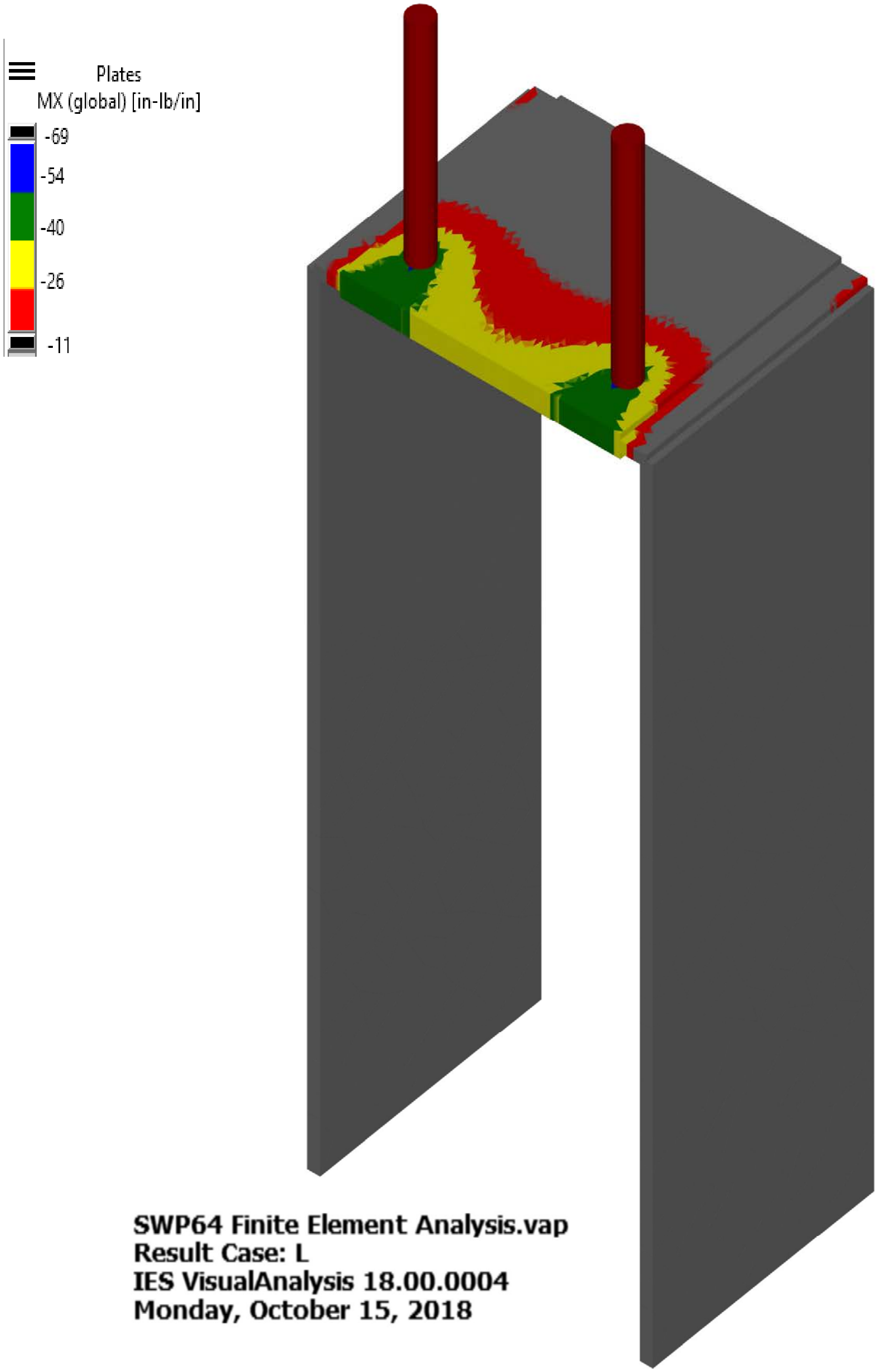
SWP63 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0004
Monday, October 15, 2018
(SWP46 Similar)



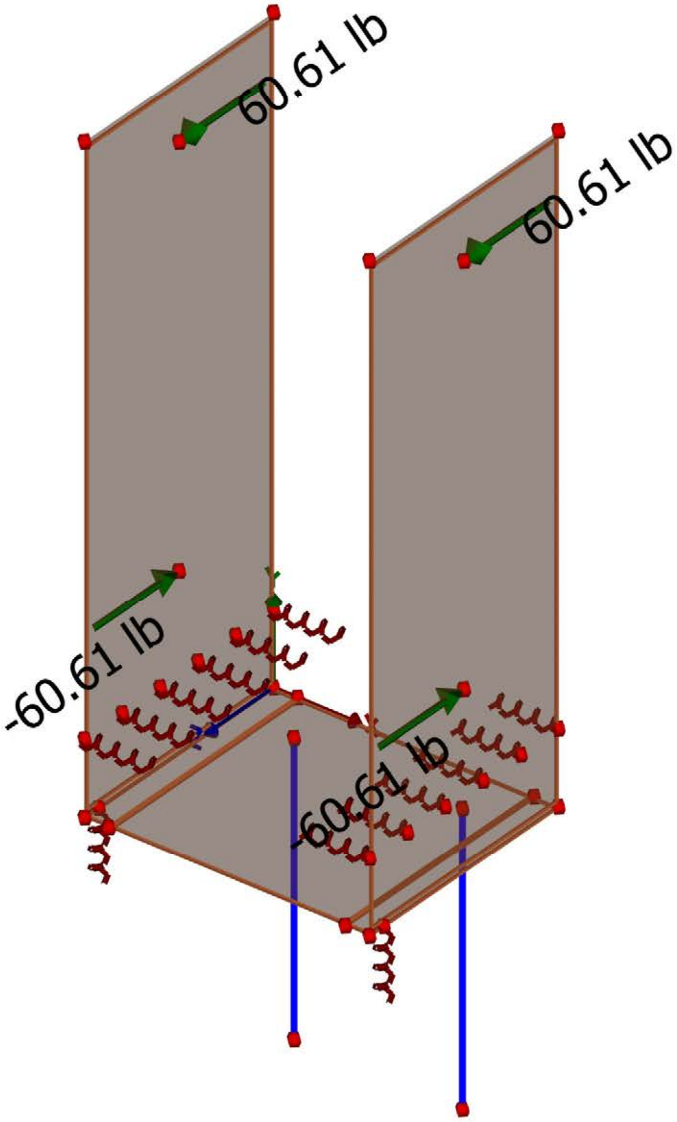
SWP64 Finite Element Analysis.vap
Service Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



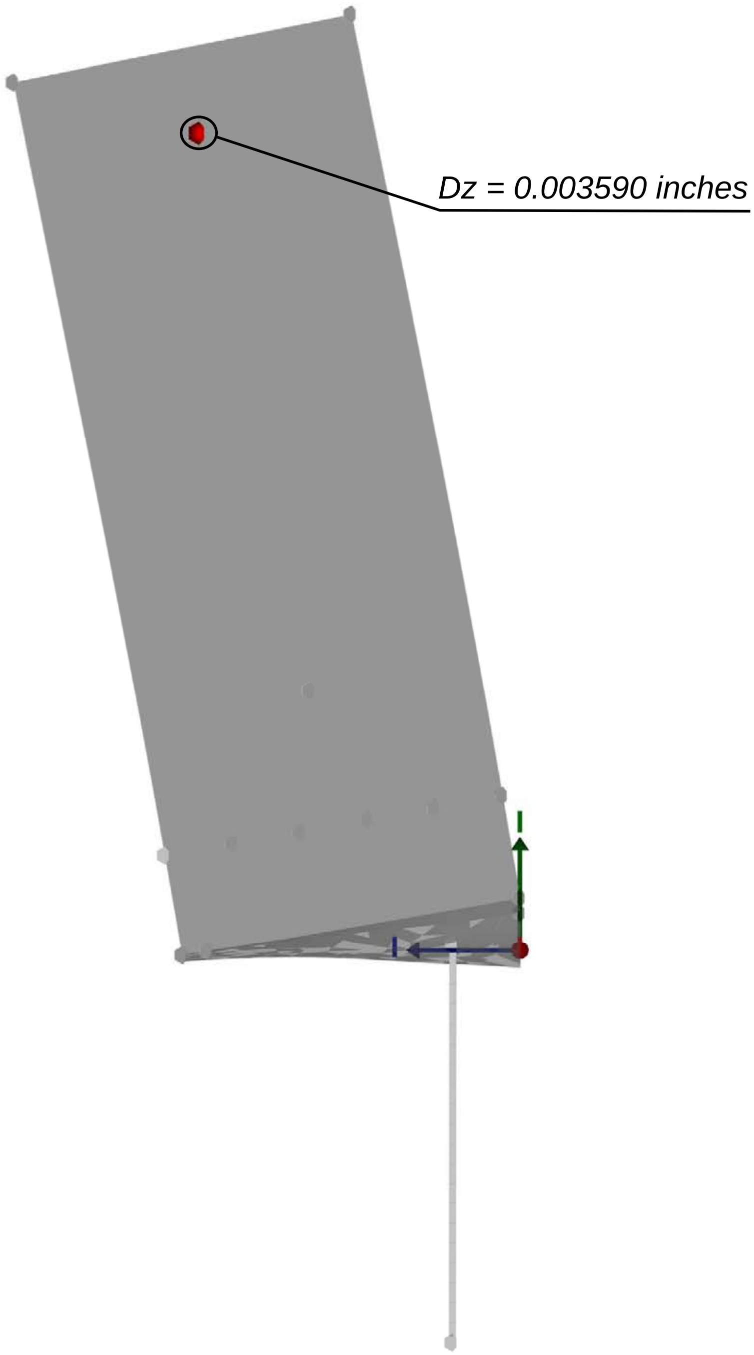
SWP64 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



