Louden Residence

3315 97th Ave SE
Mercer Island, WA 98040

STRUCTURAL CALCULATIONS

April 13, 2020

4-13-20

GARY E. GILL
STATE OF WASHINGTON
29912
REGISTERED PROFESSIONAL ENGINEER

Gary Gill

EXPIRES: 10-11-21

Prepared by

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General Approach

The main portions of the building that will be affected by this remodel have been modeled in Risa3D. The graphic output that is included in the calulations is sufficient for review. A more detailed output from the model is also included at the end of the calculations for deeper review if that is desired. Please let me know if there is anything I can provide during the review process that will facilitate the reviewer's understanding of my process and findings.

Because the remodel will not affect the roof or walls and beams above the second floor that support it, the roof trusses were not modeled in exact detail. Instead, simple beams and rafters were used which will distribute the loads the same way simple trusses will. Beause of this, the beam sizes used in the model for the roof framing have not been shown in the calculations. The configuration of the framing and the loads applied are shown.

The shear walls on the first floor are the only part of the lateral system that is affected by this remodel. Where existing shear walls were removed, new shear walls have been added that, in conjuction with the existing shear walls, keep the unit shear in the existing shear walls below the design unit shear for those shear walls in the original design. The existing hold downs have also been checked in the existing shear walls that have been affected by the remodel and in all cases, the existing hold downs have a greater capacity than the new demand from the revised configuration.

A great deal of interaction between the model and the design has taken place to arrive at a sound and realistic design. Not all of the output values used in the calculations below have output from the Risa runs that verify the values used. If this is a concern please let me know any output you would like to see and I will provide it either during your review or in my comments response.

Design Criteria

Address: Louden Residence
3315 97th Ave SE
Mercer Island, WA 98040

Seismic (2015 IBC)

$Sds := .928$ soil factors included
site class D
$Sd1 := .485$

R = 3.5 and 6.5

Wind (ASCE 7-10)

Wind Speed = 110 mph Exposure D Kzt = 1.0
(exposure and Kzt from Google Earth)
$V := 110$

Roof Snow (ASCE 7-10)

$Pg := 25$ psf $ws := Pg$

Dead Loads

Roof

Roofing	Conc Tile	$rf := 10$
sheathing	1/2" pw	$shtg := 1.5$
Trusses	gang nail trusses	$rtrs := 5.0$
insul	10" BATT	$rins := 1$
ceiling	5/8 gyp	$rclg := 2.2$
Mech/misc		$rspac := 1.3$

$wr := rf + shtg + rtrs + rins + rspac + rclg$
$wr = 21$ psf

2nd Floor

Flooring	tile	$lf := 3$
Sheathing	1.125" pw	$lshtg := 4.3$
purlins	12"TJI@ 16"	$lpurl := 2$
beams	5 1/4X18PSL@16	$lbm := 1.5$
Ceiling	5/8 gyp	$lclg := 2.2$
Misc/Mech		$lmisc := 2$

$w2 := lf + lpurl + lbm + lclg + lshtg + lmisc$
$w2 = 15$ psf $w2l := 40$

1st Floor

Flooring	Tile/Wood	$lf := 3$
Sheathing	1.125" pw	$lshtg := 4.3$
purlins	14" TJI@16"	$lpurl := 3$
beams	5.25x12PSL@16	$lbm := 1.3$
Ceiling	5/8 gyp	$lclg := 2.2$
Misc/Insul		$lmisc := 1.2$

$w1 := lf + lpurl + lbm + lclg + lshtg + lmisc$

$w1 = 15$ psf $\quad wl1 := 40$

Lateral

Building

Seismic

$AreaURoof := 3281$ sf $\qquad WUroof := AreaURoof\cdot(wr + 2) \qquad WUroof = 75463$

$Ct := .028 \quad x := .8 \qquad hn := 22 \qquad Ta := Ct\cdot hn^x \qquad Ta = 0.3$

$Rlong := 6.5 \qquad Cslong := \dfrac{Sds}{Rlong\cdot 1.4} \qquad Cslong = 0.1$ above min and below max

$Cslongmax := \dfrac{Sds}{Rlong\cdot Ta\cdot 1.4} \qquad Cslongmax = 0.3 \qquad Cslongmin := .044\cdot Sds \qquad Cslongmin = 0.041$

$Ct := .02 \quad x := .75 \qquad hn := 22 \qquad Ta := Ct\cdot hn^x \qquad Ta = 0.2$

$Rtrans := 6.5 \qquad Cstrans := \dfrac{Sds}{Rtrans\cdot 1.4} \qquad Cstrans = 0.1$

$Area2ndFloor := 2520$

$W2ndFloor := Area2ndFloor\cdot(w2 + 3)$

$W2ndFloor = 45360$

$hr := 25 \qquad hrxWroof := hr\cdot WUroof \qquad hrxWroof = 1886575$

$h2 := 10.66 \quad h2xW2ndFloor := h2\cdot W2ndFloor \qquad h2xW2ndFloor = 483537.6$

$SumhxW := hrxWroof + h2xW2ndFloor \qquad SumhxW = 2370112.6$

$Csr := \dfrac{hrxWroof}{SumhxW} \qquad Csr = 0.8$

$Cs2 := \dfrac{h2xW2ndFloor}{SumhxW} \qquad Cs2 = 0.2$

$Wtot := WUroof + W2ndFloor \qquad Wtot = 120823$

$Vsrlong := Wtot\cdot Csr\cdot Cslong \qquad Vsrlong = 9807.6$

$Vsrtrans := Wtot \cdot Csr \cdot Cstrans$ $Vsrtrans = 9807.6$

$Vs2long := Wtot \cdot Cs2 \cdot Cslong$ $Vs2long = 2513.7$

$Vs2trans := Wtot \cdot Cs2 \cdot Cstrans$ $Vs2trans = 2513.7$

$Vsrtrans + Vs2trans = 12321.3$

$Vsrlong + Vs2long = 12321.3$

Wind

Basic Wind Speed $BWS := 110$ mph **Exposure D**

$Alpha := 11.5$ $Zg := 700$ $Ht := 25$

$Kd := .85$ $Kz := 2.01 \cdot \left(\frac{Ht}{Zg}\right)^{\frac{2}{Alpha}}$ $Kz = 1.1$ $Kzt := 1.0$

$q := .00256 \cdot Kd \cdot Kz \cdot Kzt \cdot \frac{BWS^2}{1.4}$ $q = 21.2$

$Vwrlong := 50 \cdot 10 \cdot q$ $Vwrlong = 10587.7$

$Vwrtrans := 64 \cdot 10 \cdot q$ $Vwrtrans = 13552.3$

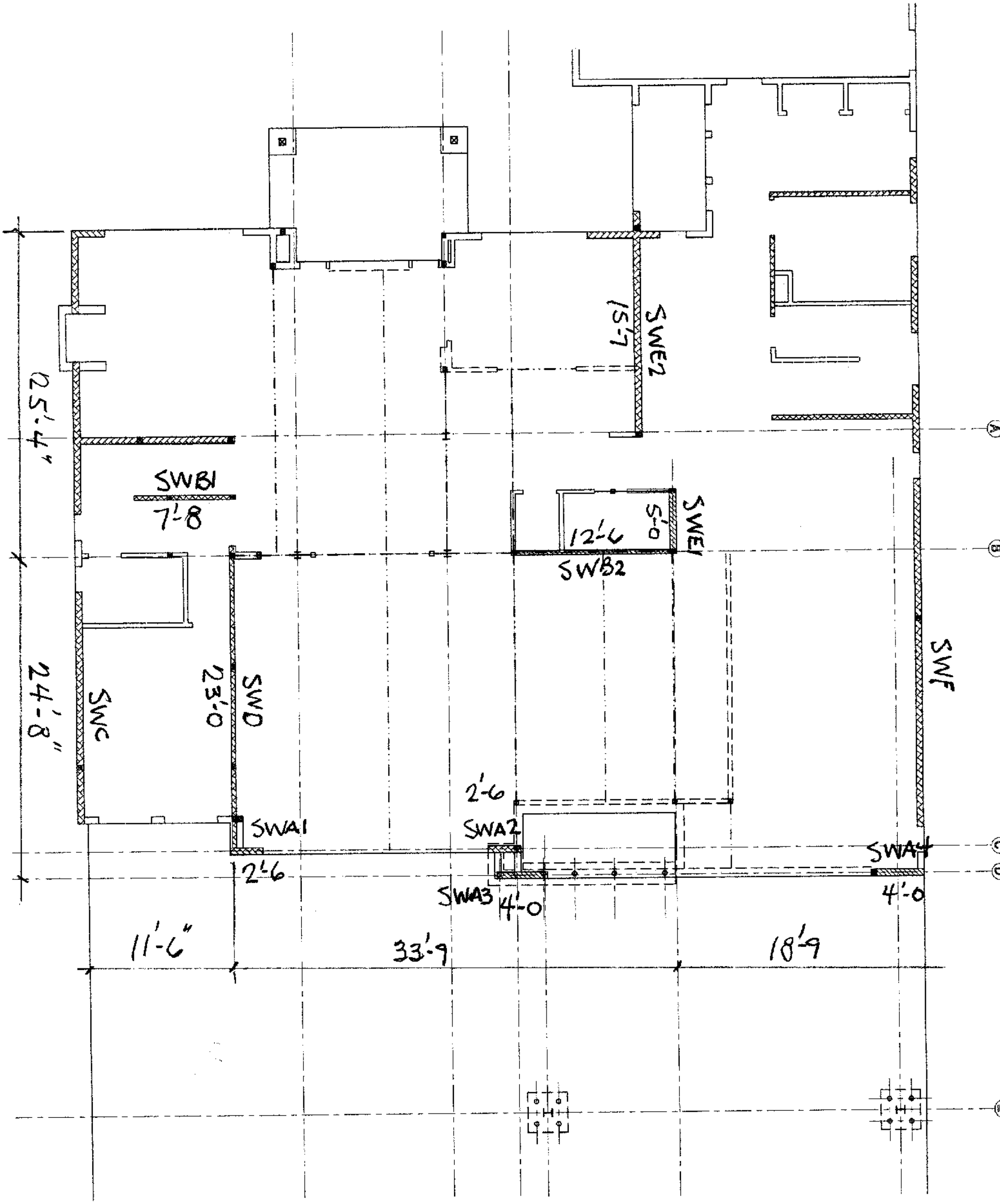
$Vw2long := 50 \cdot 10 \cdot q$ $Vw2long = 10587.7$

$Vw2trans := 64 \cdot 10 \cdot q$ $Vw2trans = 13552.3$

$Vwtrans := Vwrtrans + Vw2trans$ $Vwtrans = 27104.6$

$Vwlong := Vwrlong + Vw2long$ $Vwlong = 21175.4$

Wind Controls in Both Direction



MAIN FLOOR SHEAR WALLS

Shear Walls

1st Level to 2nd Level

Shear Wall Redundancy

As long as twice the total shear wall length divided by the story height is greater than 2, a redundancy factor of 1.0 is allowed.

$LA := 2.5 + 2.5 + 4 + 4 \qquad LA = 13 \qquad NoBays := \frac{2 \cdot LA}{10} \qquad NoBays = 2.6 \qquad$ OK

$LB := 7.66 + 12.5 \qquad LB = 20.2 \qquad NoBays := \frac{2 \cdot LB}{10} \qquad NoBays = 4 \qquad$ OK

$LC := 23 \qquad LC = 23 \qquad NoBays := \frac{2 \cdot LC}{10} \qquad NoBays = 4.6 \qquad$ OK

$LD := 20.57 \qquad LD = 20.6 \qquad NoBays := \frac{2 \cdot LD}{10} \qquad NoBays = 4.1 \qquad$ OK

Shear Wall SWA1

$V := \frac{(Vwrlong + Vw2long)24.66 \cdot 2.5}{2 \cdot 50 \cdot LA} \qquad V = 1004.2 \qquad L := 2.5 \qquad v := \frac{V}{L} \qquad v = 401.7$

$Pdl := 42 \qquad HDl := v \cdot 10 - Pdl \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10 \qquad HDl = 3851.6 \qquad HDlu := 1.4 \cdot HDl \qquad HDlu = 5392.2$

$Pdr := 1100 \qquad HDr := v \cdot 10 - Pdr \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10 \qquad HDr = 3153.3 \qquad HDru := 1.4 \cdot HDr \qquad HDru = 4414.6$

This is an existing shear wall that was designed for a unit shear of 471 plf which is greater than the demand that exists in the new configuration. Therefore the existing wall is adequate. The existing holdowns have also been evaluated for the worst case which is HDlu developed above. Please see anchor bolt calc following.

Shear Wall SWA2

$V := \frac{(Vwrlong + Vw2long)24.66 \cdot 2.5}{2 \cdot 50 \cdot LA} \qquad V = 1004.2 \qquad L := 2.5 \qquad v := \frac{V}{L} \qquad v = 401.7$

$Pdl := 2100 \qquad HDl := v \cdot 10 - Pdl \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10 \qquad HDl = 2493.3 \qquad HDlu := 1.4 \cdot HDl \qquad HDlu = 3490.6$

$Pdr := 6100 \qquad HDr := v \cdot 10 - Pdr \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10 \qquad HDr = -146.7 \qquad HDru := 1.4 \cdot HDr \qquad HDru = -205.4$

This is an existing shear wall that was designed for a unit shear of 471 plf which is greater than the demand that exists in the new configuration. Therefore the existing wall is adequate. The existing holdowns are the same as those for SWA1 while the holdown tensions are less than those evaluated for SWA1 therefore, the existing shear wall and hold downs are adequate as is.

Shear Wall SWA3 (the 635 is the seismic from the patio roof)

$V := \frac{(Vwrlong + Vw2long)24.66\cdot 4}{2\cdot 50\cdot LA} + \frac{635}{2}$ $\quad V = 1924.2 \quad L := 4 \quad v := \frac{V}{L} \quad v = 481.1$

$Pdl := -100 \quad HDl := v\cdot 10 - Pdl\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 \quad HDl = 4656.6$

$Pdr := 5900 \quad HDr := v\cdot 10 - Pdr\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 \quad HDr = 696.6$

This is a new shear wall with a new footing. Use: SW3 per the shear wall schedule on Sheet S4.1 For the hold down use HD3 from hold down schedule on sheet S4.1 with SB 5/8 x 24 AB.

Shear Wall SWA4 (the 635 is the seismic from the patio roof)

$V := \frac{(Vwrlong + Vw2long)24.66\cdot 4}{2\cdot 50\cdot LA} + \frac{635}{2}$ $\quad V = 1924.2 \quad L := 4 \quad v := \frac{V}{L} \quad v = 481.1$

$Pdl := 4200 \quad HDl := v\cdot 10 - Pdl\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 \quad HDl = 1818.6 \quad HDlu := 1.4\cdot HDl \quad HDlu = 2546$

$Pdr := 300 \quad HDr := v\cdot 10 - Pdr\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 \quad HDr = 4392.6 \quad HDru := 1.4\cdot HDr \quad HDru = 6149.6$

This is a new shear wall with an existing footing. Use: SW3 per the shear wall schedule on Sheet S4.1 For the hold down use HD3 from hold down schedule on sheet S4.1 with 5/8" x 10" emb dia epoxy AB with Simpson AT epoxy. See Anchor calc below.