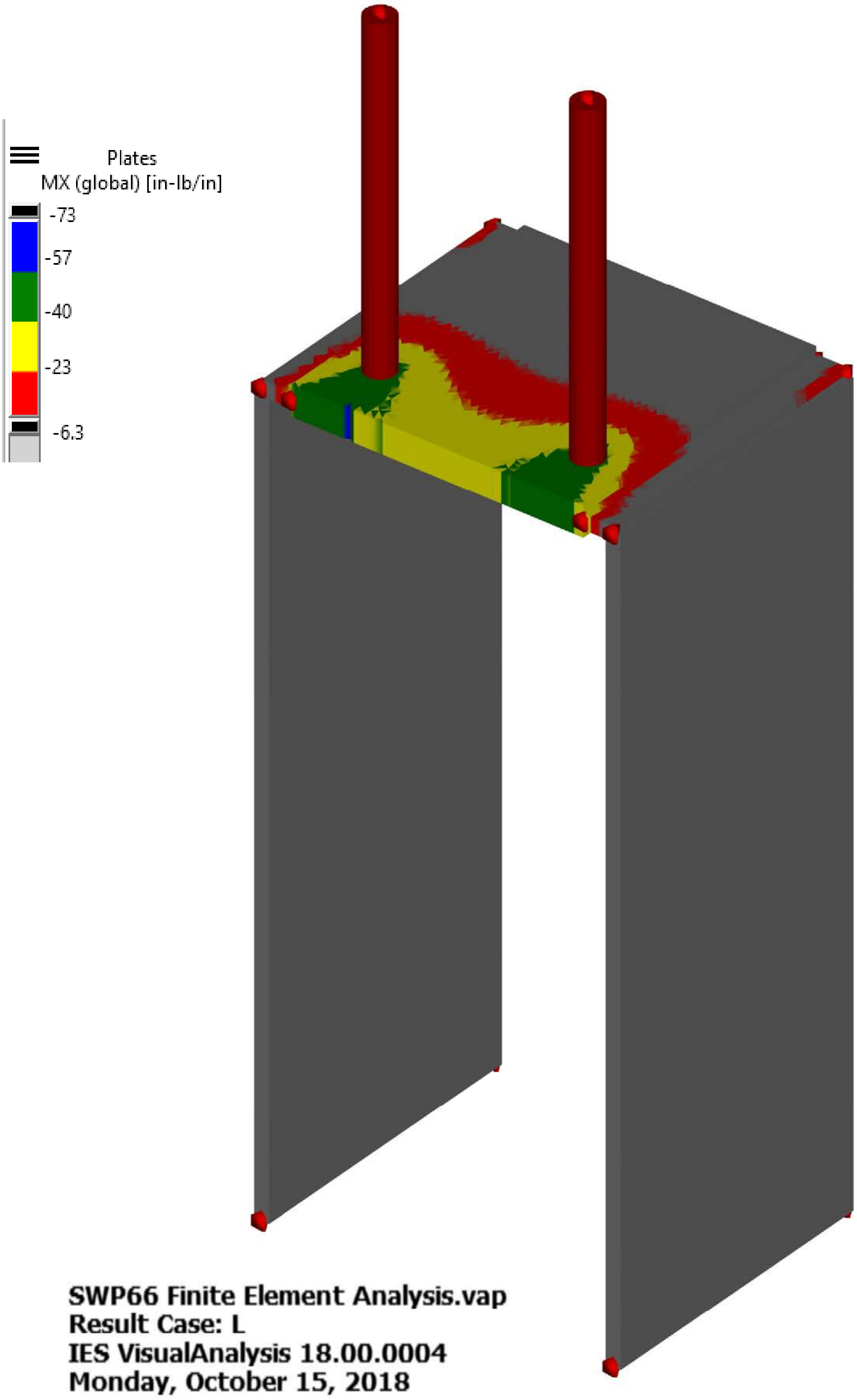
SWP64 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0004
Monday, October 15, 2018



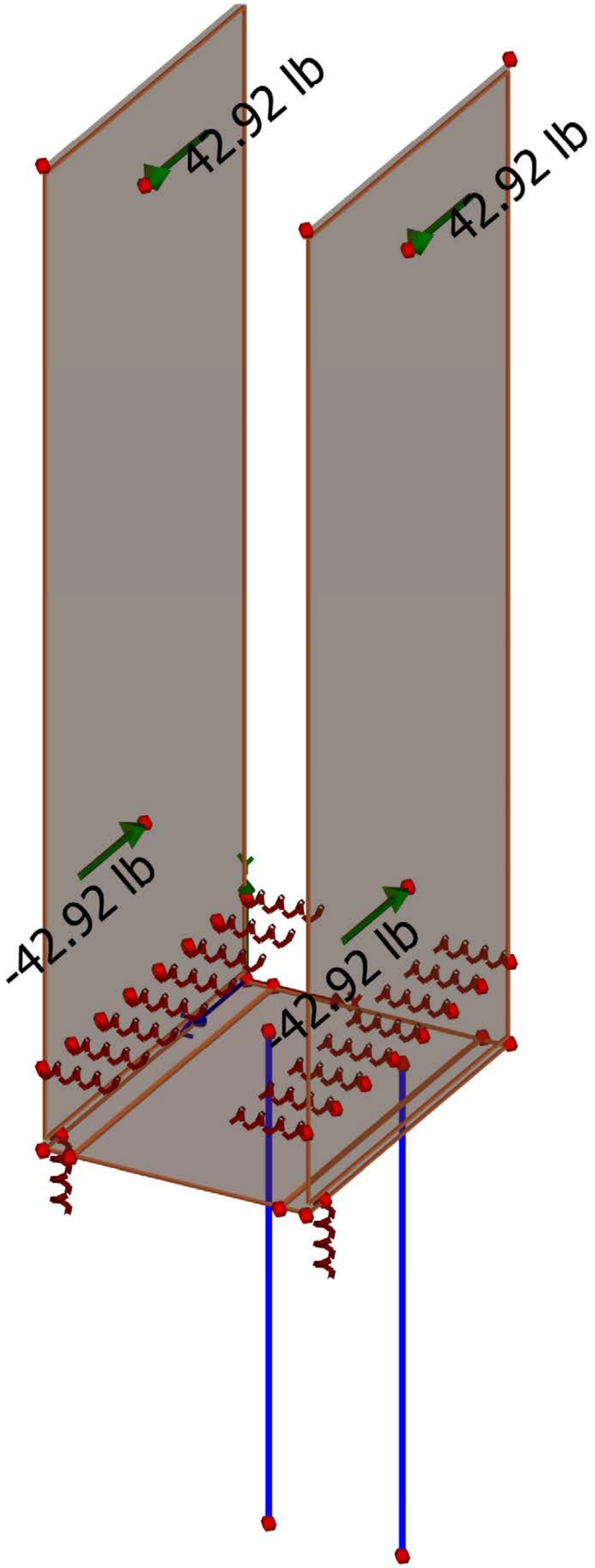
SWP66 Finite Element Analysis.vap
Service Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



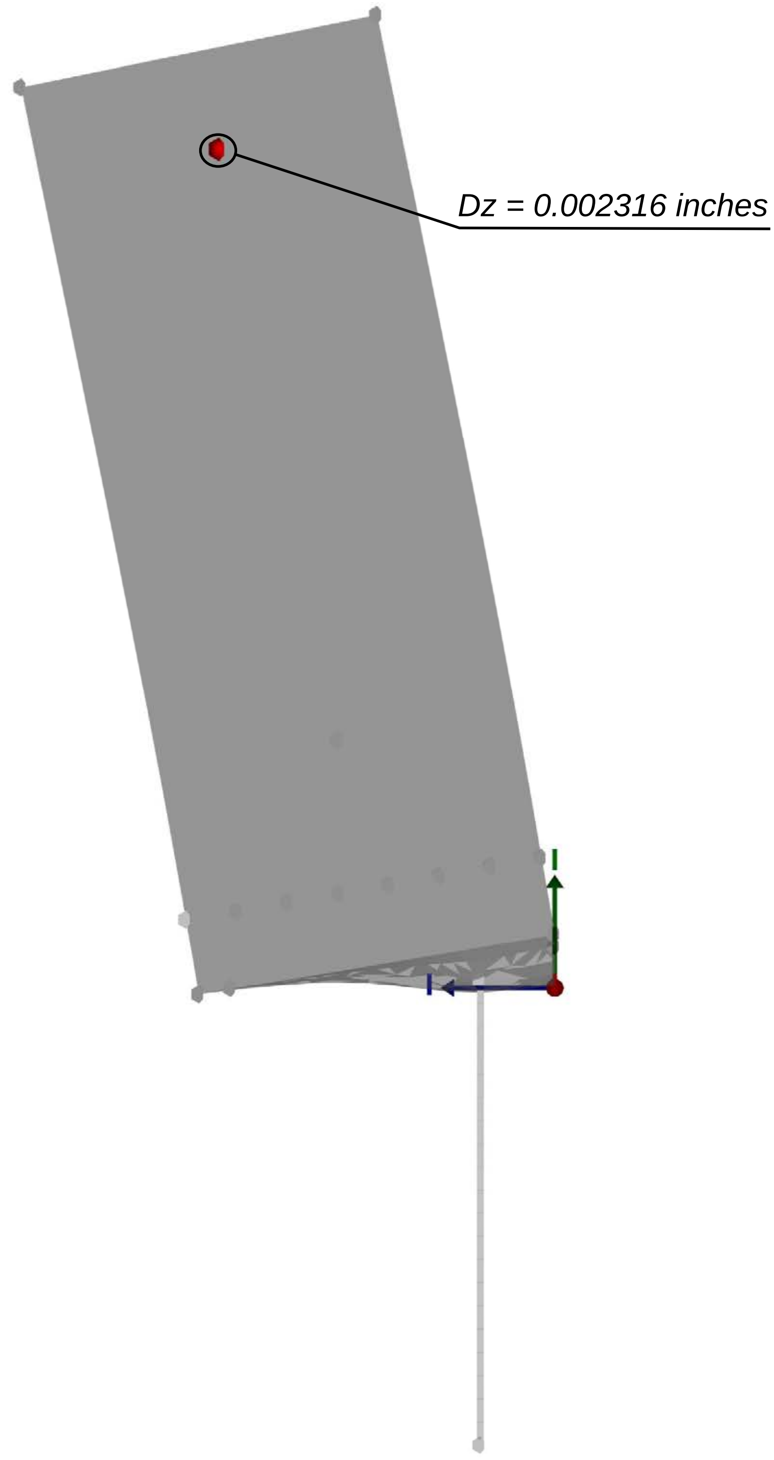
SWP66 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