Shear Wall SWB1

$V := \frac{(Vwrlong + Vw2long)\cdot 7.66}{2\cdot LB}$ $\quad V = 4022.9 \quad L := 7.66 \quad v := \frac{V}{L} \quad v = 525.2$

$Pdl := 100 \quad HDl := v\cdot 10 - Pdl\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 \quad HDl = 4764.5$

$Pdr := 500 \quad HDr := v\cdot 10 - Pdr\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 \quad HDr = 4500.5$

This is an existing shear wall that has an allowable unit shear of 640 plf which is greater than the demand that calculated above. The hold downs that are specified for this shear wall is an HTT22 with an allowable tension value of 5090pds which is greater than the calculated hold down above.

$Tall := 6\cdot 1.6\cdot 130 + 650 \quad Tall = 1898$ Which is greater than the demand developed above

The blocking is also attached to both shear walls with 8d @ 6"

$Tall := 10\cdot 220 \quad Tall = 2200$ Which is also greater than the demand developed above

Shear Wall SWB2

$V := \frac{(Vwrlong + Vw2long) \cdot 12.5}{2 \cdot LB}$ $V = 6564.8$ $L := 12.5$ $v := \frac{V}{L}$ $v = 525.2$

$Pdl := 4000$ $HDl := v \cdot 10 - Pdl \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10$ $HDl = 1924.3$

$Pdr := 3870$ $HDr := v \cdot 10 - Pdr \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10$ $HDr = 2010.1$

This is a new shear wall. Use: SW3 from the shear schedule on Sheet S4.1 and HD2 hold downs from the Hold Down Schedule on sheet S4.1,

SWC

This is an existing shear wall for which the lateral load has not be changed and therefore is ok as is.

Shear Wall SWD

$V := \frac{(Vwrtrans + Vw2trans) \cdot (11.5 + 33.75)}{2 \cdot 64}$ $V = 9581.9$ $L := 23$ $v := \frac{V}{L}$ $v = 416.6$

$Pdl := 4000$ $HDl := v \cdot 10 - Pdl \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10$ $HDl = 261$

$Pdr := 3870$ $HDr := v \cdot 10 - Pdr \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10$ $HDr = 346.8$

This is an existing shear wall. The allowable shear for this wall is 840 plf per the original calcs which is greater than the calculated value above. The existing hold downs are more than adequate for the hold downs calculated above.

Shear Wall SWE1

$V := \frac{(Vwrtrans + Vw2trans) \cdot (18.75 + 33.75) \cdot 5}{2 \cdot 64 \cdot LD}$ $V = 2702.3$ $L := 5$ $v := \frac{V}{L}$ $v = 540.5$

$Pdl := 314$ $HDl := v \cdot 10 - Pdl \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10 - \frac{L}{2} \cdot 171$ $HDl = 4494.8$

$Pdr := 3698$ $HDr := v \cdot 10 - Pdr \cdot .66 - \frac{L}{2} \cdot 11 \cdot 10 - \frac{L}{2} \cdot 171$ $HDr = 2261.3$

This is a new shear wall. Use: SW3 from the shear schedule on Sheet S4.1 and HD3 hold downs from the Hold Down Schedule on sheet S4.1,

Shear Wa ll SWE2

$V := \frac{(Vwrtrans + Vw2trans)\cdot(18.75 + 33.75)\cdot 15.57}{2\cdot 64\cdot LD}$ $V = 8414.8$ $L := 15.57$ $v := \frac{V}{L}$ $v = 540.5$

$Pdl := 564$ $HDl := v\cdot 10 - Pdl\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 - \frac{L}{2}\cdot 191$ $HDl = 2689$

$Pdr := 1807$ $HDr := v\cdot 10 - Pdr\cdot .66 - \frac{L}{2}\cdot 11\cdot 10 - \frac{L}{2}\cdot 191$ $HDr = 1868.6$

This is a new shear wall. Use: SW3 from the shear schedule on Sheet S4.1 and HD2 hold downs from the Hold Down Schedule on sheet S4.1,

SWF

This is an existing shear wall for which the lateral load has not be changed and therefore is ok as is.

Patio Roof

Seismic

$AreaURoof := 660$ sf $WUroof := AreaURoof\cdot(10)$ $WUroof = 6600$

$Ct := .028$ $x := .8$ $hn := 22$ $Ta := Ct\cdot hn^x$ $Ta = 0.3$

$Rlong := 3.5$ $Cslong := \frac{Sds}{Rlong\cdot 1.4}$ $Cslong = 0.19$ above min and below max

$Cslongmax := \frac{Sds}{Rlong\cdot Ta\cdot 1.4}$ $Cslongmax = 0.6$ $Cslongmin := .044\cdot Sds$ $Cslongmin = 0.041$

$Ct := .02$ $x := .75$ $hn := 22$ $Ta := Ct\cdot hn^x$ $Ta = 0.2$

$Rtrans := 6.5$ $Cstrans := \frac{Sds}{Rtrans\cdot 1.4}$ $Cstrans = 0.1$

$Vsrtrans := Cstrans\cdot WUroof$ $Vsrtrans = 673.1$

$Vsrlong := Cslong\cdot WUroof$ $Vsrlong = 1250$

Wind

Basic Wind Speed $BWS := 110$ mph Exposure D

$Alpha := 11.5$ $Zg := 700$ $Ht := 25$

$Kd := .85$ $Kz := 2.01\cdot\left(\frac{Ht}{Zg}\right)^{\frac{2}{Alpha}}$ $Kz = 1.1$ $Kzt := 1.0$

$q := .00256\cdot Kd\cdot Kz\cdot Kzt\cdot\frac{BWS^2}{1.4}$ $q = 21.2$

$Vwrlong := 22\cdot 1\cdot q$ $Vwrlong = 465.9$

$Vwrtrans := 30\cdot 1\cdot q$ $Vwrtrans = 635.3$

Seismic Controls in Both Direction

Moment Frame

$Vs := \frac{Vsrlong}{2}$ $Vs = 625$ See Risa results following

Moment Frame Connections

Beam to Column

$Mmax := 12.7$ $Vmax := 1.2$ By inspection use CJP at top and bottom flanges and at beam web to column flange.

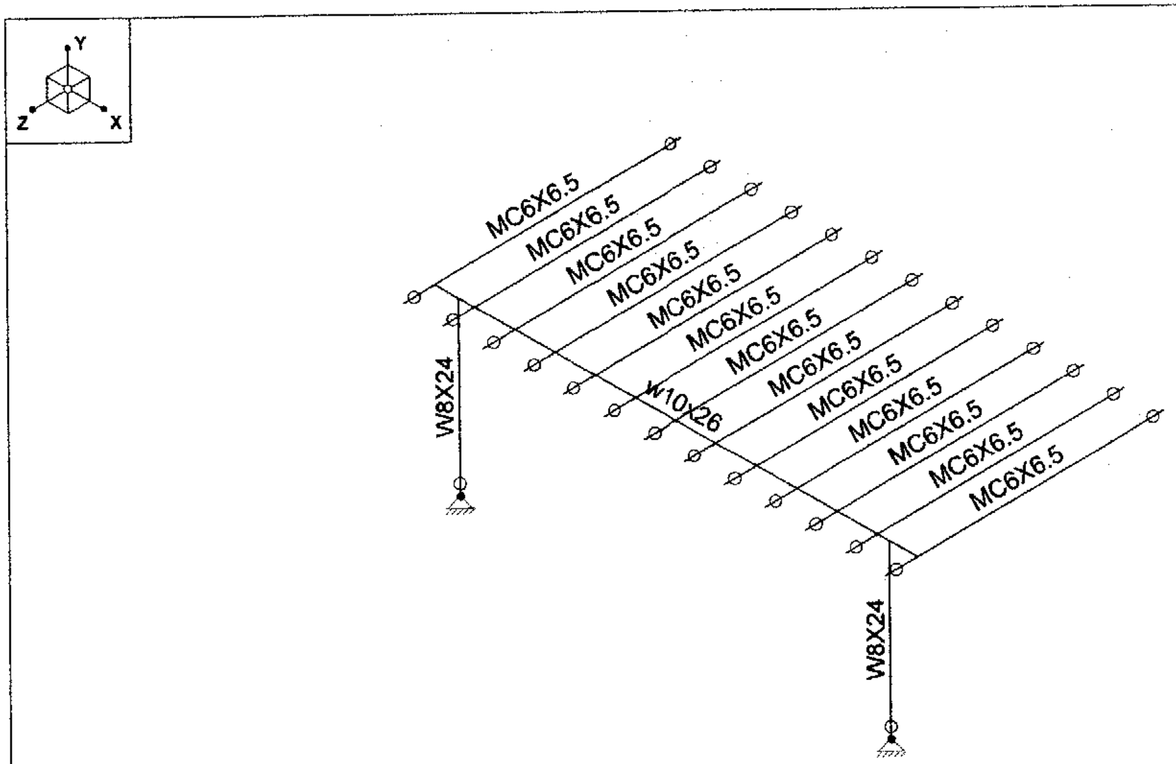
Base Plate to Footing

$Vmax := 1.2$ $Tmax := \frac{15 \cdot 22 \cdot 22}{2 \cdot 18} \cdot (q - 10)$ $Tmax = 2253.7$ Tmax from net wind uplift.

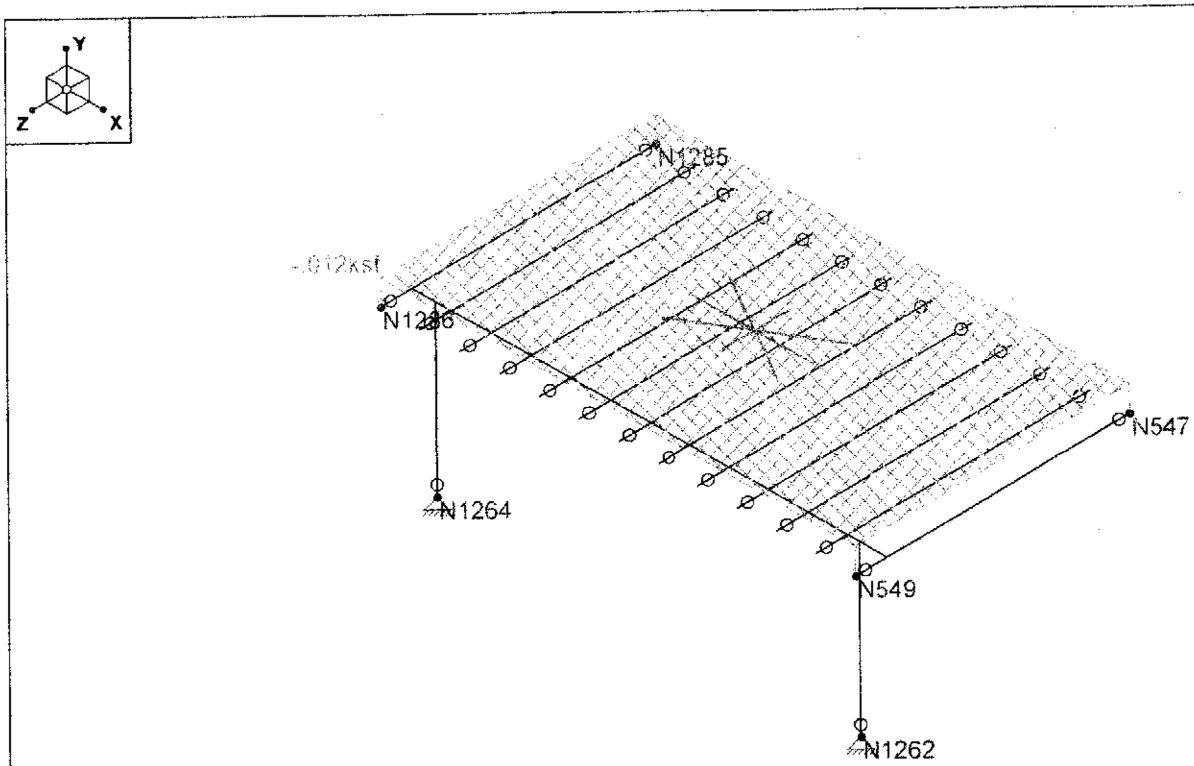
See Simpson Anchor Design following. Use: Base PL7x1/2x8 1/2 w/ 2-5/8" dia x 8" emb hvy hex anchor bolts.

Footing

$Pmax := Tmax + 5500$ $Pmax = 7753.7$ By Inspection Use: 4 - 2" dia pipe piles w/48" SQ x 12" D footing w/ 5-#5 ea wat top and bot.



		Patio Roof - 1
		Apr 11, 2020 at 5:29 PM
	Member Sizes	louden risa model 4-9-20 3.r3d