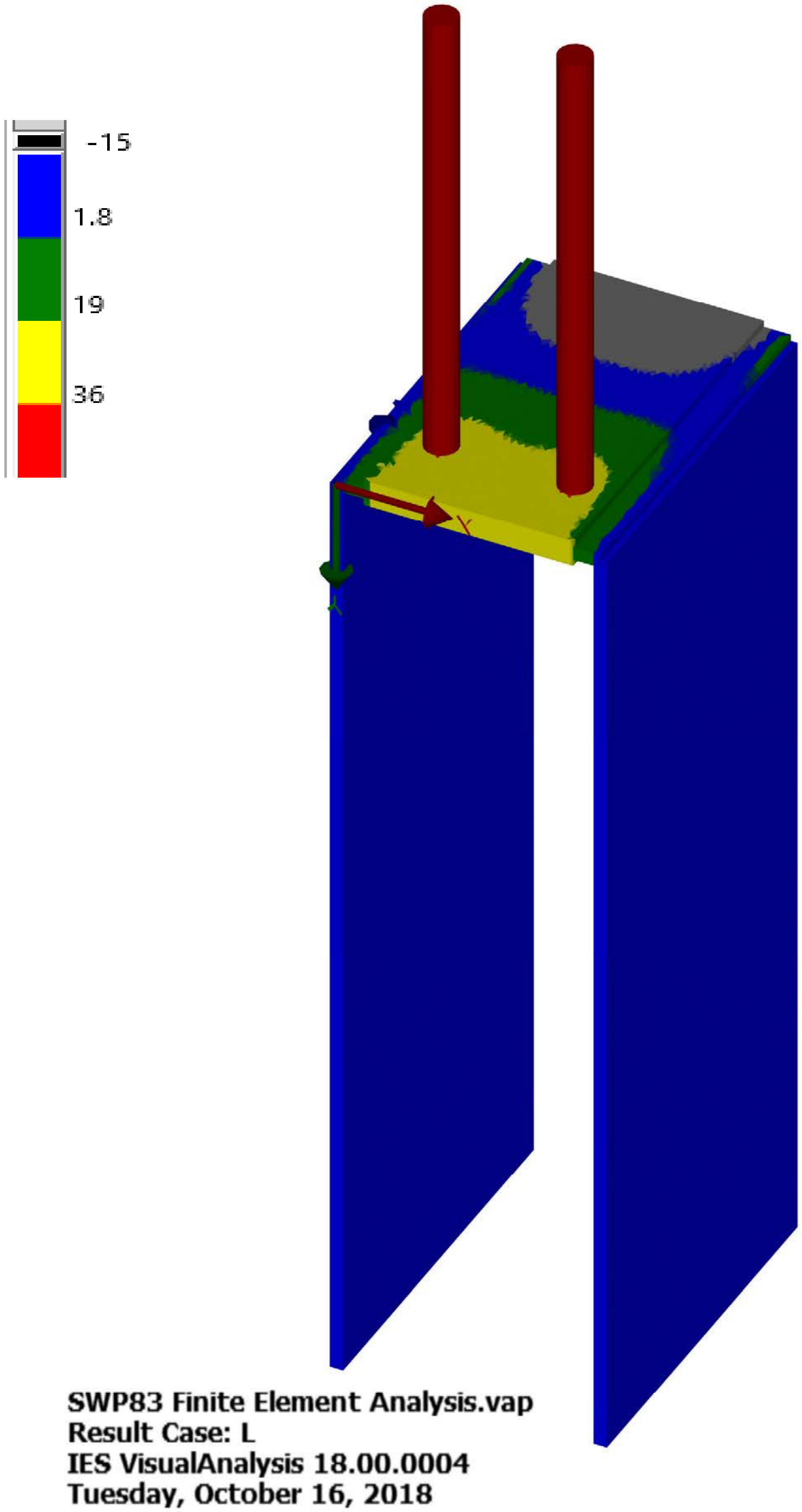
SWP66 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0004
Monday, October 15, 2018



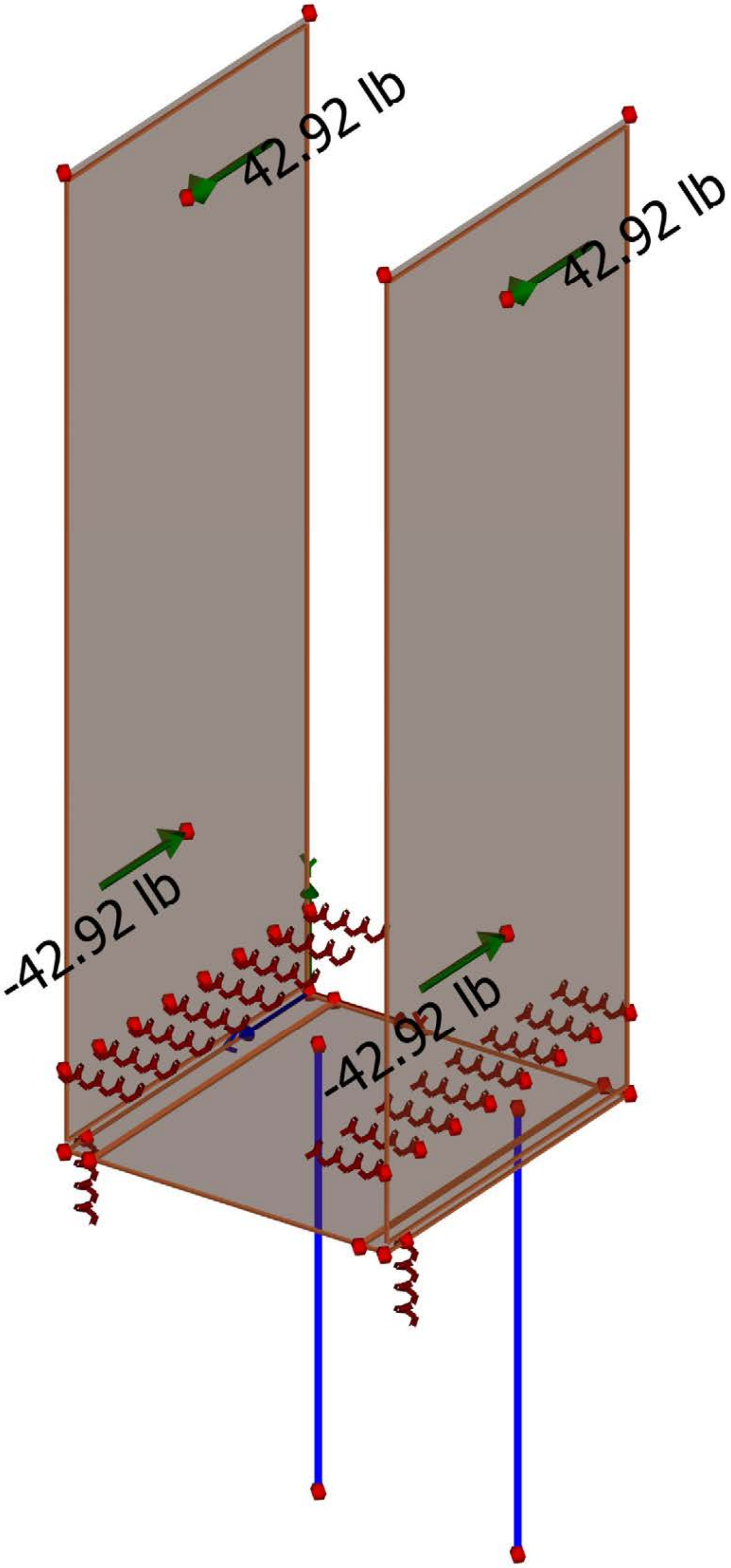
SWP83 Finite Element Analysis.vap
Service Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



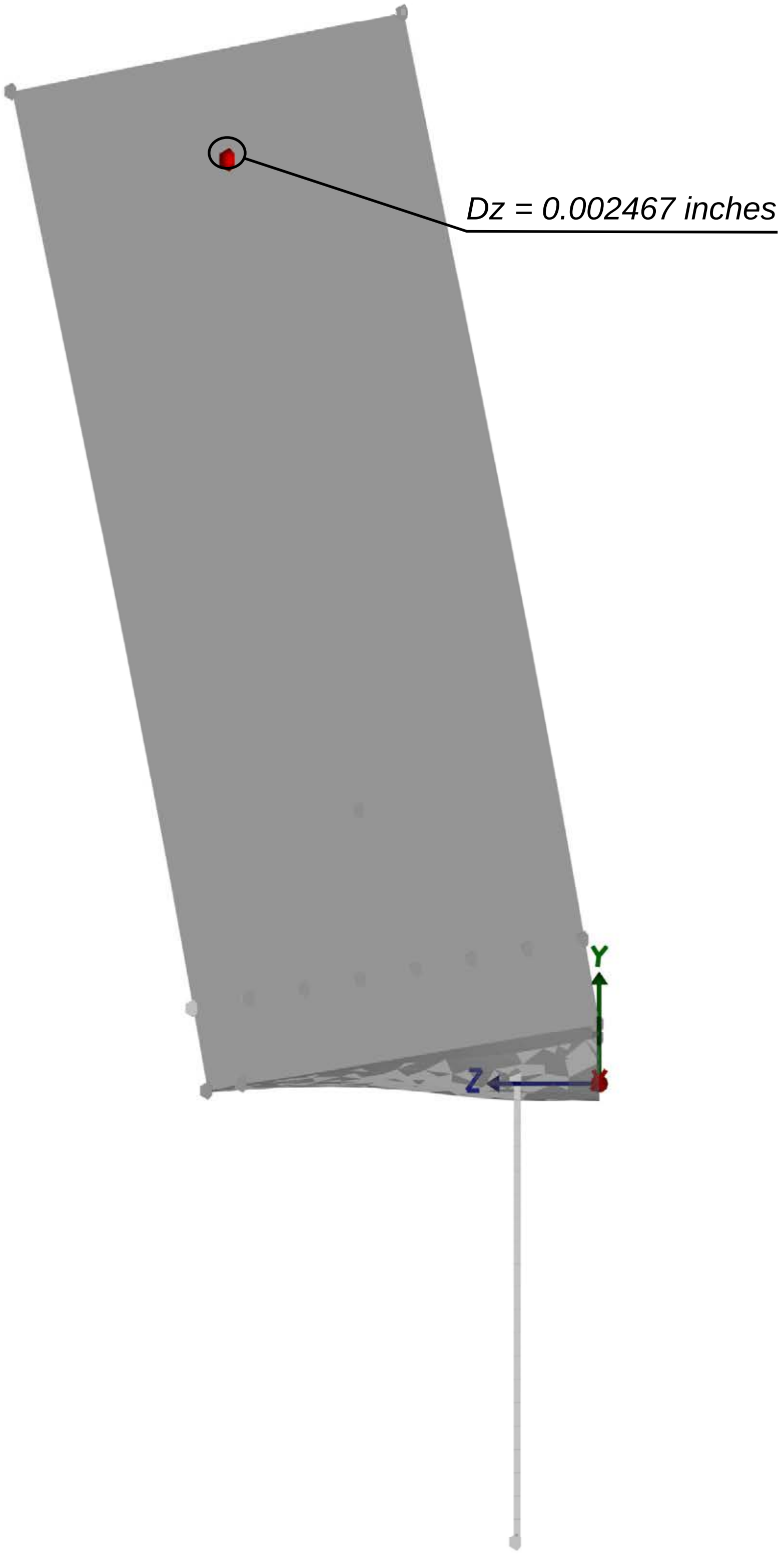
SWP83 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