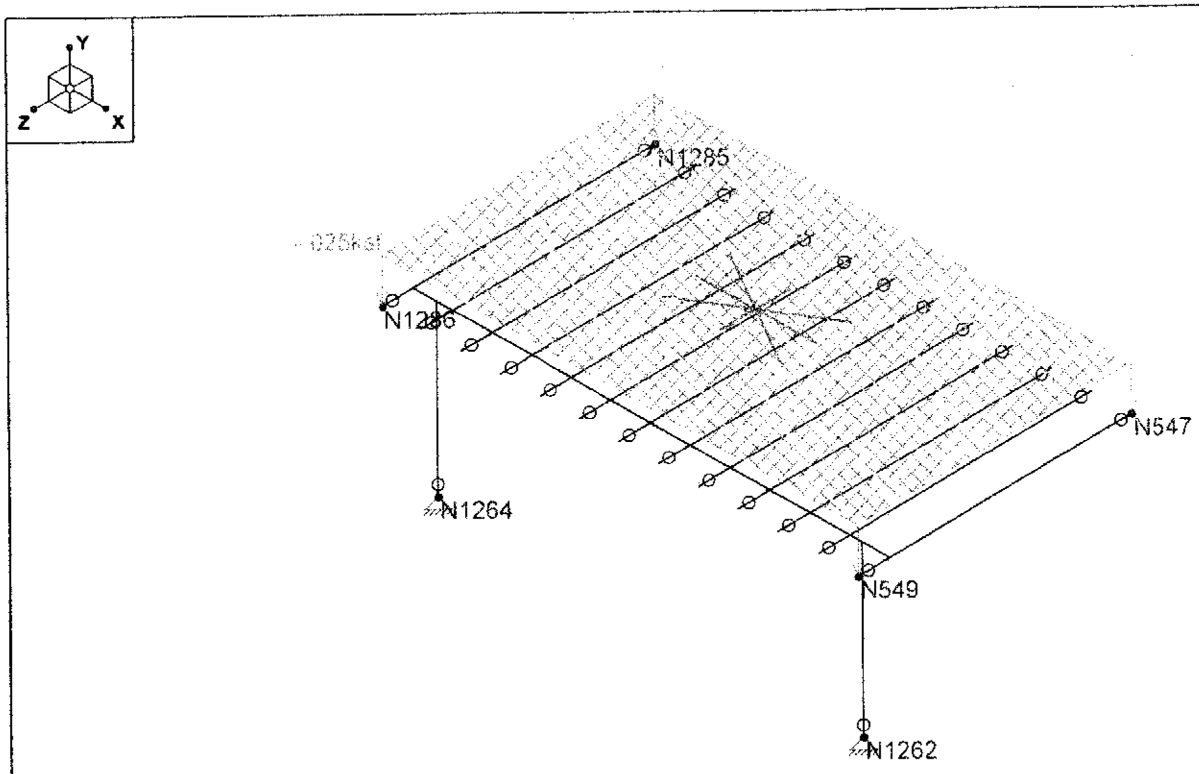
Loads: BLC 1, dead

Dead Load

Patio Roof - 2

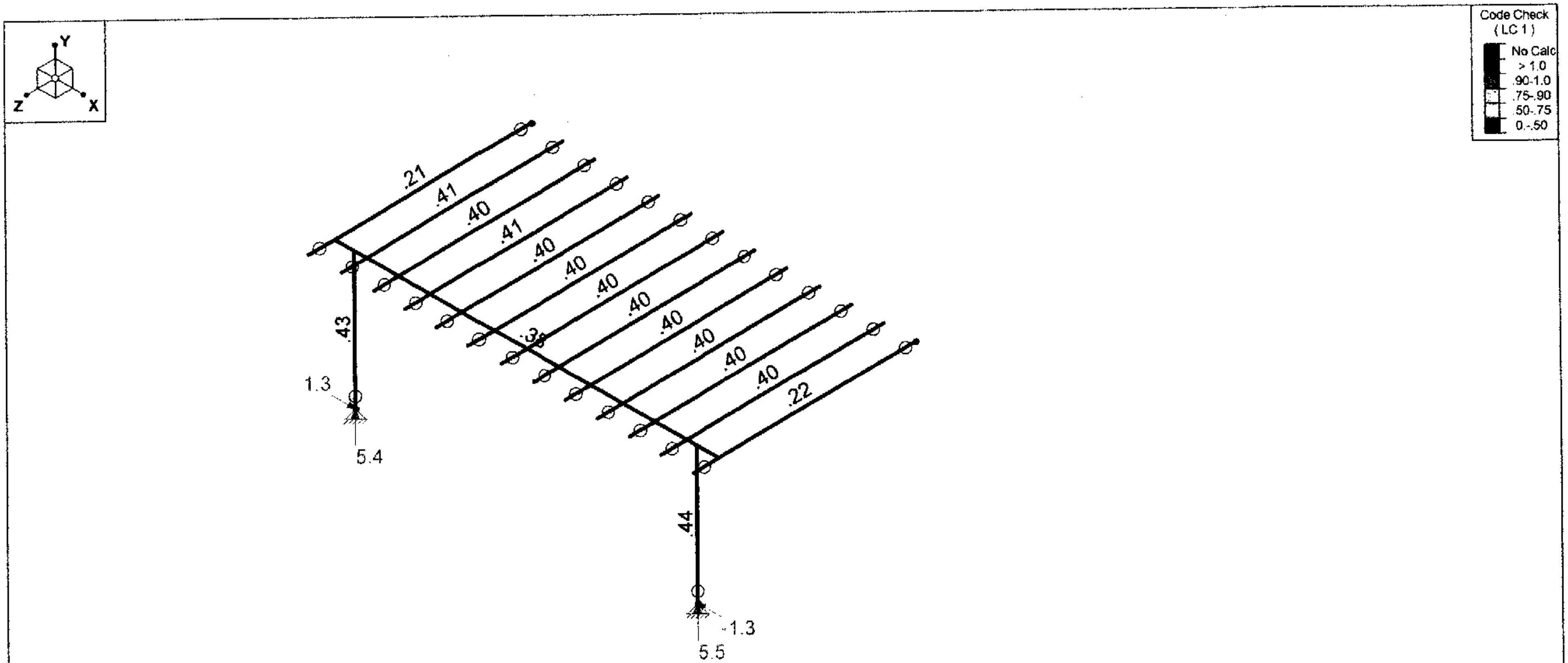
Apr 11, 2020 at 5:34 PM

louden risa model 4-9-20 3.r3d



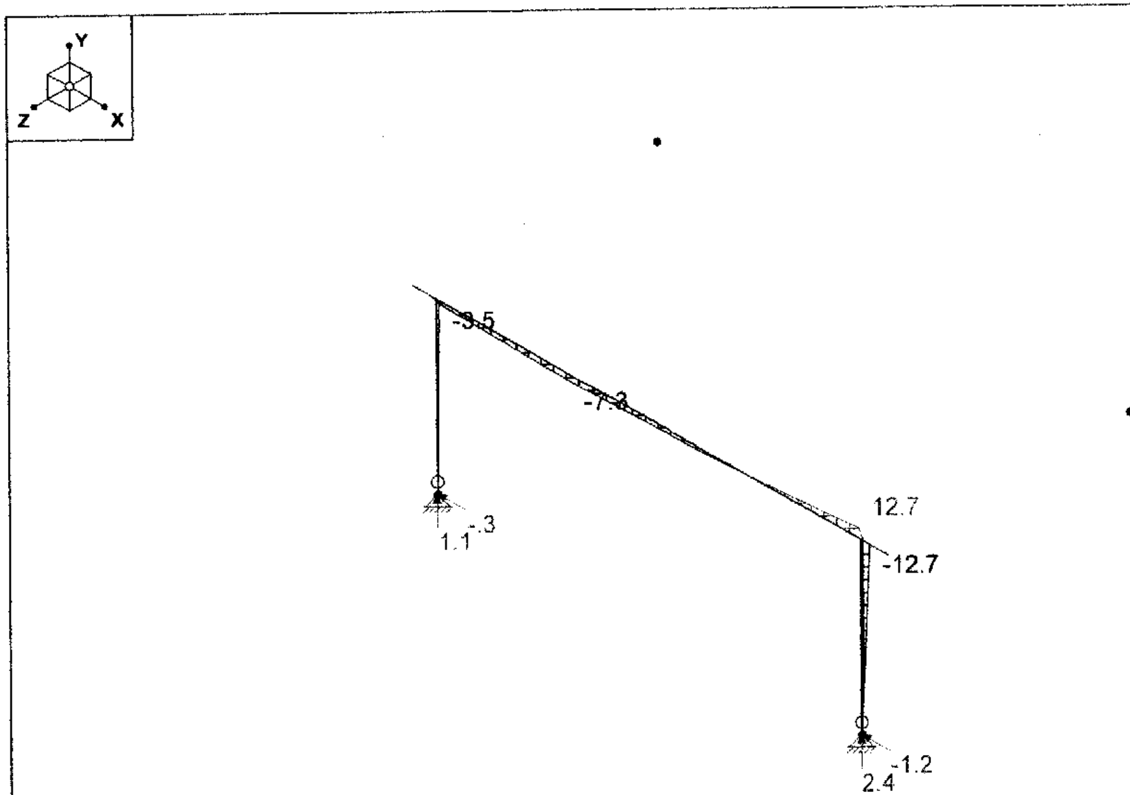
Loads: BLC 2, snow

		Patio Roof - 3
		Apr 11, 2020 at 5:35 PM
	Snow Load	louden risa model 4-9-20 3.r3d



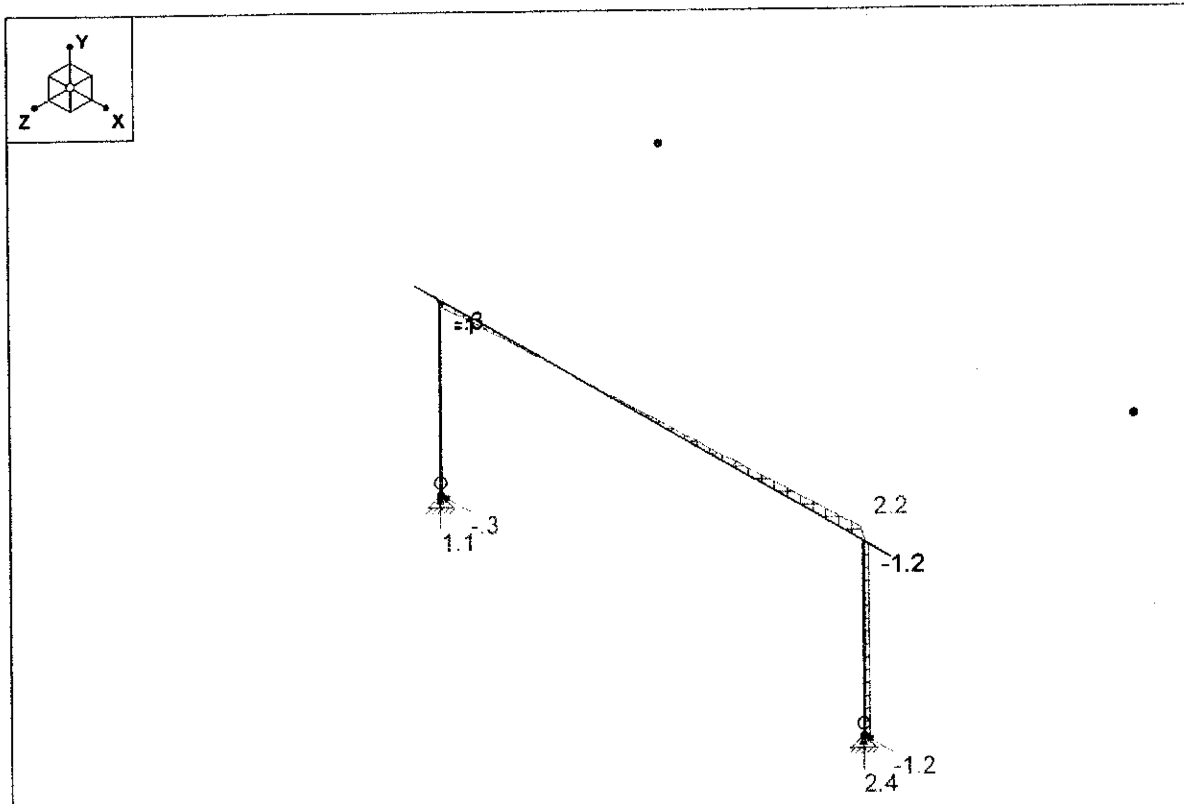
Member Code Checks Displayed
Results for LC 1, d+sn
Reaction and Moment Units are k and k-ft

	Member Stress Levels and Reactions for Dead + Snow	Patio Roof - 4
		Apr 11, 2020 at 5:38 PM
		louden risa model 4-9-20 3.r3d



Results for LC 11, d+2.5patio seis
Member z Bending Moments (k-ft)
Reaction and Moment Units are k and k-ft

	Moments and Reactions for Dead + 2.5 seis	Patio Roof - 5
		Apr 11, 2020 at 5:40 PM
		louden risa model 4-9-20 3.r3d



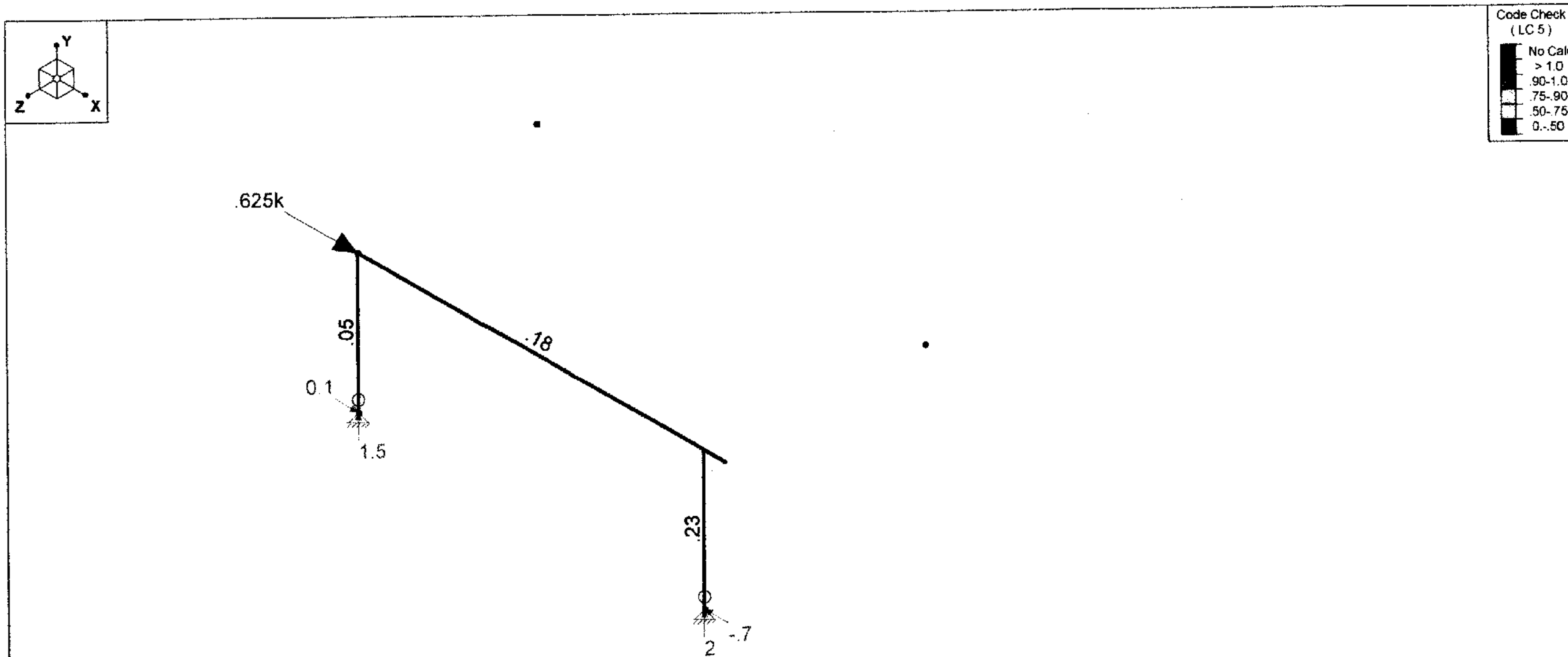
Results for LC 11, d+2.5patio seis
Member y Shear Forces (k)
Reaction and Moment Units are k and k-ft

Shears and Reactions for Dead + 2.5 seis

Patio Roof - 6

Apr 11, 2020 at 5:42 PM

louden risa model 4-9-20 3.r3d



Member Code Checks Displayed
Loads: BLC 4, patio roof seism
Results for LC 5, d+patio seis
Reaction and Moment Units are k and k-ft

		Patio Roof - 7
		Apr 11, 2020 at 5:48 PM
	Member Stresses, Seismic Load and Reactions for Dead + Patio Roof Seis	louden risa model 4-9-20 3.r3d

Company:	Gary E. Gill, SE	Date:	4/10/2020
Engineer:	GEG	Page:	1/6
Project:	Louden		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Base Plate for Patio Roof Columns
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

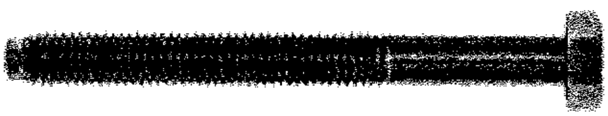
Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.38
C_{min} (inch): 3.75
S_{min} (inch): 3.75

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 8.50 x 7.00 x 0.50
Yield stress: 34084 psi

Profile type/size: W8X24

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 5/8"Ø Heavy Hex Bolt, F1554 Gr. 36