SWP83 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0004
Tuesday, October 16, 2018

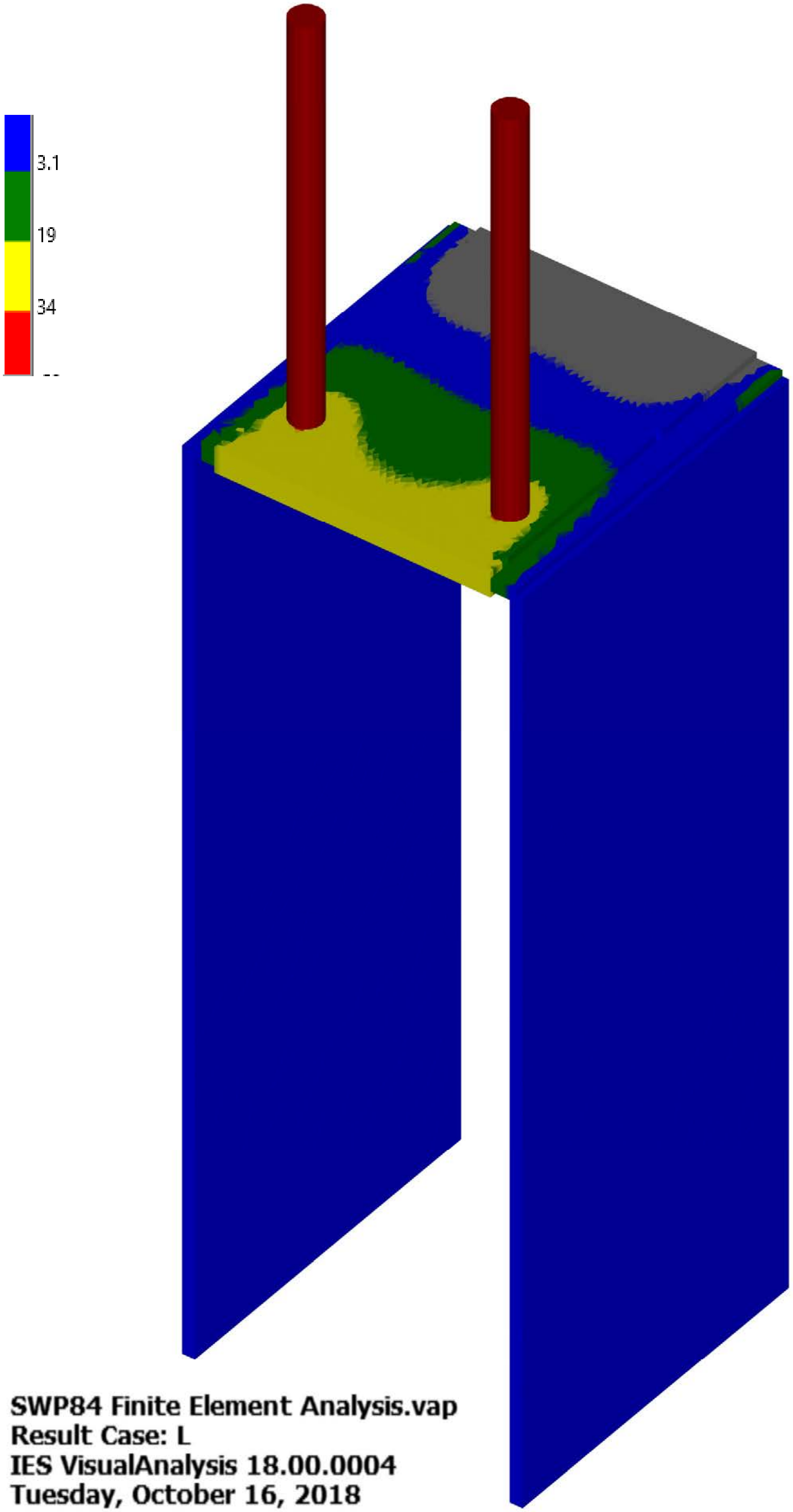


SWP84 Finite Element Analysis.vap
Service Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018

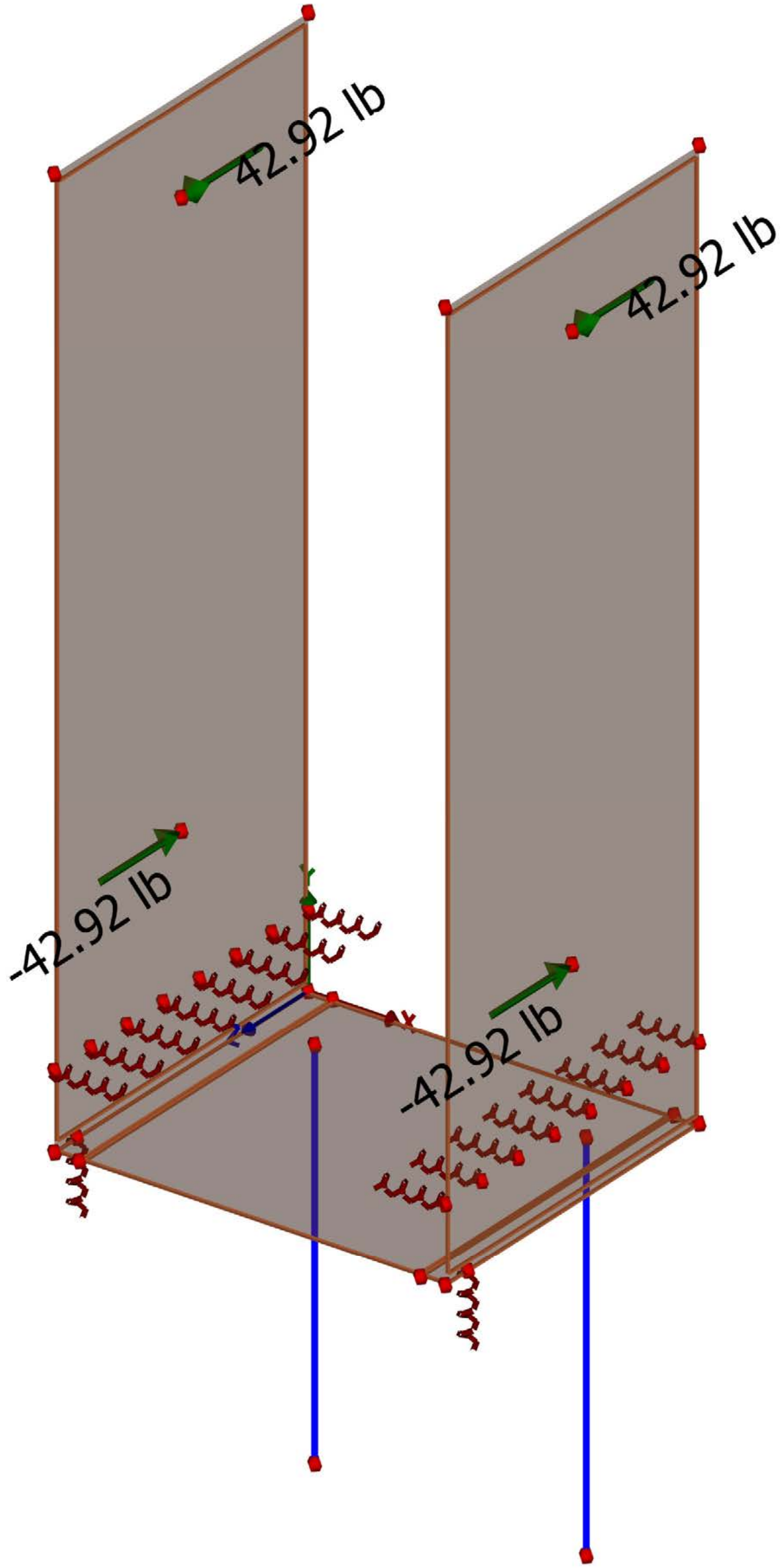


Dz = 0.002467 inches

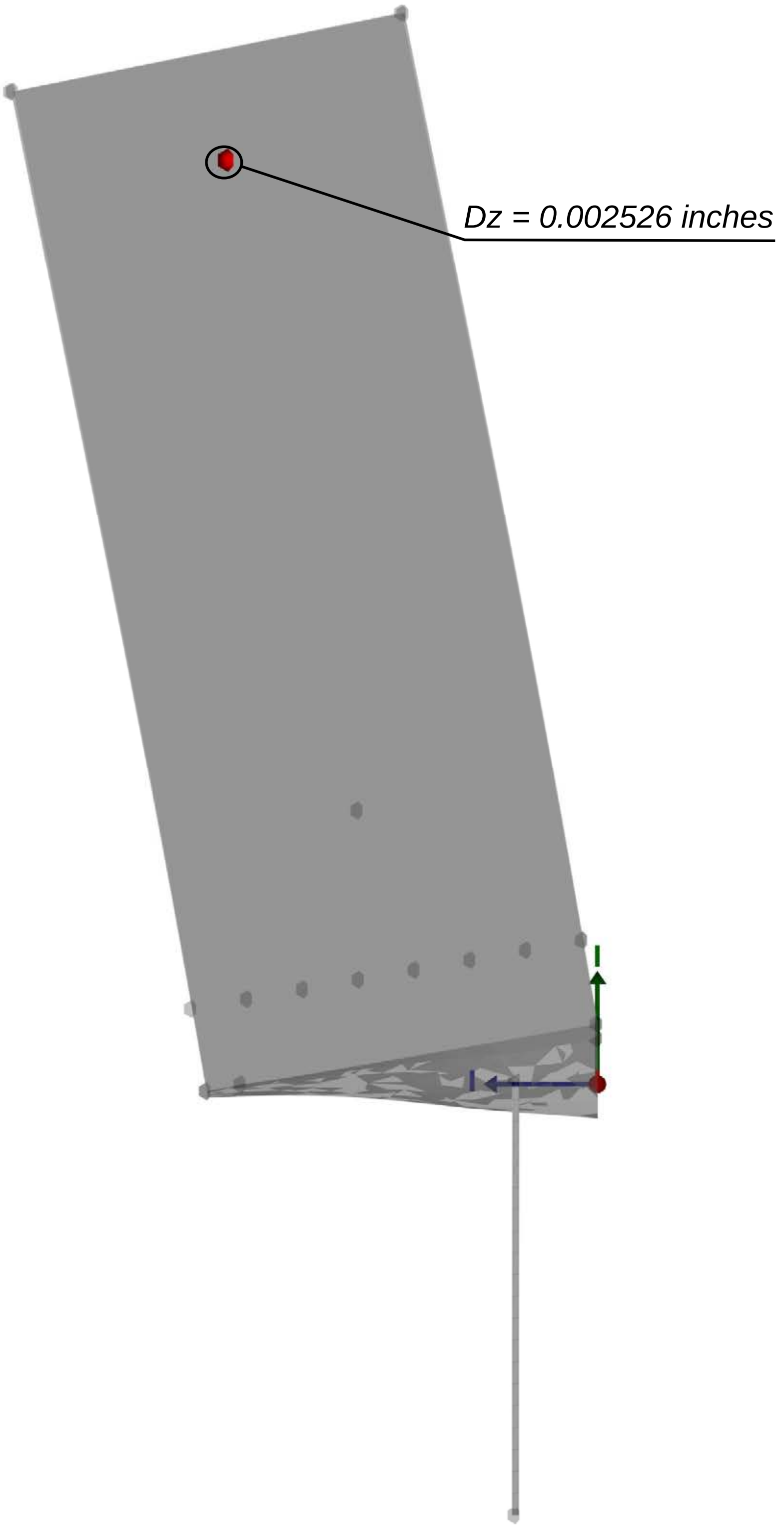
SWP84 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