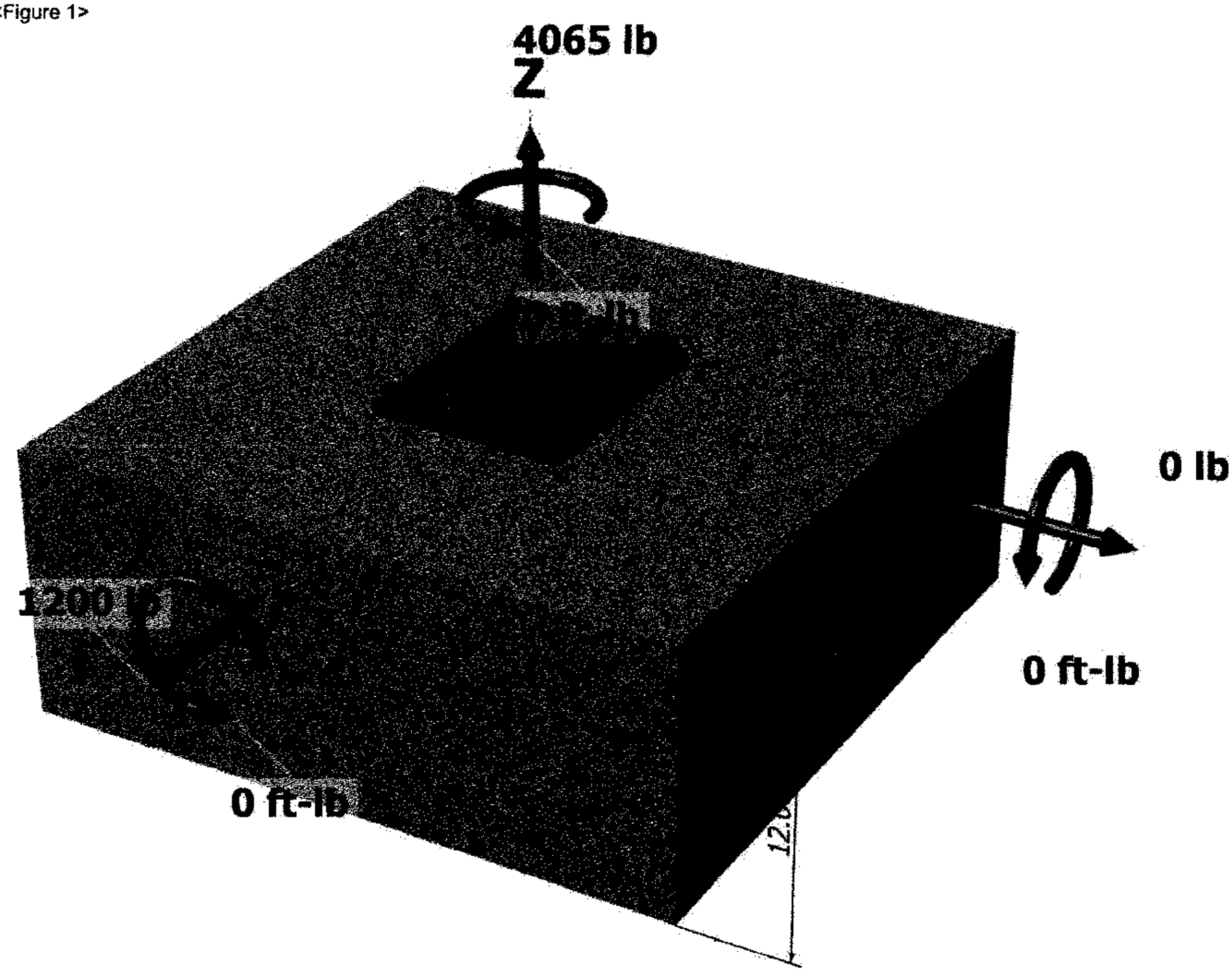


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

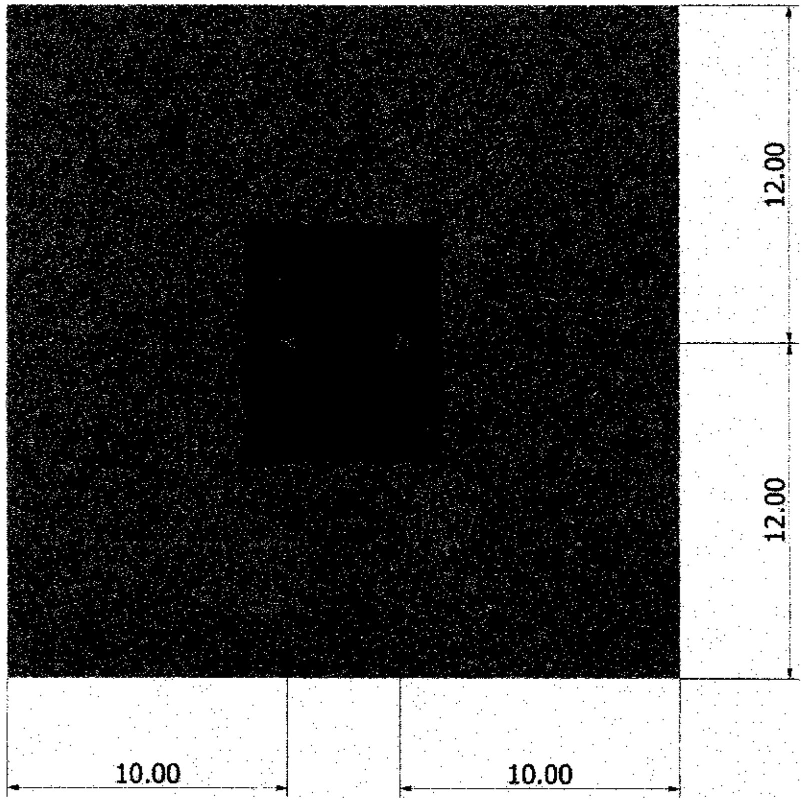
Strength level loads:

N_{ua} [lb]: 4065
V_{uax} [lb]: 1200
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

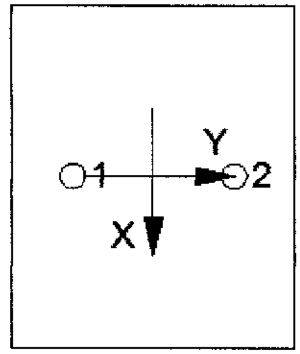


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	2032.5	600.0	0.0	600.0
2	2032.5	600.0	0.0	600.0
Sum	4065.0	1200.0	0.0	1200.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4065
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13100	0.75	9825

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	8.000	27153

$\phi N_{cbg} = \phi\, (A_{Nc} / A_{Nco})\, \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
576.00	576.00	10.00	1.000	0.950	1.00	1.000	27153	0.70	18057

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.67	2500	0.70	9394

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
7865	0.8	0.65	4090

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.625	1.00	2500	8.00	9490

$\phi V_{cbgx} = \phi\ (A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
288.00	288.00	1.000	0.950	1.000	1.000	9490	0.70	6311

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
5.00	0.625	1.00	2500	8.00	9490

$\phi V_{cbx} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
288.00	288.00	1.000	1.000	1.000	9490	0.70	13286

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	576.00	576.00	1.000	0.950	1.000	1.000	27153	0.70	36113

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	2033	9825	0.21	Pass
Concrete breakout	**4065**	**18057**	**0.23**	**Pass (Governs)**
Pullout	2033	9394	0.22	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	600	4090	0.15	Pass
T Concrete breakout x+	**1200**	**6311**	**0.19**	**Pass (Governs)**
|| Concrete breakout y-	**600**	**13286**	**0.05**	**Pass (Governs)**
Pryout	1200	36113	0.03	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.23	0.00	22.5%	1.0	Pass

5/8"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.291 inch

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 12.000
Code report: IAPMO UES ER-263
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.00
c_{ac} (inch): 23.52
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

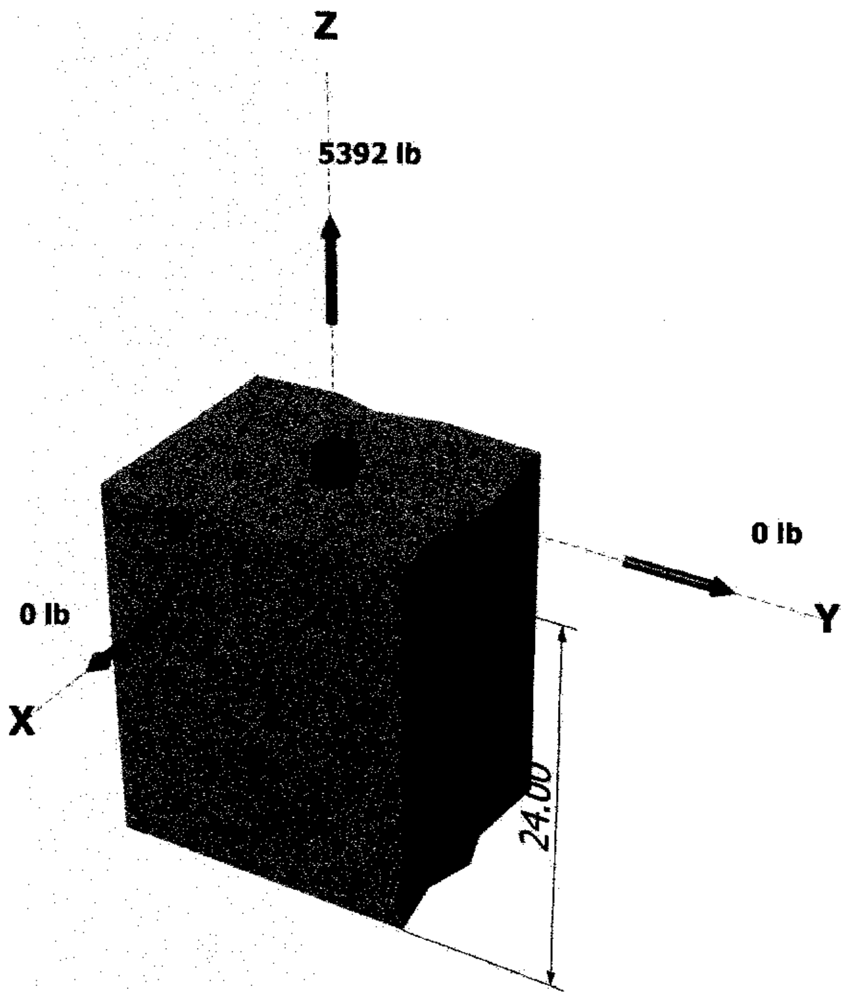
Recommended Anchor
Anchor Name: AT-XP® - AT-XP w/ 1"Ø F1554 Gr. 36
Code Report: IAPMO UES ER-263

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 5392
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



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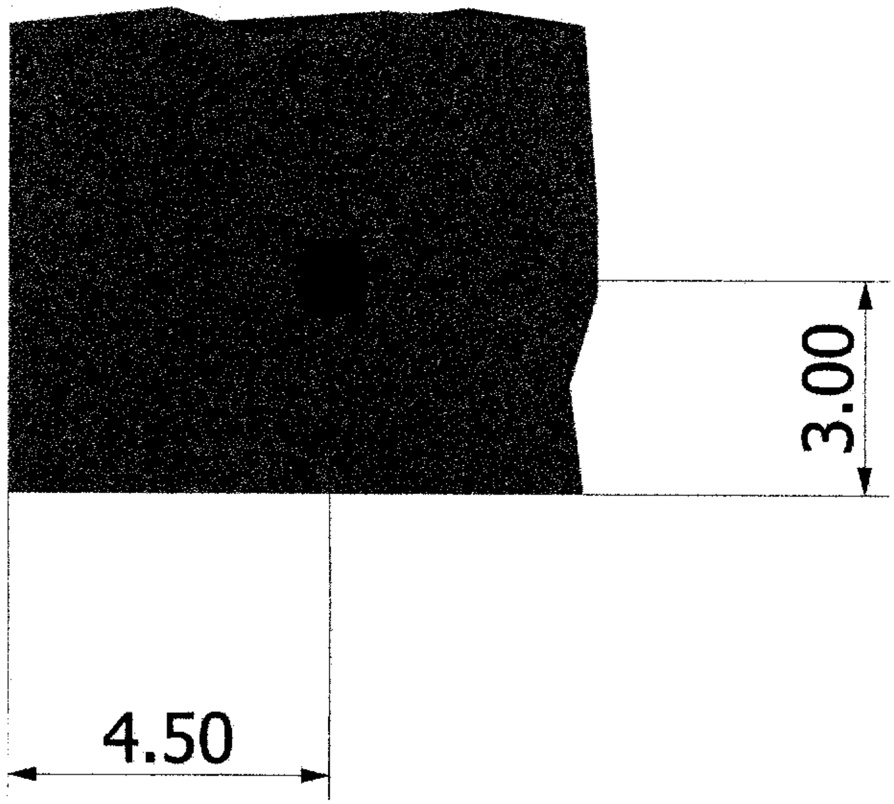


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	5392.0	0.0	0.0	0.0
Sum	5392.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 5392
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	12.000	44694

$\phi N_{cb} = \phi\ (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
472.50	1296.00	3.00	0.750	1.00	1.000	44694	0.75	9166

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
800	1.00	1.00	800

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	800	1.00	12.000	30159

$\phi N_a = \phi\ (A_{Na} / A_{Na0}) \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
255.95	601.82	12.27	3.00	0.773	1.000	30159	0.55	5456

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	5392	26363	0.20	Pass
Concrete breakout	5392	9166	0.59	Pass
Adhesive	**5392**	**5456**	**0.99**	**Pass (Governs)**

AT-XP w/ 1"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

SIMPSON Strong-Tie® **Anchor Designer™ Software** Version 2.8.7094.1

Company:	Gary E. Gill, SE	Date:	4/10/2020
Engineer:	GEG	Page:	1/5
Project:	Louden		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Anchor Bolt for SWA4 corner hold down
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 12.000
Code report: IAPMO UES ER-263
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 13.25
c_{ac} (inch): 19.92
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 30.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: Yes

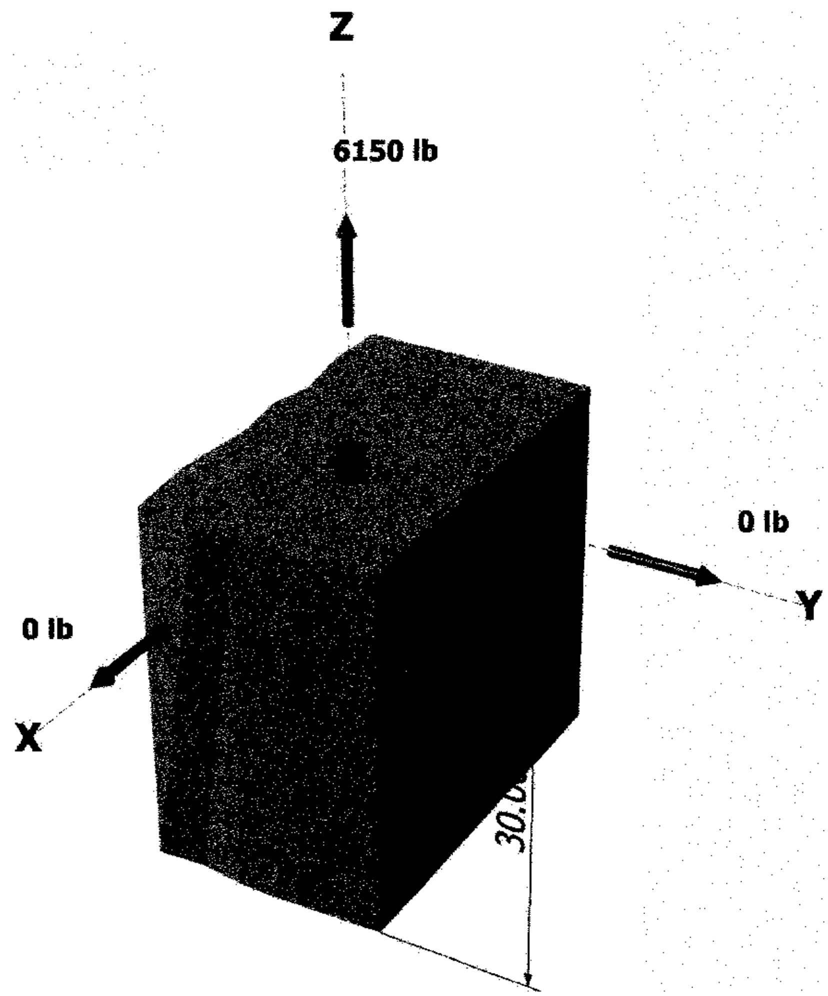
Recommended Anchor
Anchor Name: AT-XP® - AT-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: IAPMO UES ER-263

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

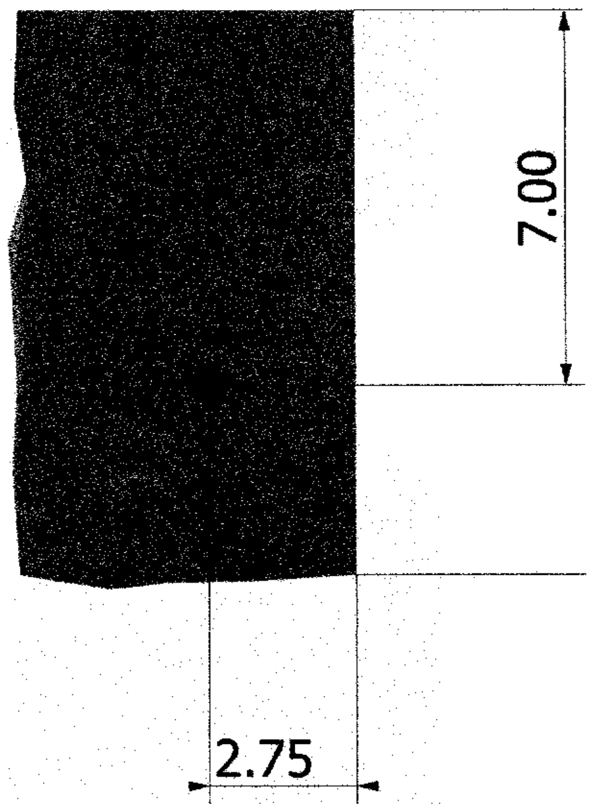
Strength level loads:

N_{ua} [lb]: 6150
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	6150.0	0.0	0.0	0.0
Sum	6150.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 6150
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	12.000	35334

$\phi N_{cb} = \phi\, (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
518.75	1296.00	2.75	0.746	1.00	1.000	35334	0.65	6856

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
980	1.00	1.00	980

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	980	0.63	12.000	23091

$\phi N_a = \phi\, (A_{Na} / A_{Na0}) \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
156.24	243.61	7.80	2.75	0.806	1.000	23091	0.65	7756

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

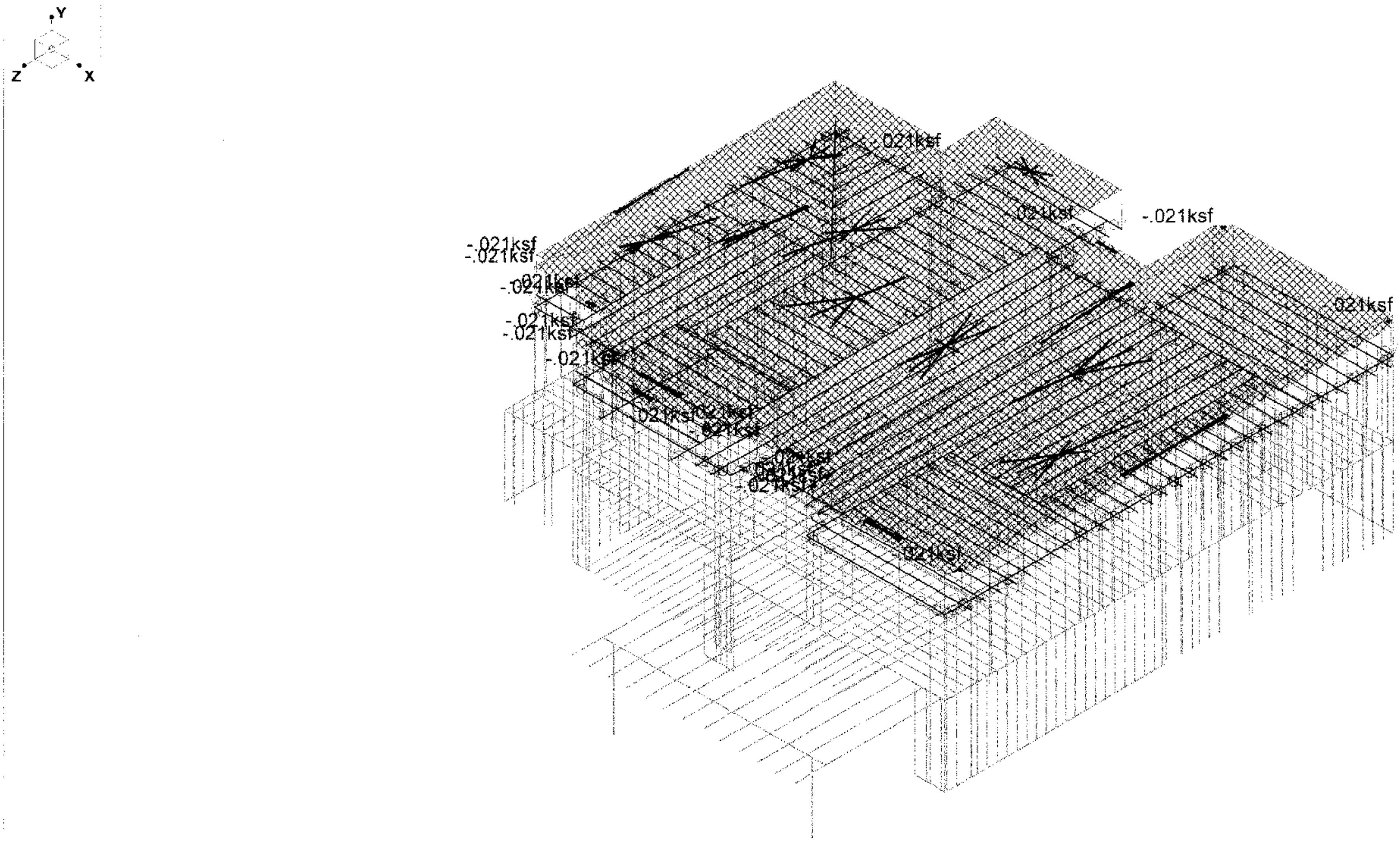
Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	6150	9833	0.63	Pass
Concrete breakout	**6150**	**6856**	**0.90**	**Pass (Governs)**
Adhesive	6150	7756	0.79	Pass

AT-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.



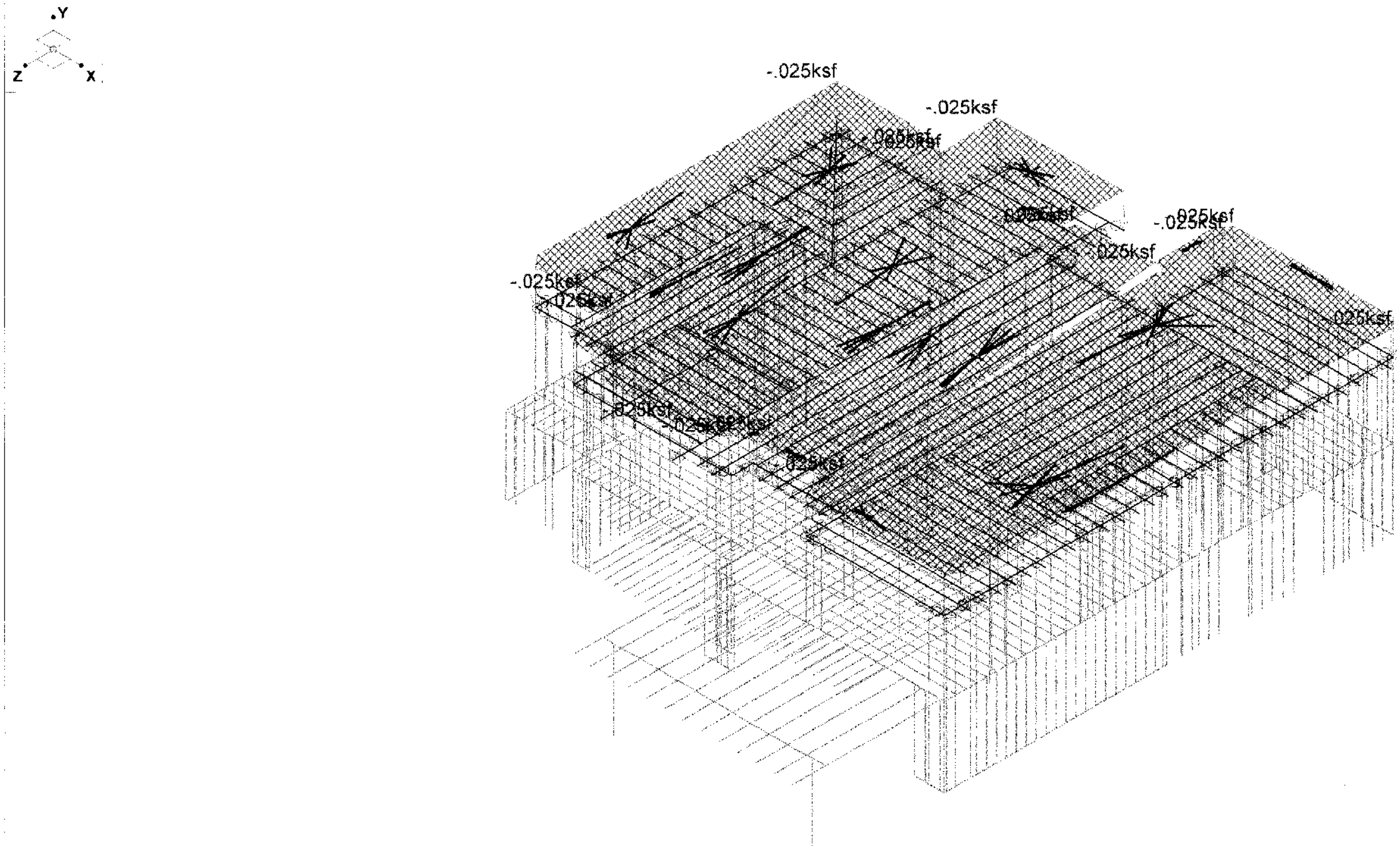
Loads: BLC 1, dead
Results for LC 4, d+.75sn+.75l
Reaction and Moment Units are k and k-ft

Roof Dead Load

Framing - 1

Apr 13, 2020 at 1:40 PM

louden risa model 4-9-20 3.r3d



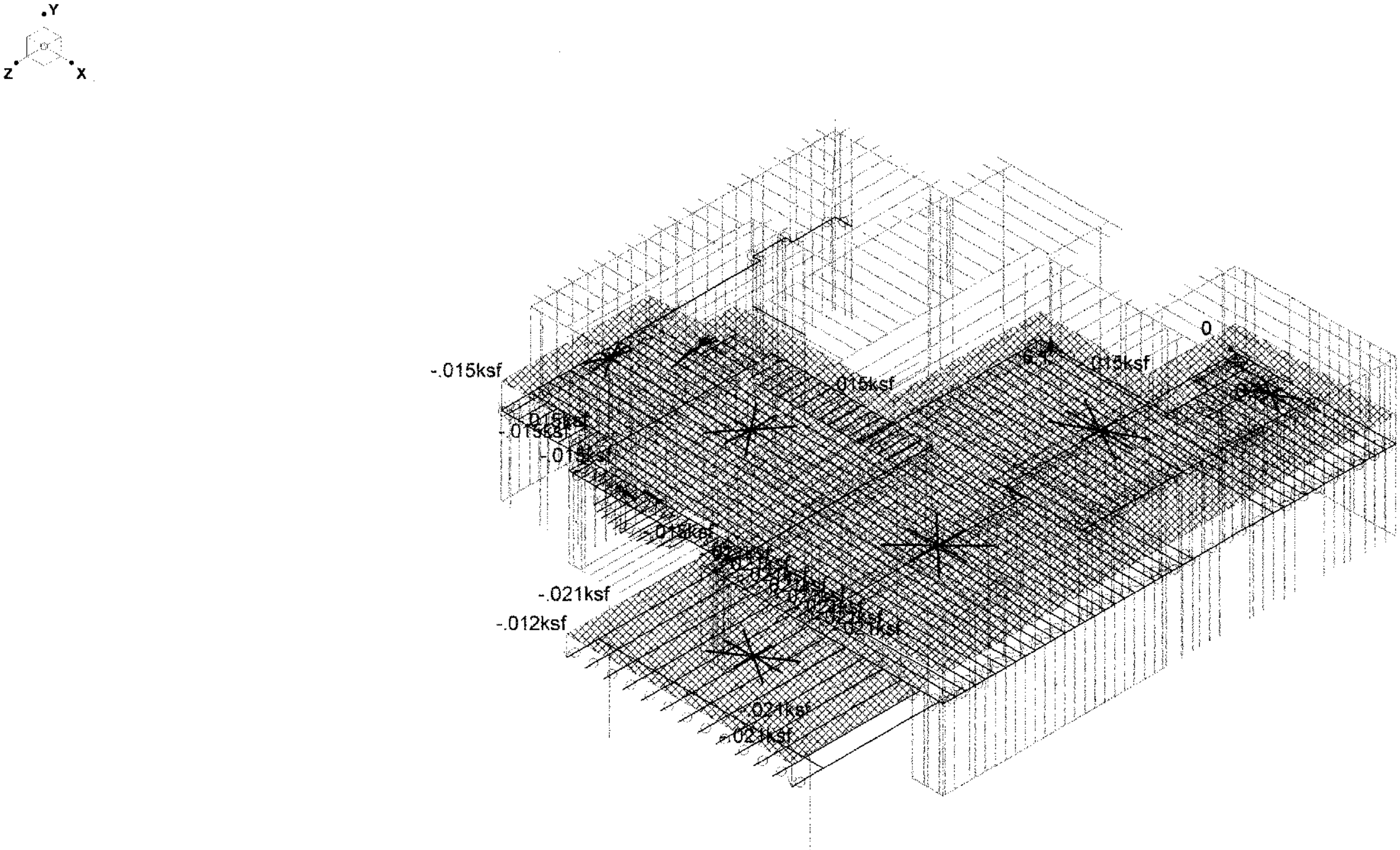
Loads: BLC 2, snow
Results for LC 4, d+.75sn+.75l
Reaction and Moment Units are k and k-ft