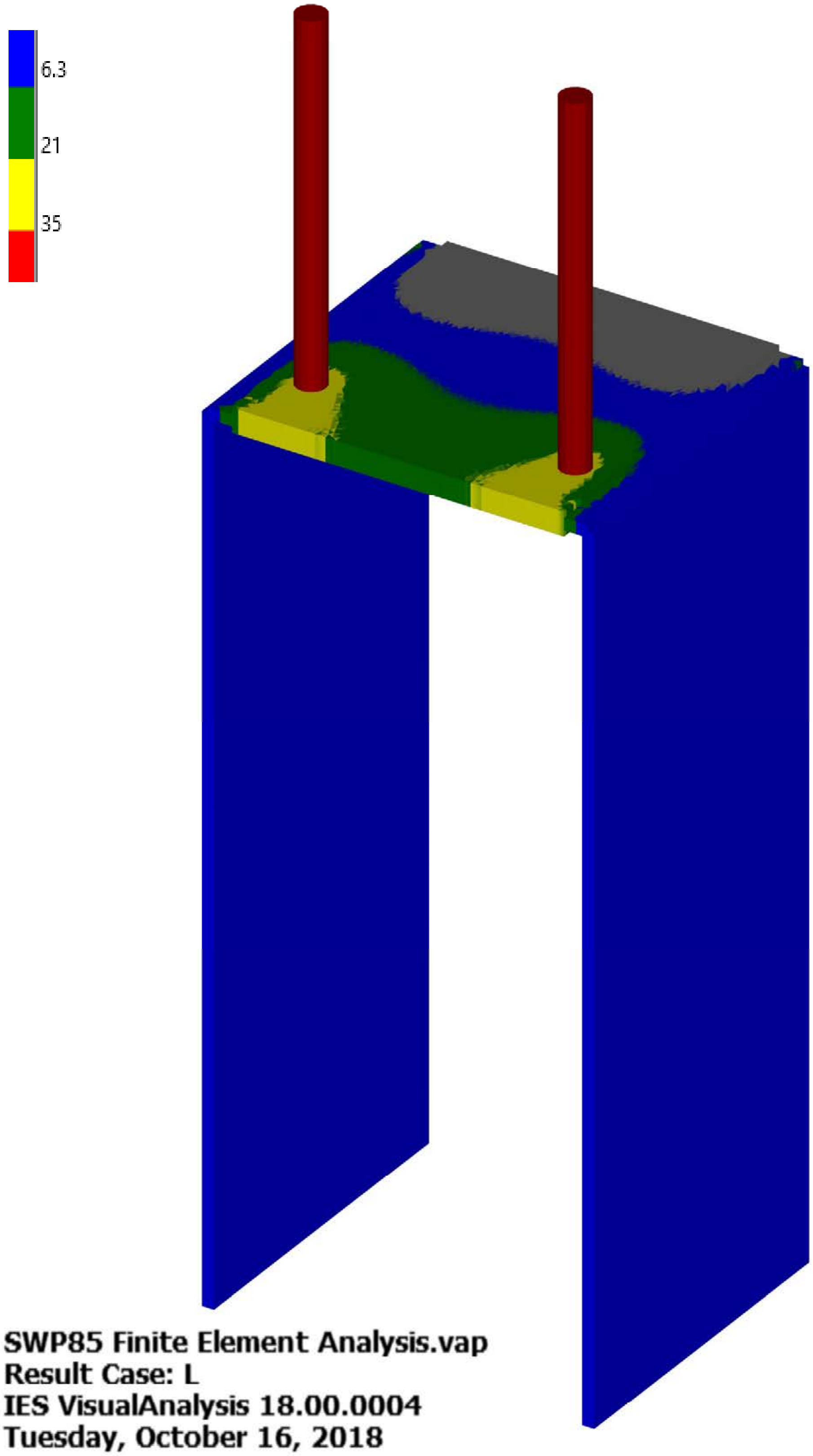
SWP84 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0004
Tuesday, October 16, 2018



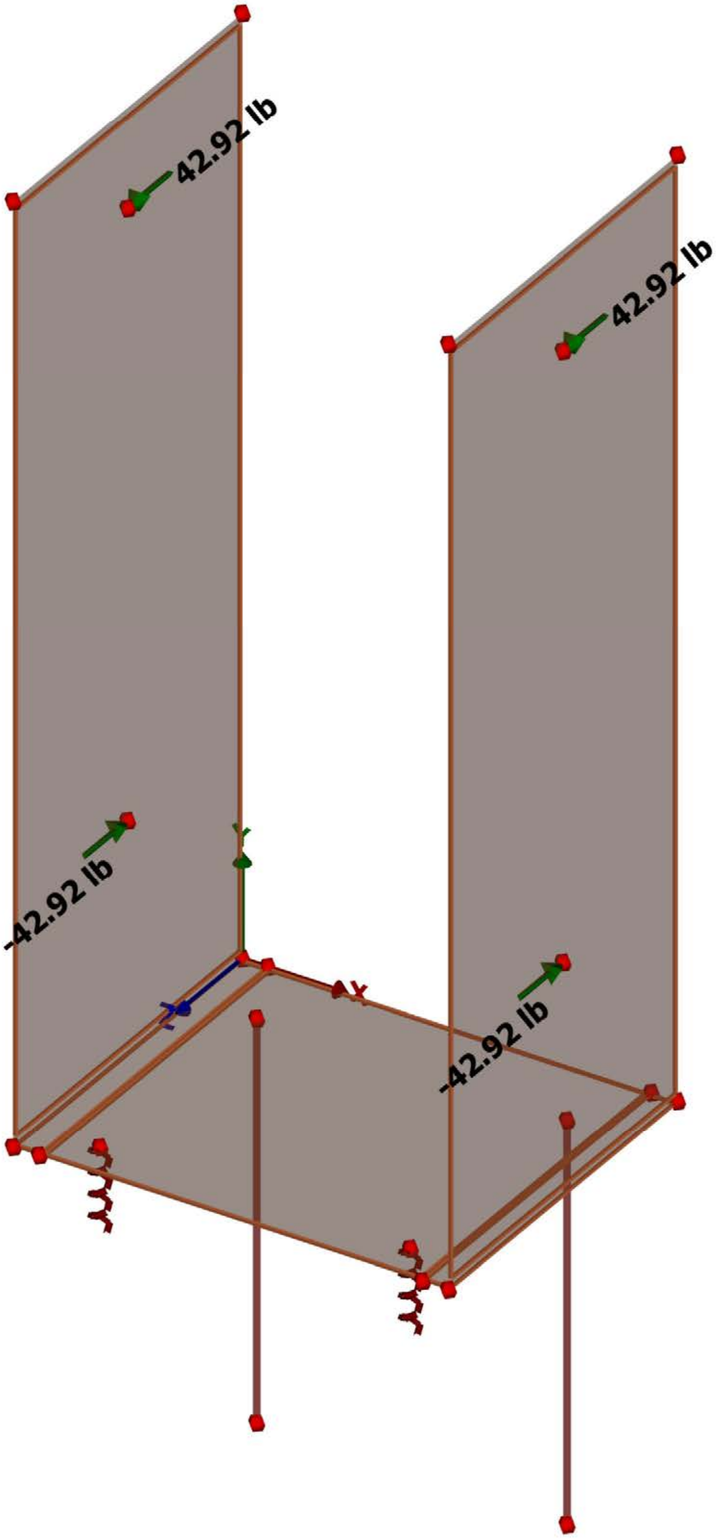
SWP85 Finite Element Analysis.vap
Service Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