Roof Snow Load

Framing - 2

Apr 13, 2020 at 1:41 PM

louden risa model 4-9-20 3.r3d



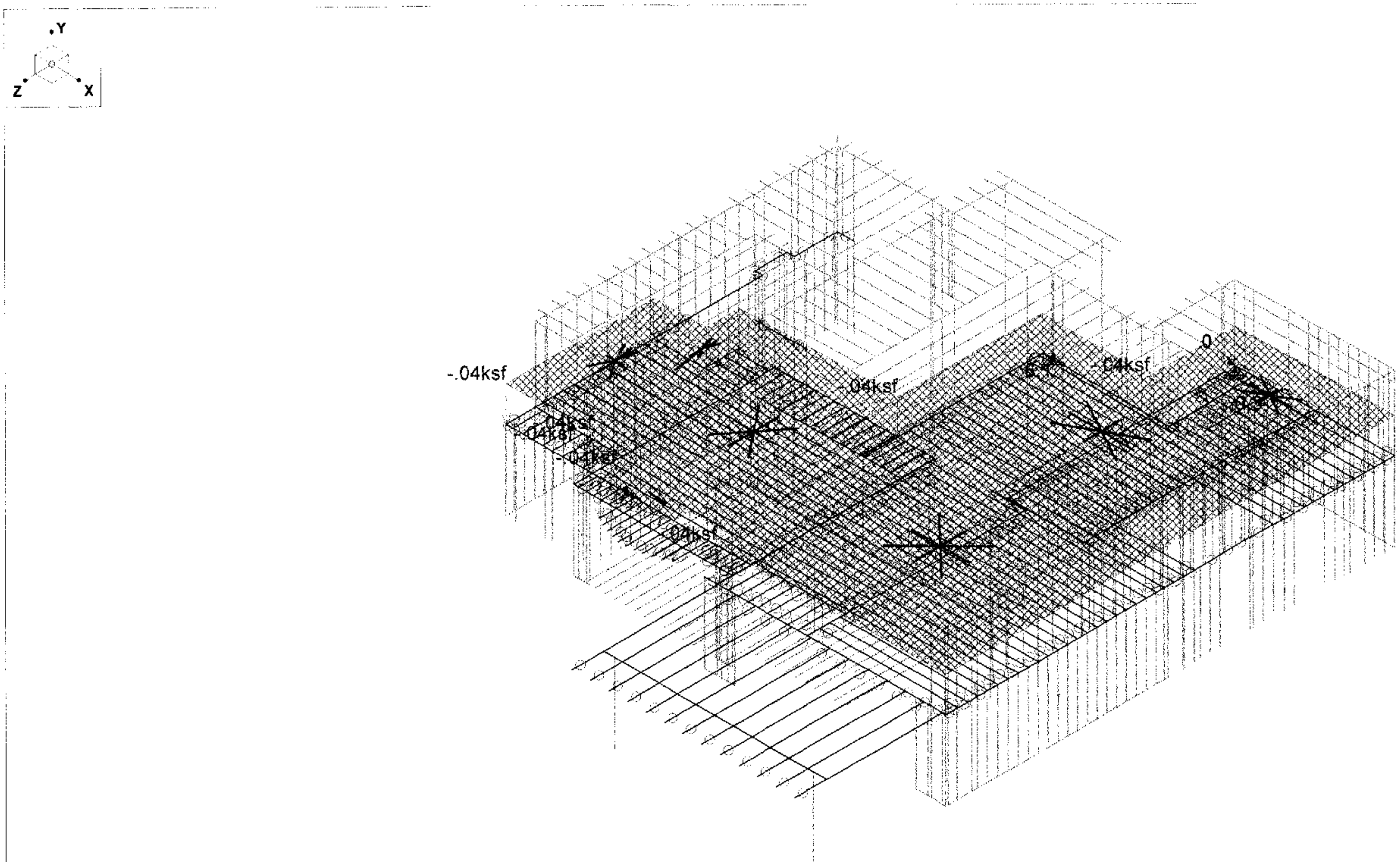
Loads: BLC 1, dead
Results for LC 4, d+.75sn+.75l
Reaction and Moment Units are k and k-ft

2nd Level Dead Load

Framing - 3

Apr 13, 2020 at 1:43 PM

louden risa model 4-9-20 3.r3d



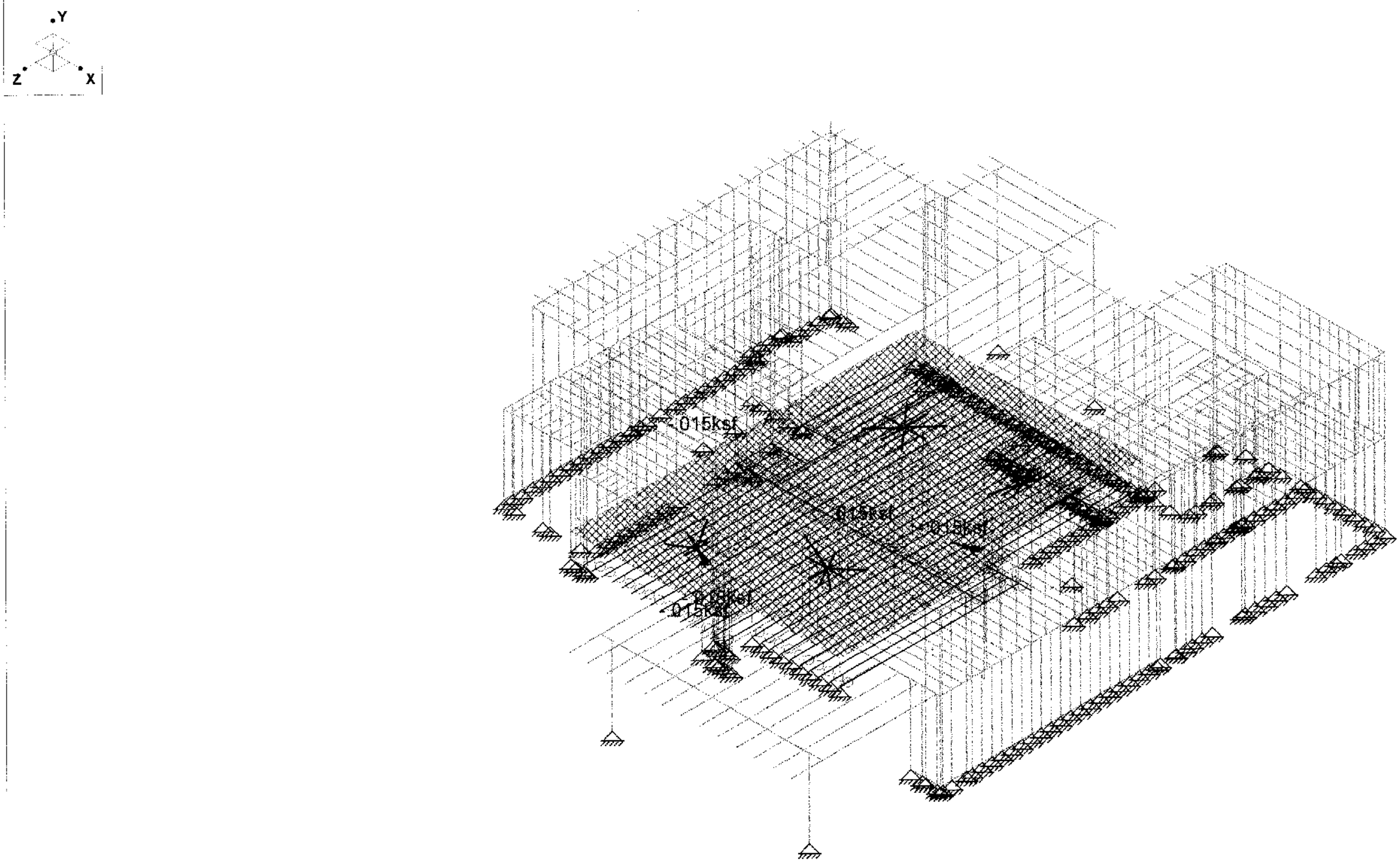
Loads: BLC 3, live
Results for LC 4, d+.75sn+.75l
Reaction and Moment Units are k and k-ft