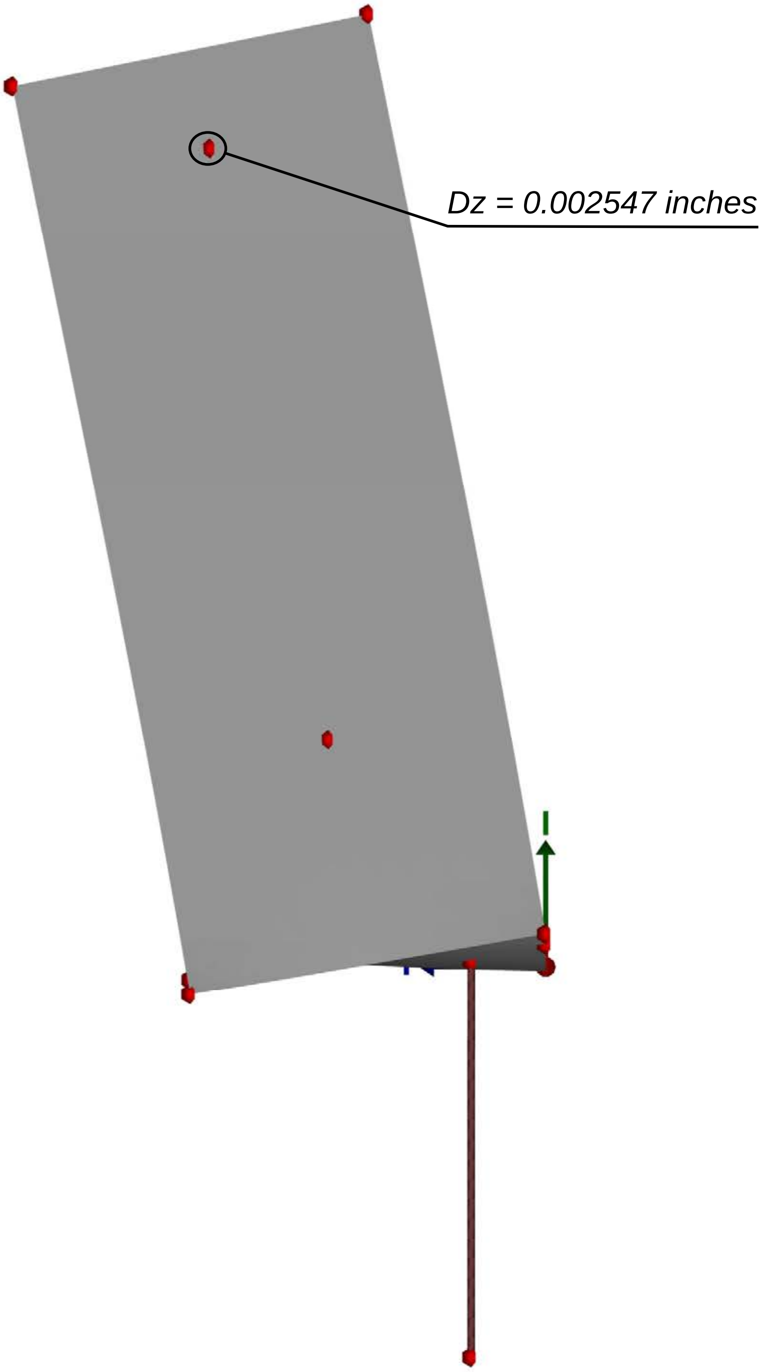
SWP85 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



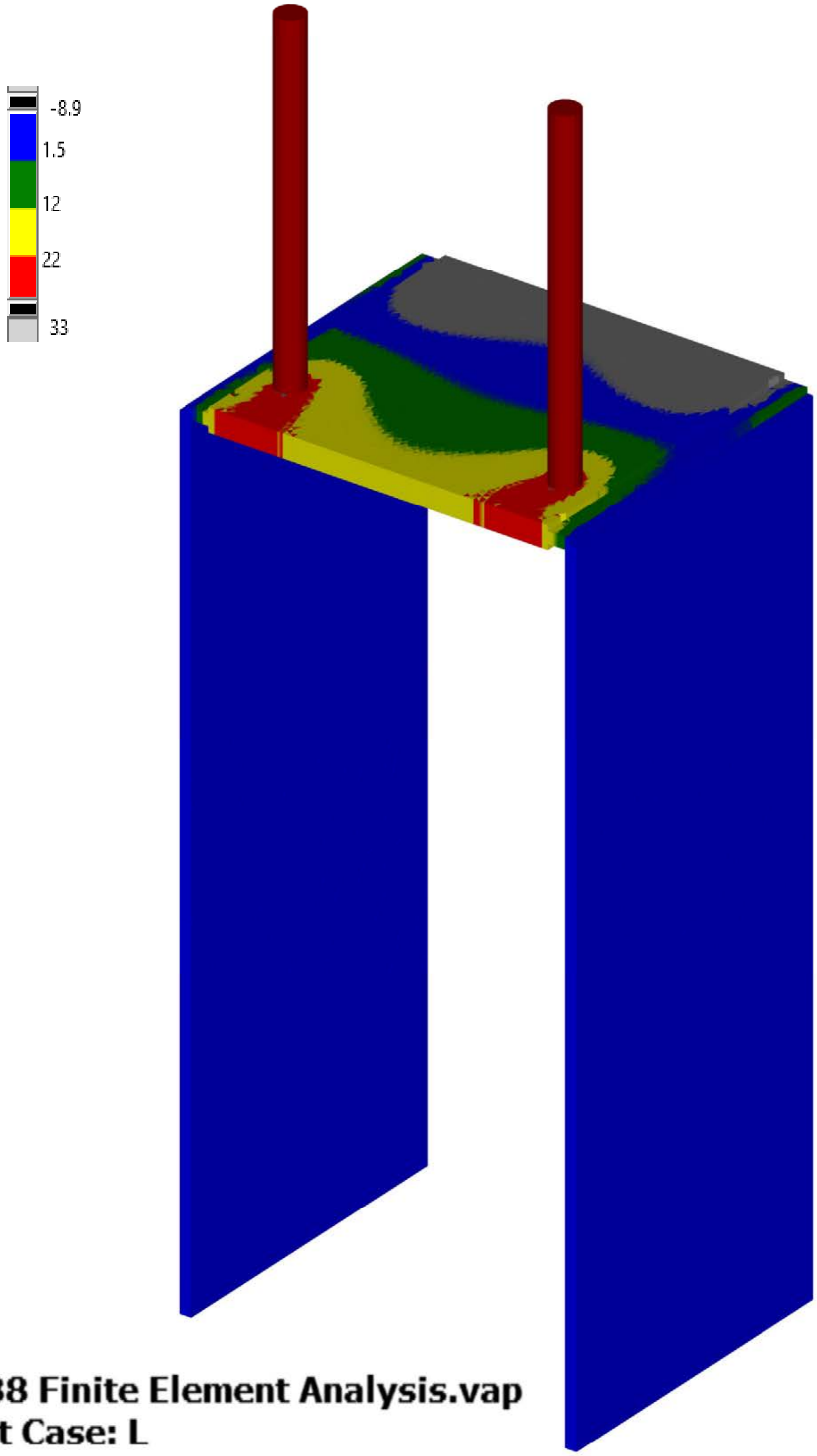
SWP85 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0004
Tuesday, October 16, 2018



SWP88 Finite Element Analysis.vap
Service Case: L
IES VisualAnalysis 18.00.0009
Friday, May 31, 2019



SWP88 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Friday, May 31, 2019



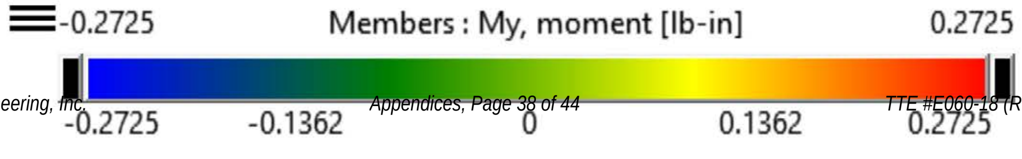
SWP88 Finite Element Analysis.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Friday, May 31, 2019

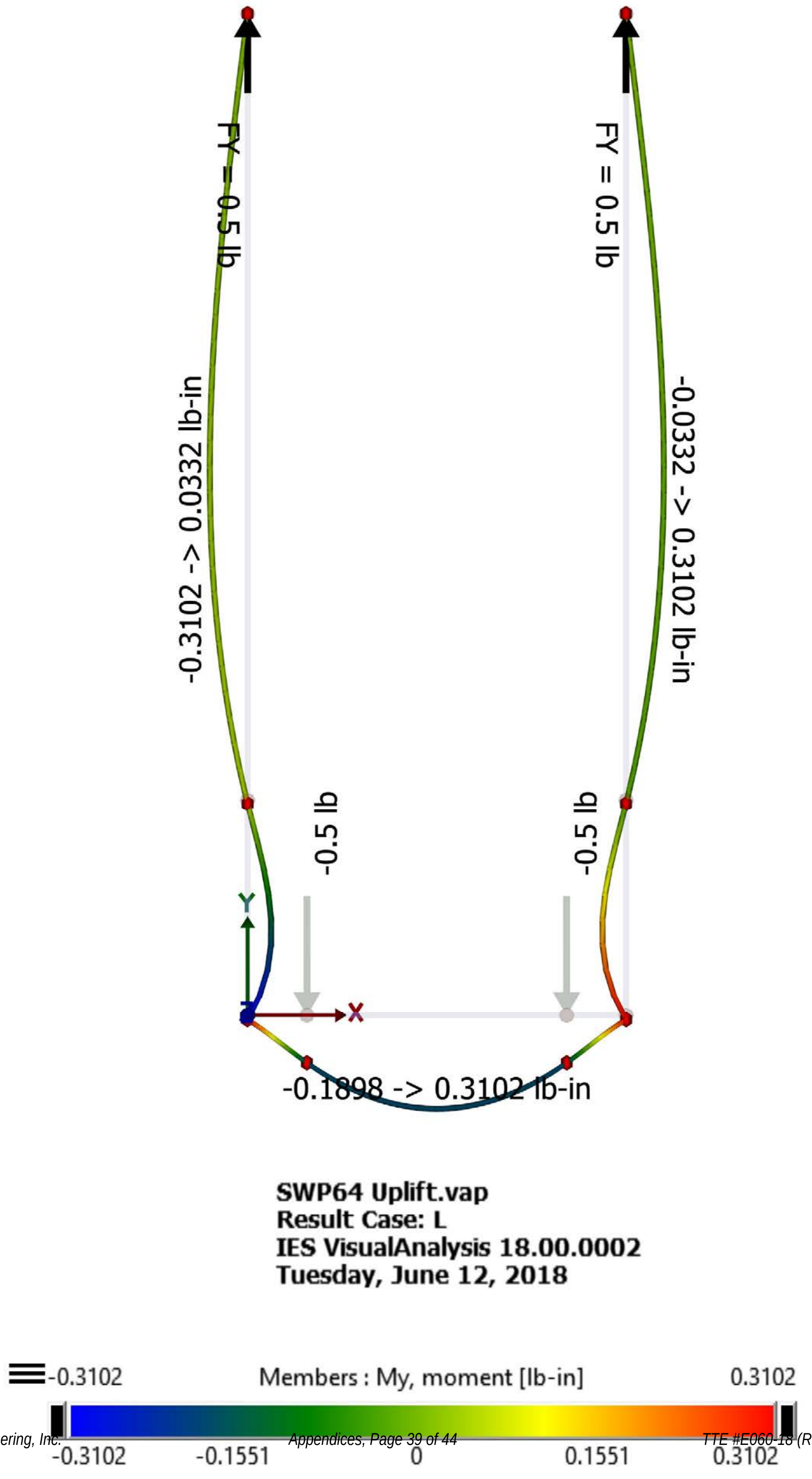
APPENDIX C

Structural Models of Sturdi-Wall Plus Brackets To Determine the Maximum Bending Moment (Unity) In Steel Bracket When Bracket is Subjected to Tensile (Uplift) Load