Framing - 4

Apr 13, 2020 at 1:44 PM

louden risa model 4-9-20 3.r3d

2nd Level Live Load



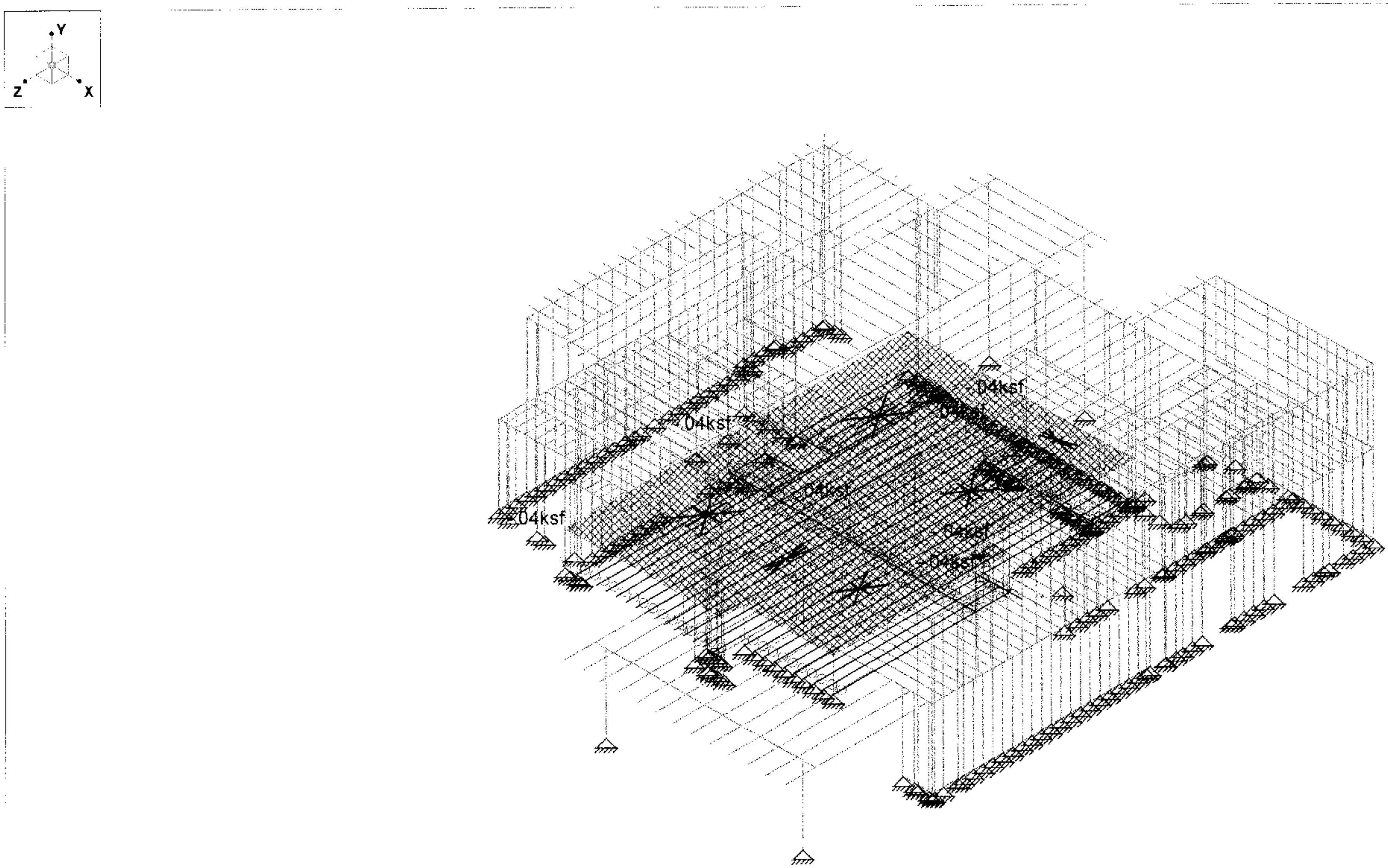
Loads: BLC 1, dead

Main Level Dead Load

Framing - 5

Apr 13, 2020 at 1:45 PM

louden risa model 4-9-20 3.r3d



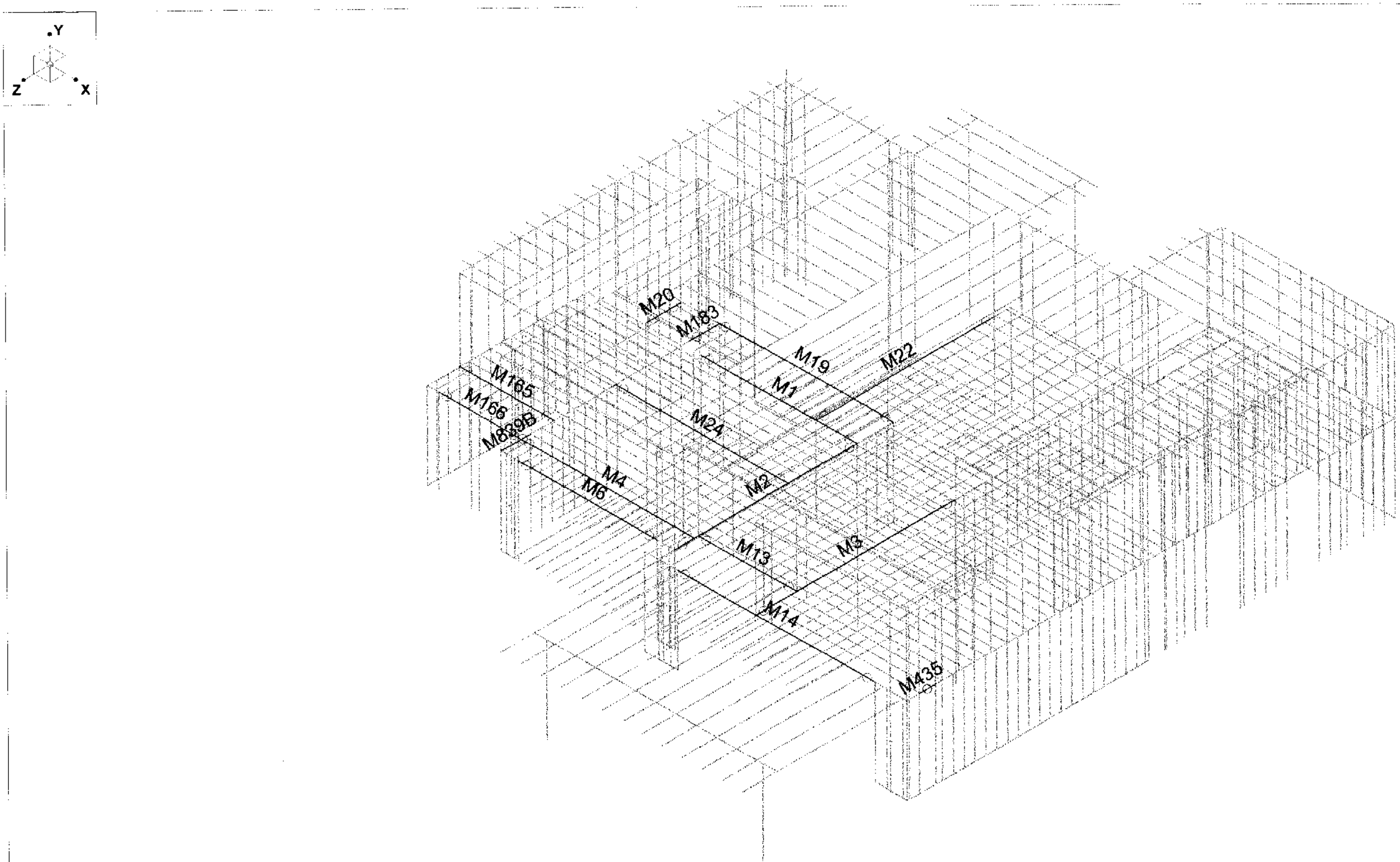
Loads: BLC 3, live

Main Level Live Load

Framing - 6

Apr 13, 2020 at 1:46 PM

louden risa model 4-9-20 3.r3d

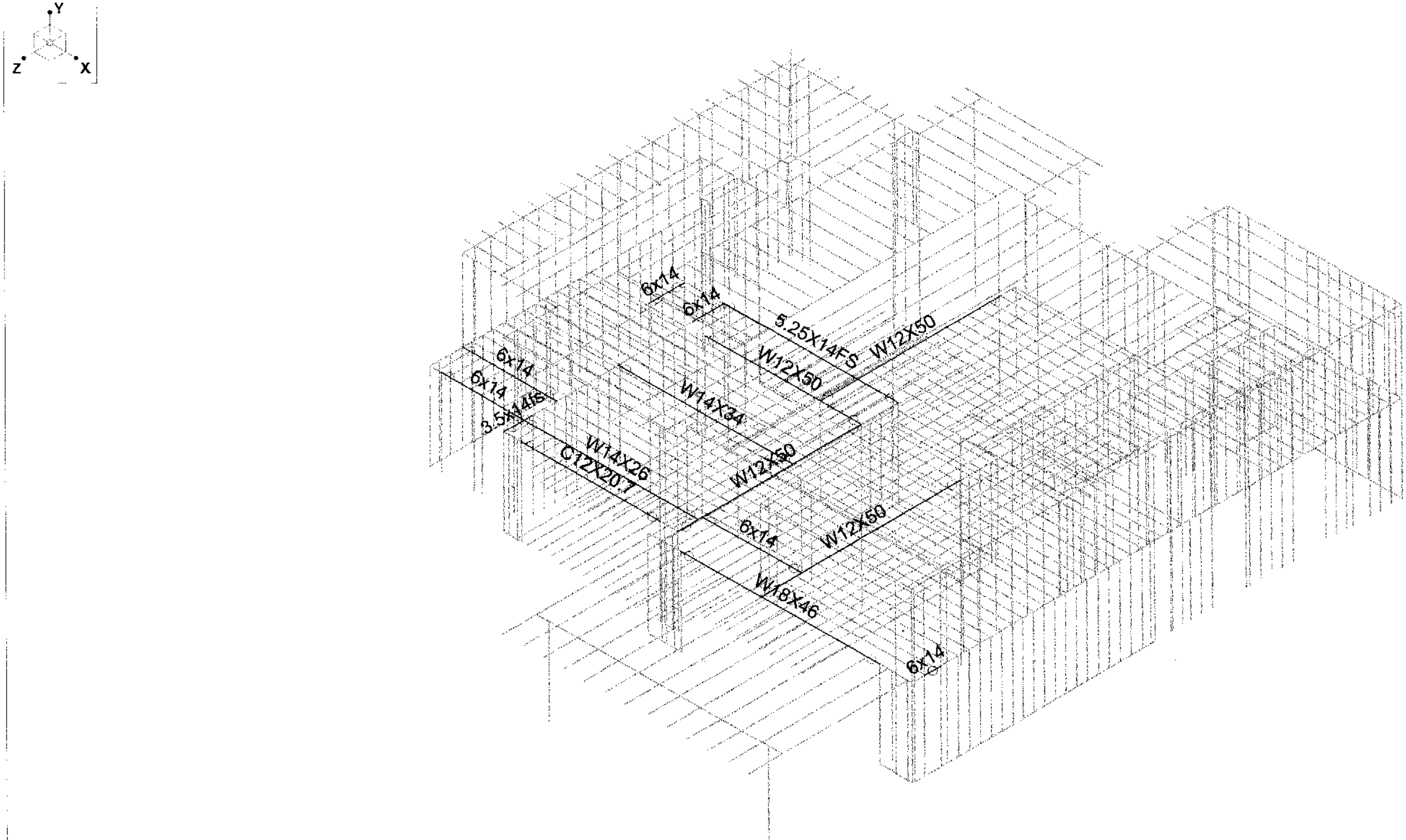


2nd Level Member Labels

Framing - 7

Apr 13, 2020 at 1:49 PM

louden risa model 4-9-20 3.r3d

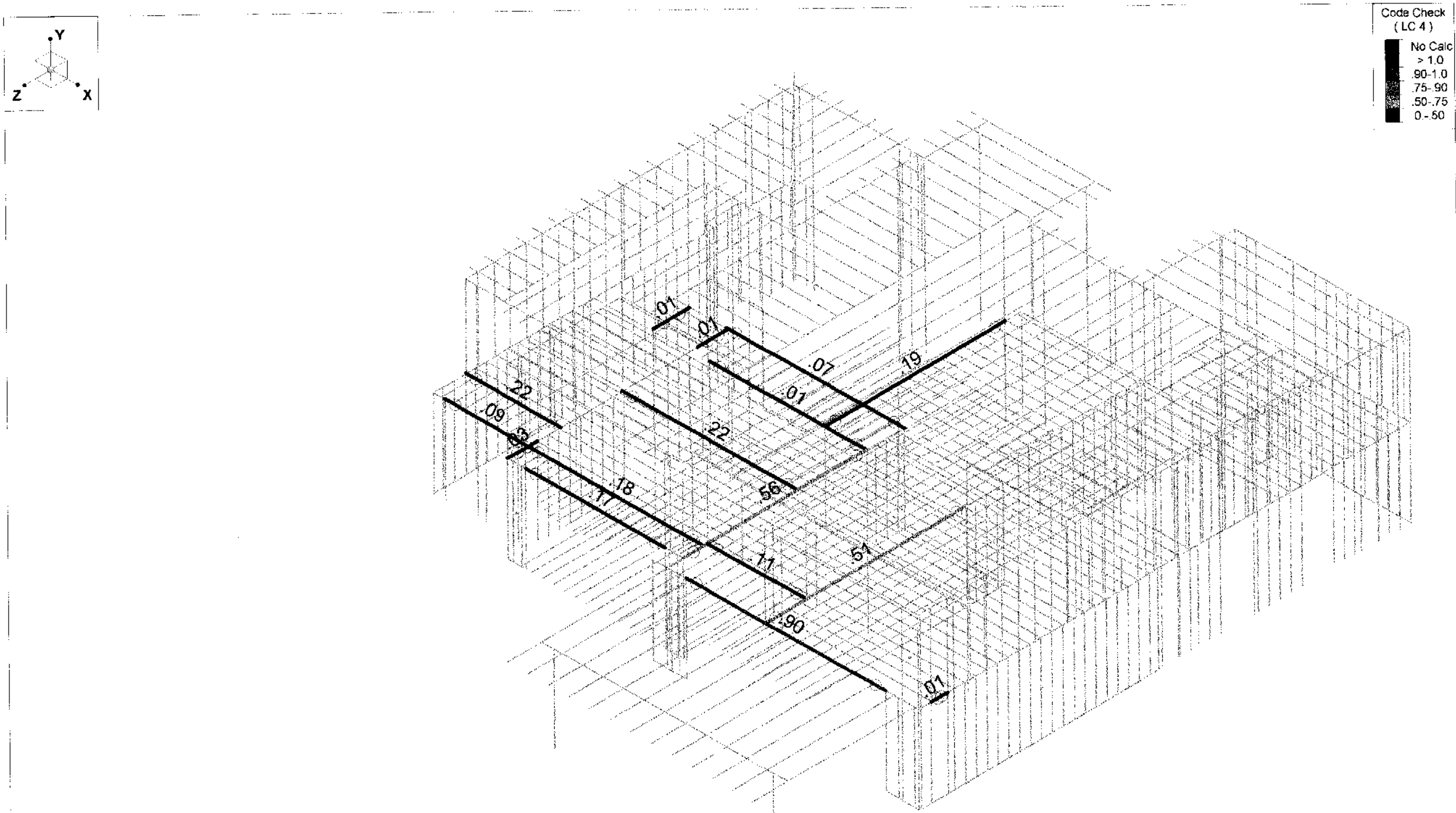


2nd Level Member Sizes

Framing - 8

Apr 13, 2020 at 1:50 PM

louden risa model 4-9-20 3.r3d



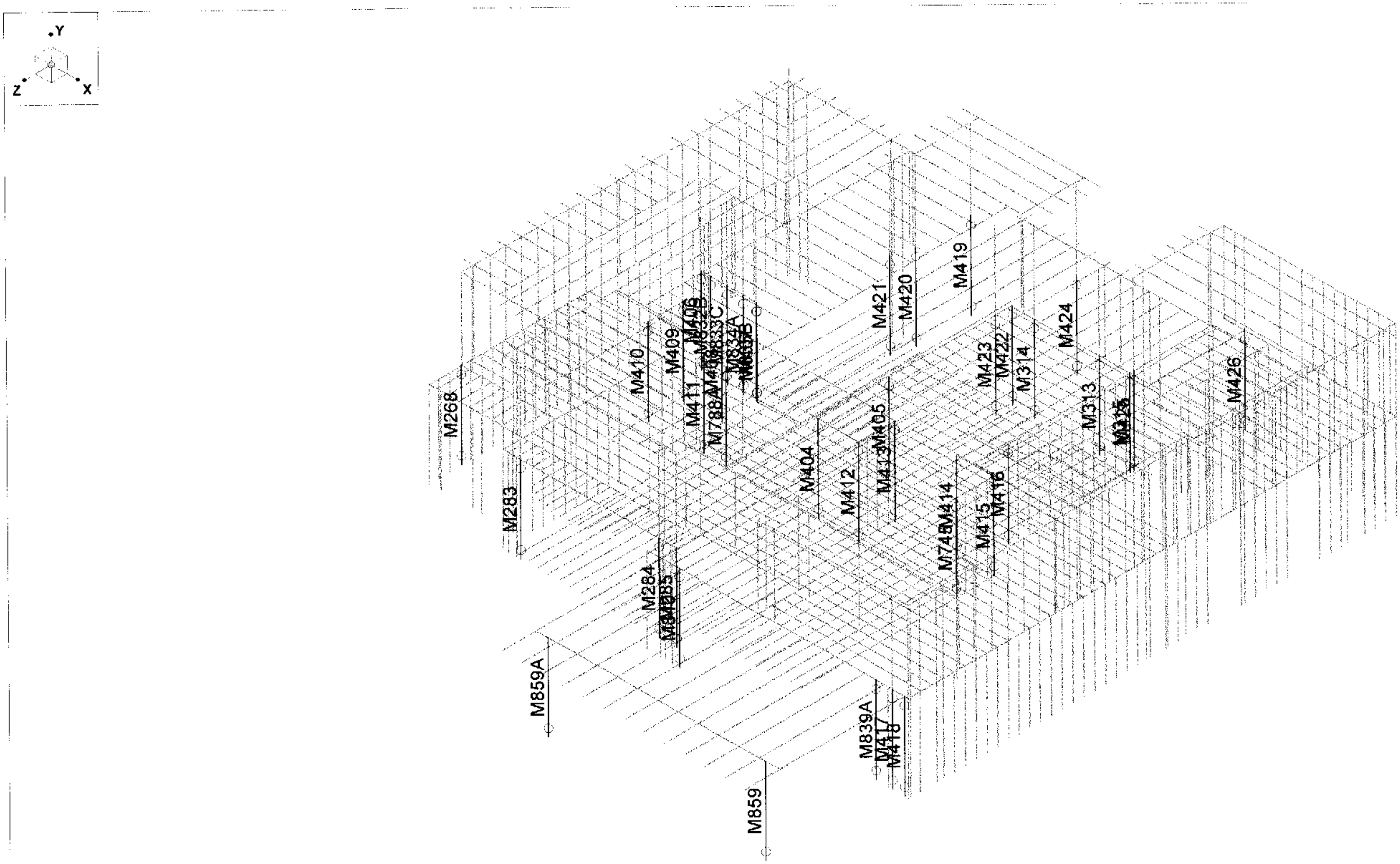
Member Code Checks Displayed
Results for LC 4, d+.75sn+.75l