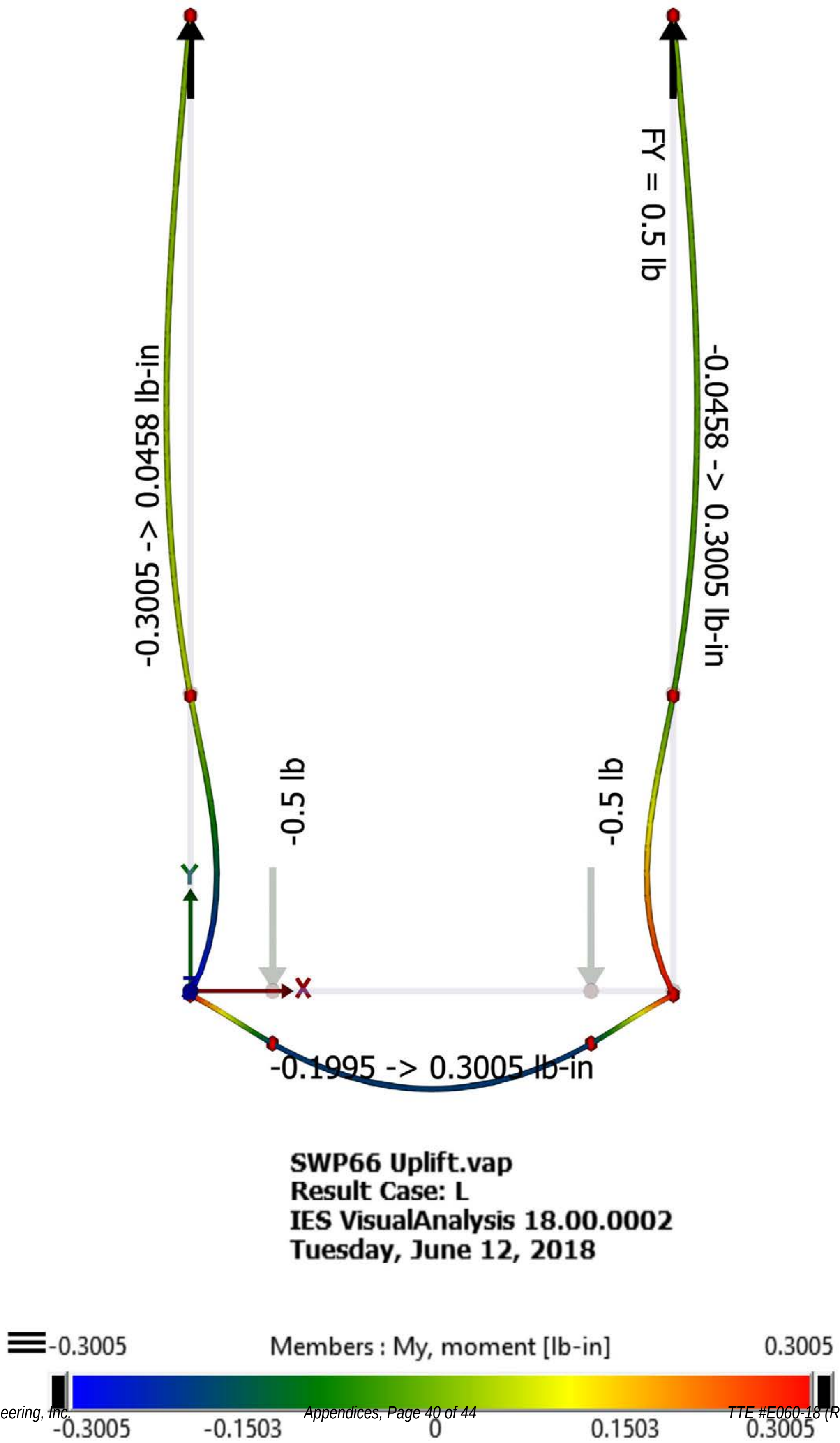
Visual Analysis by IES, Inc
Version 18

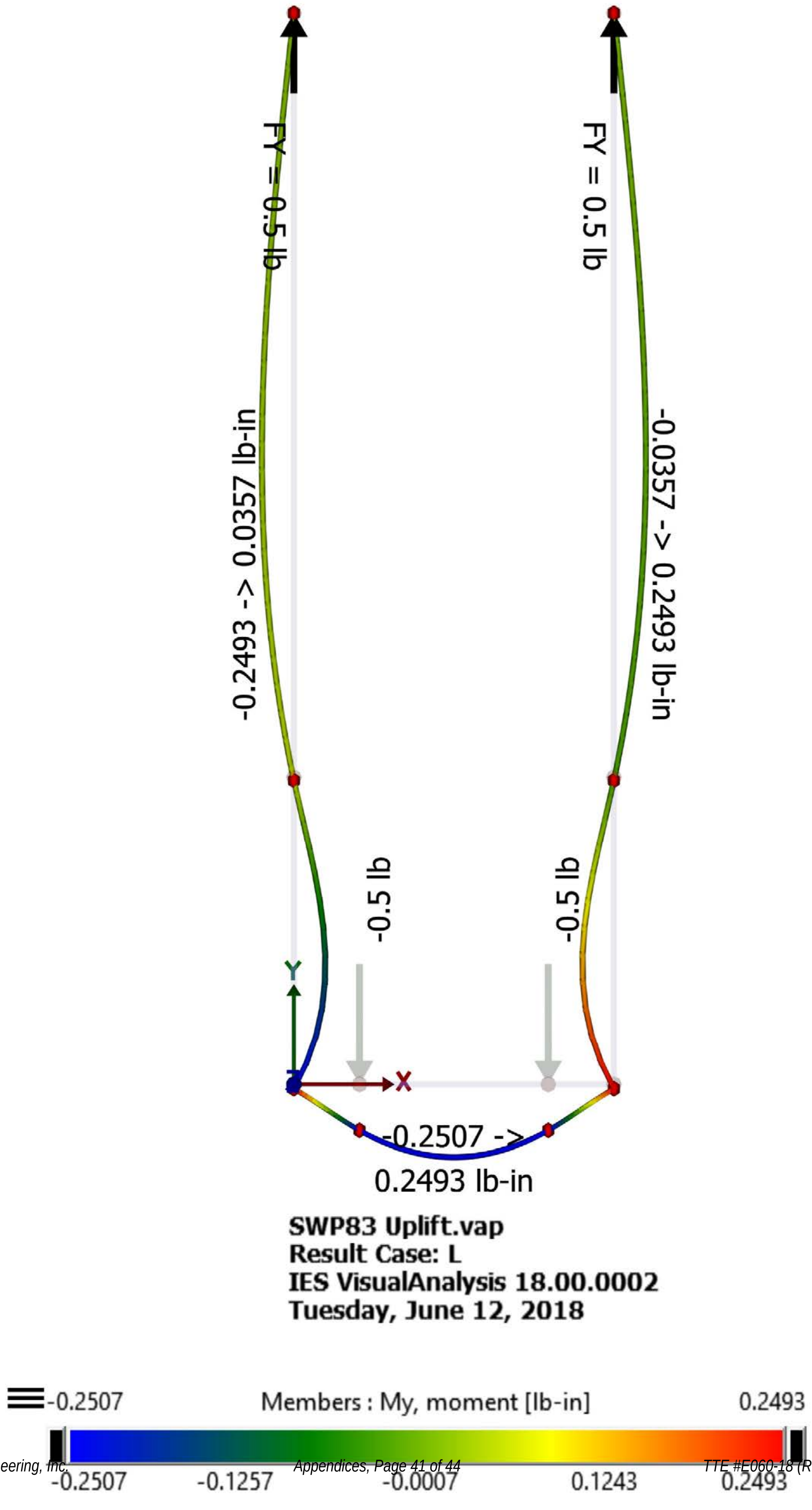
SWP63 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
(SWP46 Similar)

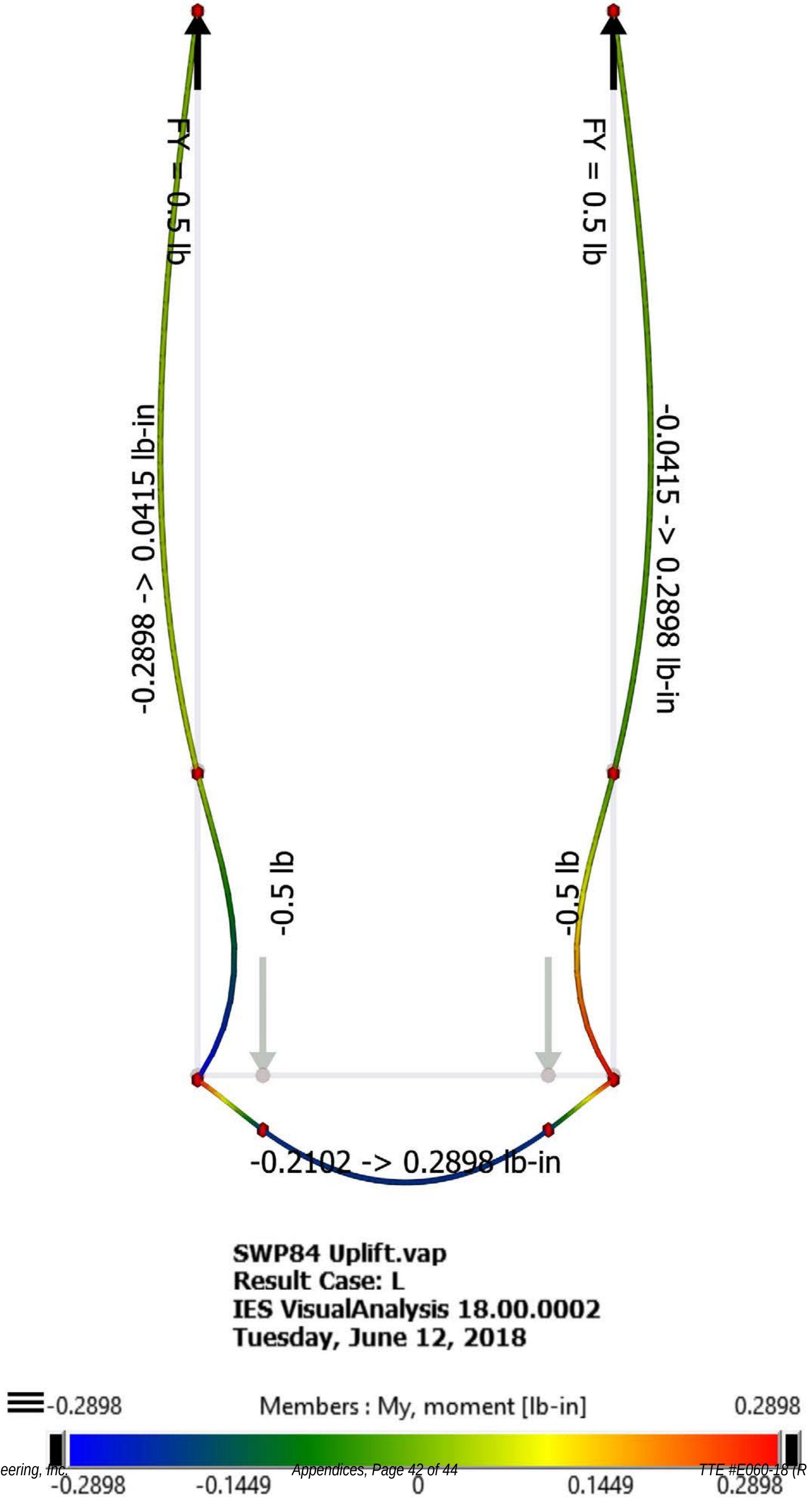




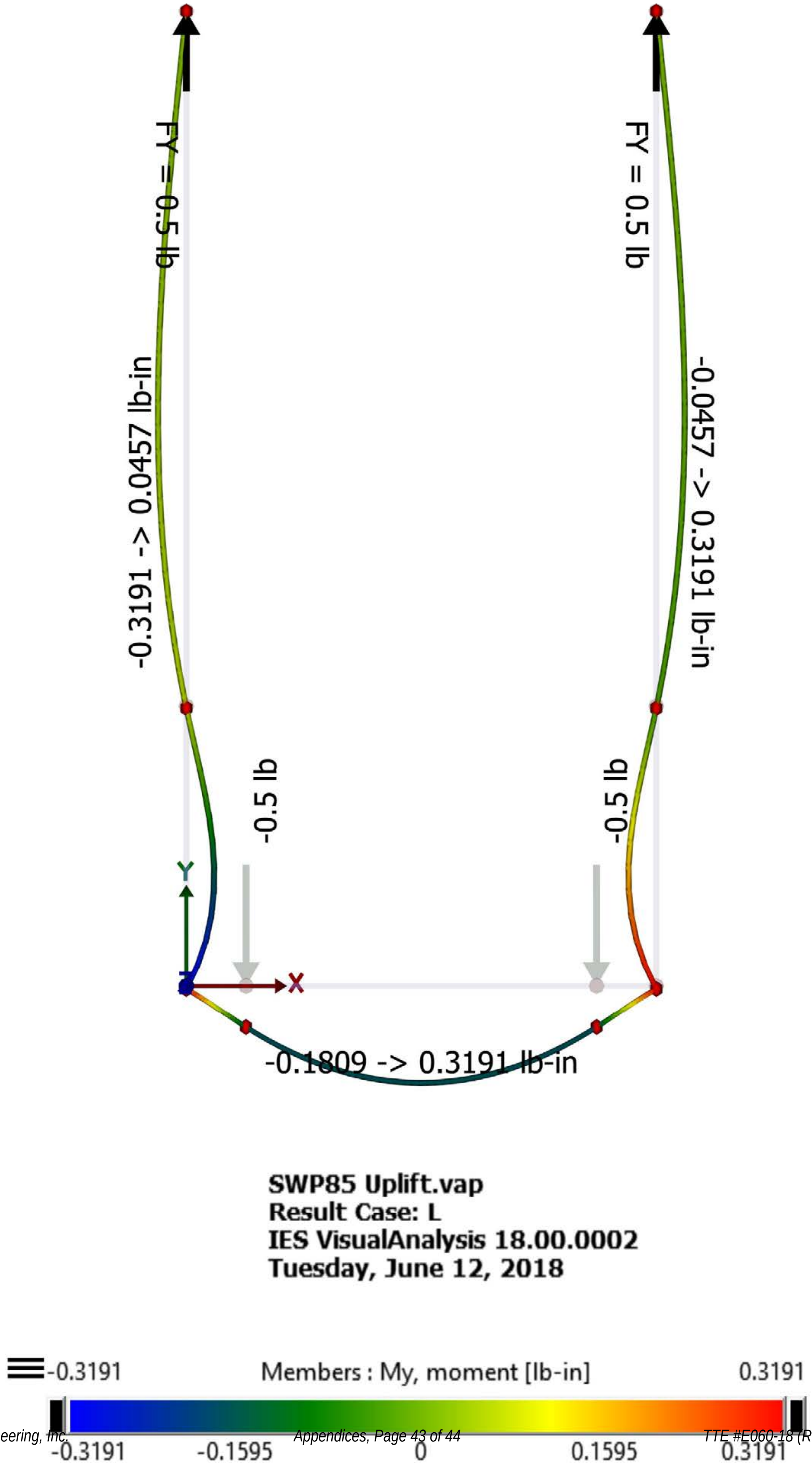
SWP64 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018


-0.3005 -> 0.0458 lb-in
FY = 0.5 lb
-0.0458 -> 0.3005 lb-in
-0.5 lb
-0.5 lb
Y
X
Z
-0.1995 -> 0.3005 lb-in
SWP66 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
-0.3005
Members : My, moment [lb-in]
0.3005
-0.3005
-0.1503
0
0.1503
0.3005

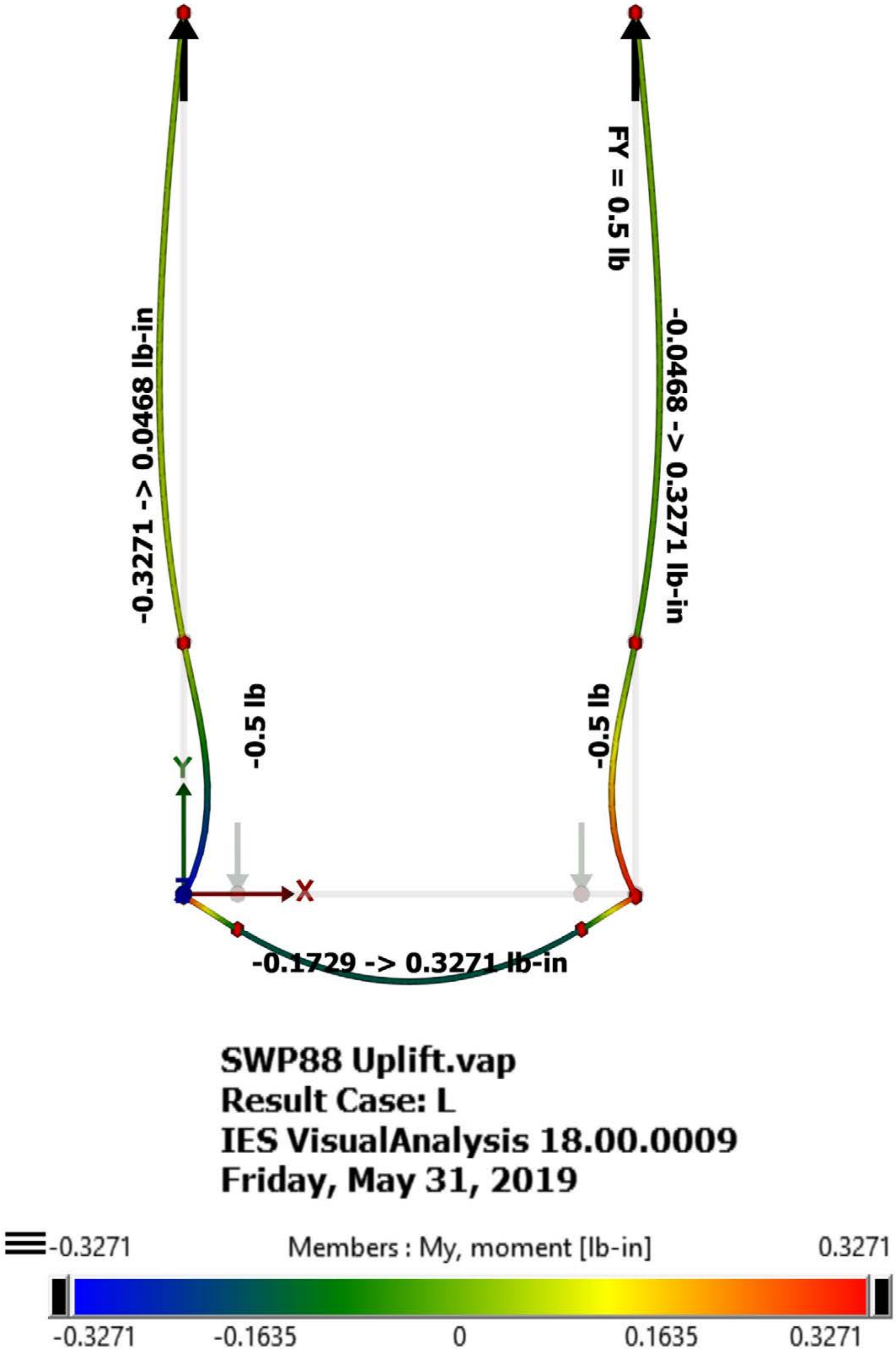

FY = 0.5 lb
FY = 0.5 lb
-0.2493 -> 0.0357 lb-in
-0.0357 -> 0.2493 lb-in
-0.5 lb
-0.5 lb
Y
X
-0.2507 ->
0.2493 lb-in
SWP83 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018
-0.2507
Members : My, moment [lb-in]
0.2493
-0.2507
-0.1257
-0.0007
0.1243
0.2493



SWP84 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018



SWP85 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0002
Tuesday, June 12, 2018


-0.3271 -> 0.0468 lb-in
FY = 0.5 lb
-0.0468 -> 0.3271 lb-in
-0.5 lb
-0.5 lb
Y
X
-0.1729 -> 0.3271 lb-in
SWP88 Uplift.vap
Result Case: L
IES VisualAnalysis 18.00.0009
Friday, May 31, 2019
-0.3271
Members : My, moment [lb-in]
0.3271
-0.3271
-0.1635
0
0.1635
0.3271