2nd Level Member Stress Check

Framing - 9

Apr 13, 2020 at 1:52 PM

louden risa model 4-9-20 3.r3d

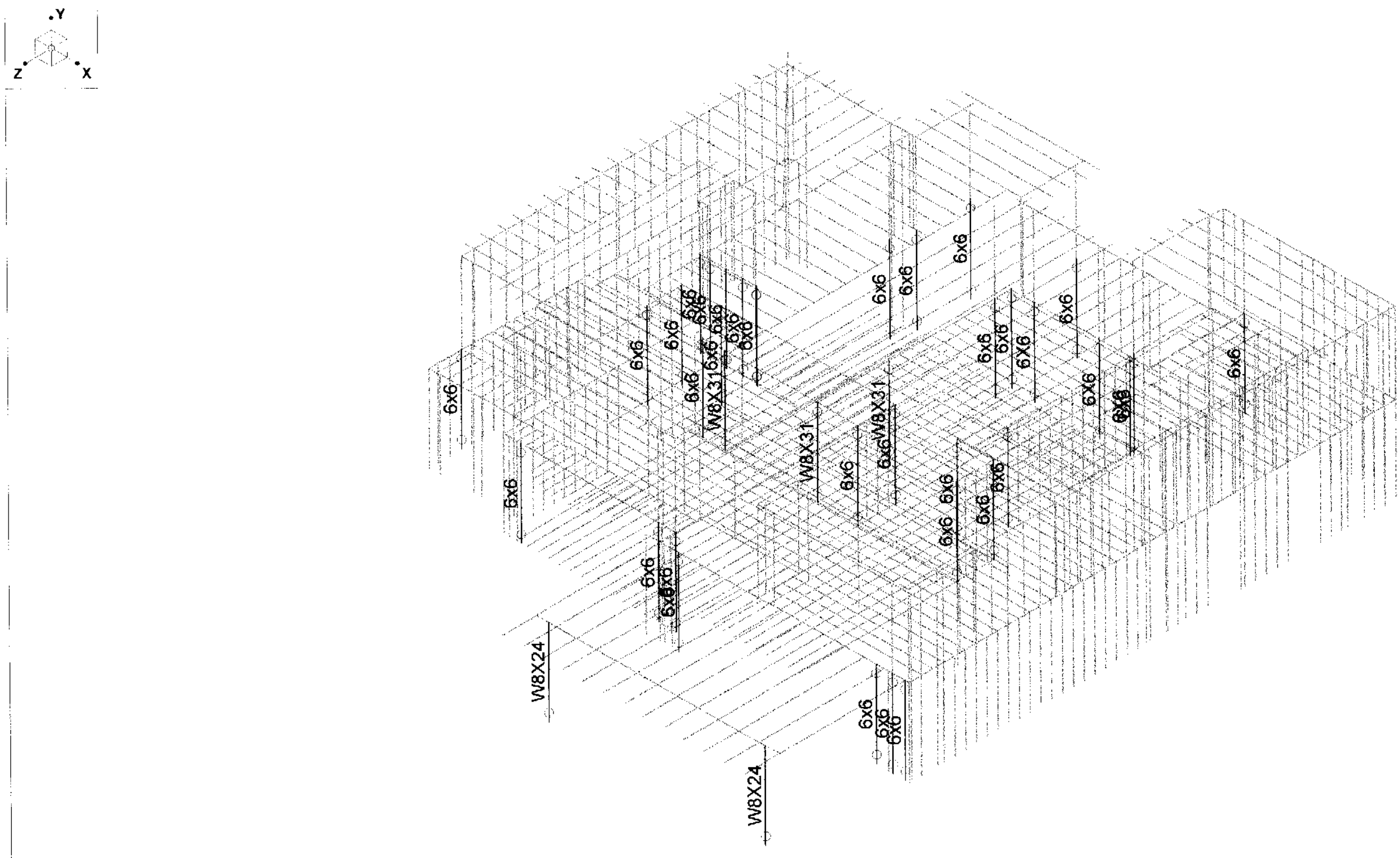


Mainf Level Column Labels

Framing - 10

Apr 13, 2020 at 1:54 PM

louden risa model 4-9-20 3.r3d

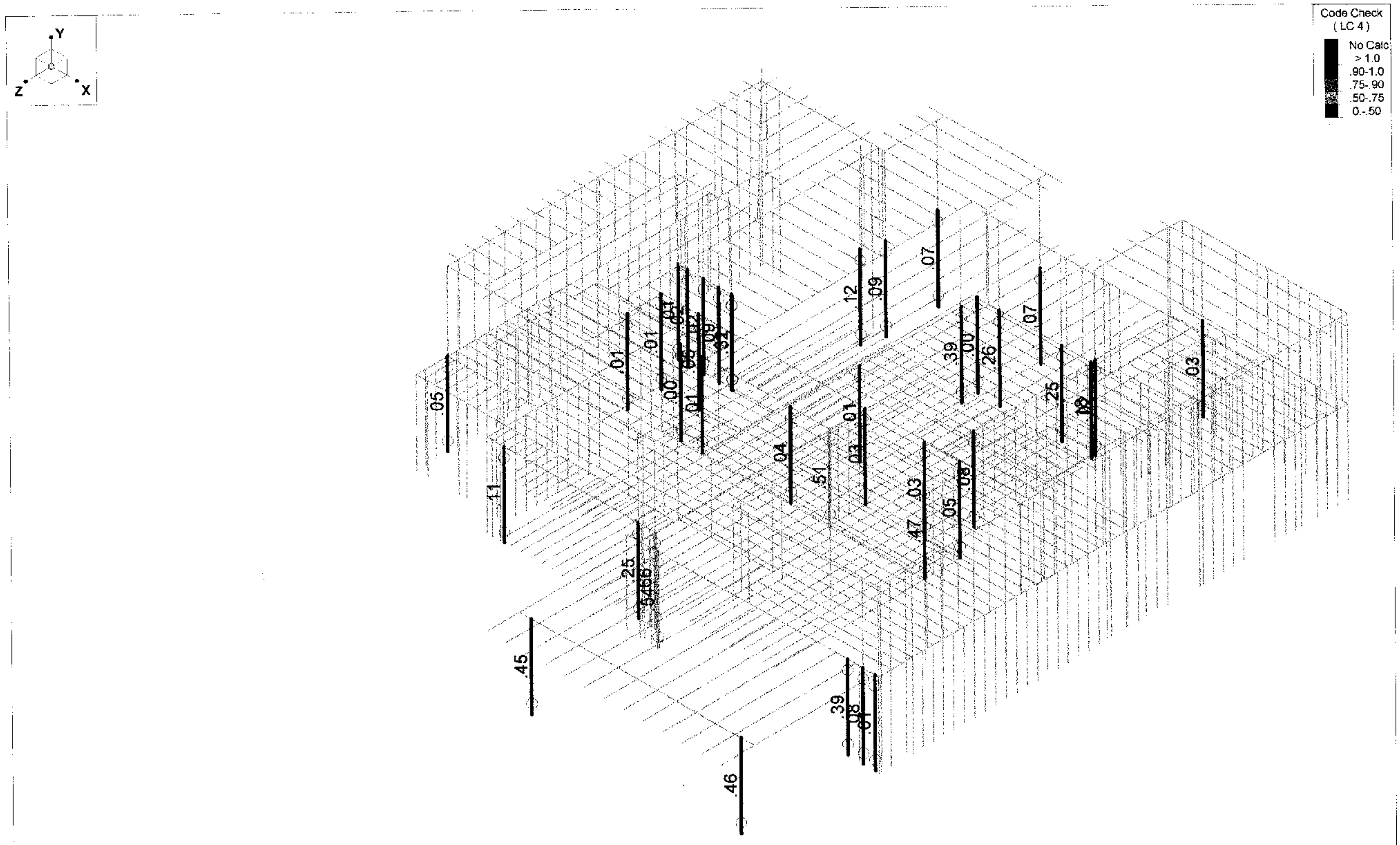


Main Level Column Sizes

Framing - 11

Apr 13, 2020 at 1:55 PM

louden risa model 4-9-20 3.r3d



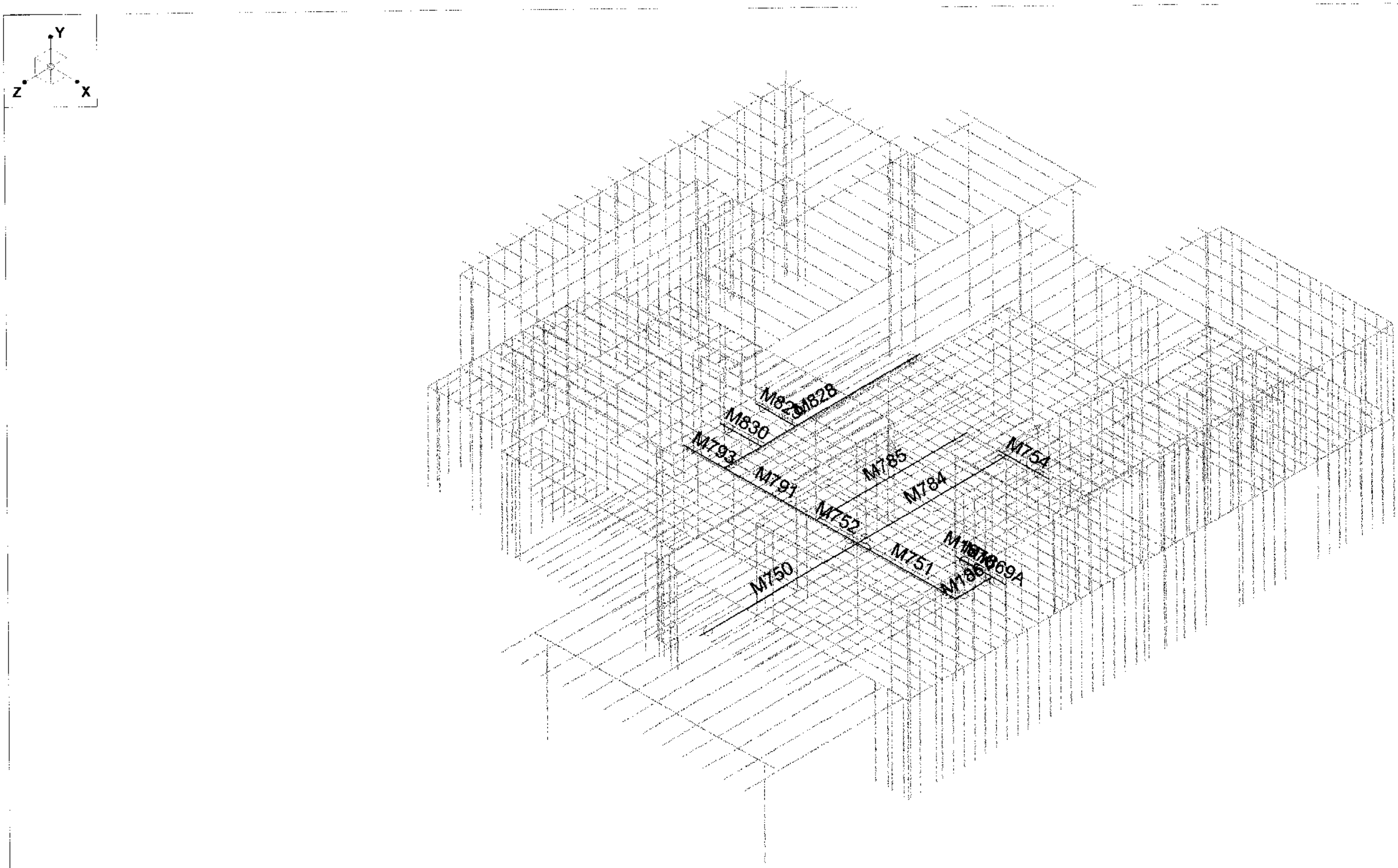
Member Code Checks Displayed
Results for LC 4, d+.75sn+.75l

Main Level Column Stress Check

Framing - 12

Apr 13, 2020 at 1:56 PM

louden risa model 4-9-20 3.r3d

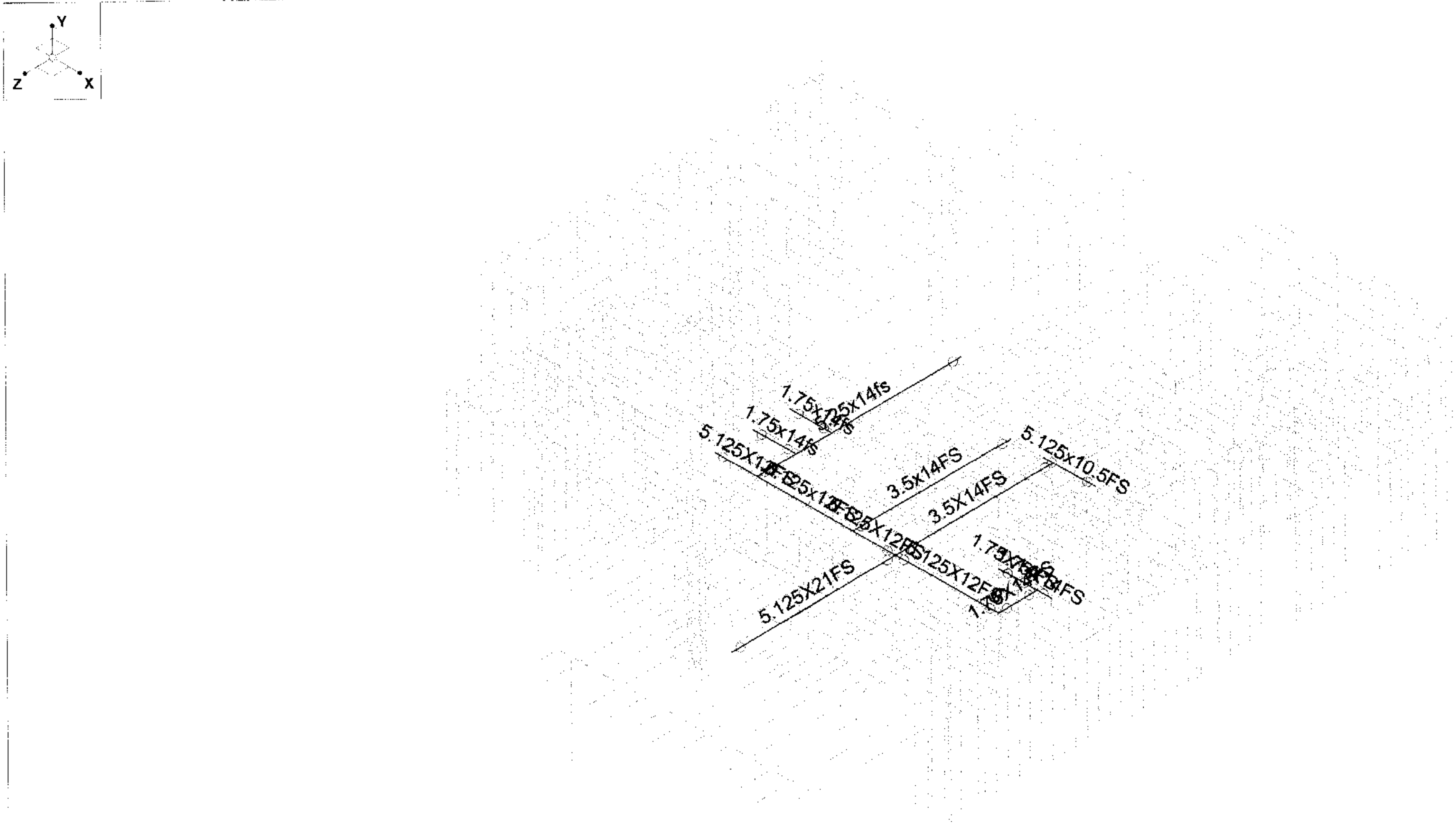


Main Level Beam Labels

Framing - 13

Apr 13, 2020 at 1:57 PM

louden risa model 4-9-20 3.r3d

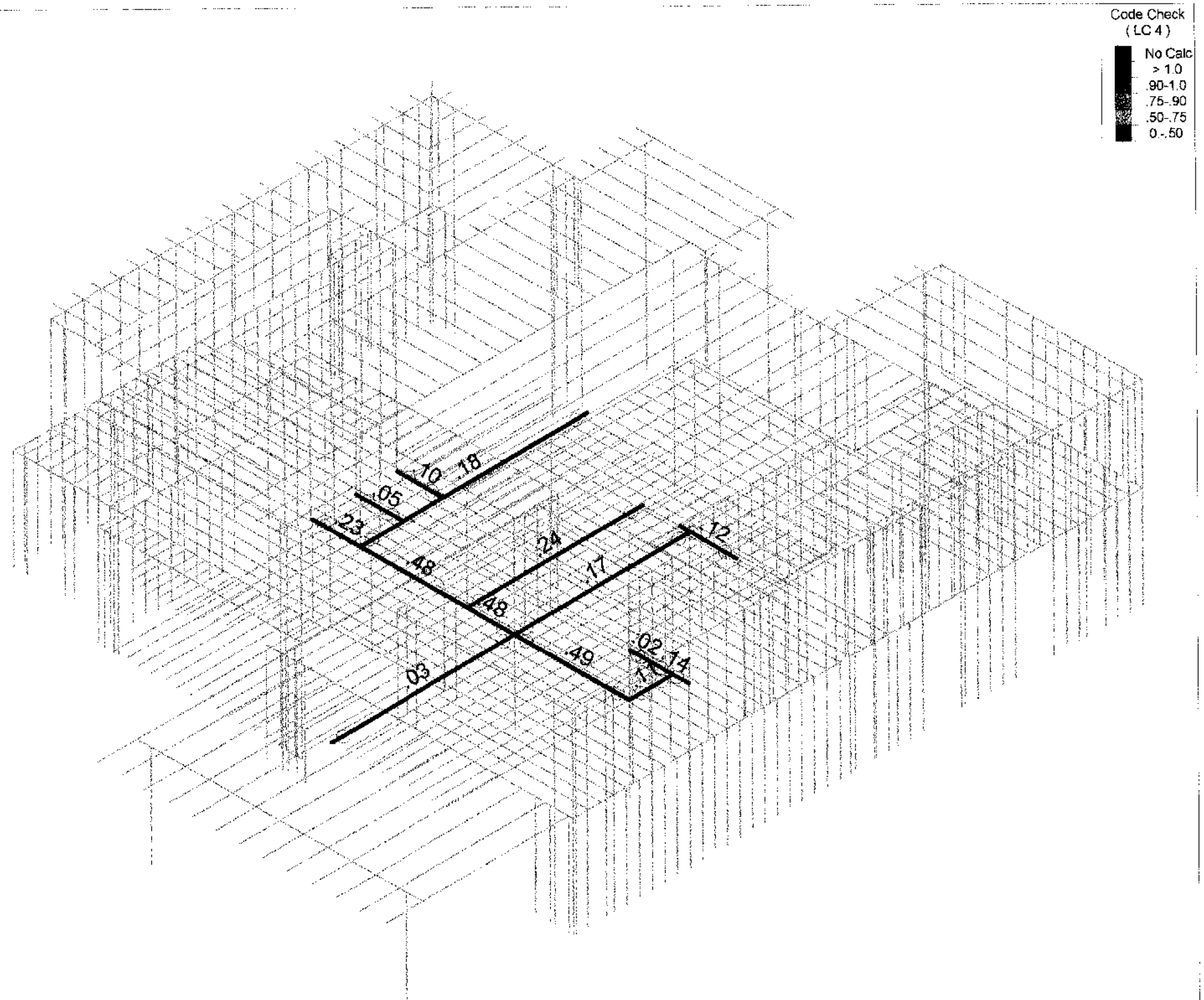


Main Level Beam Sizes

Framing - 14

Apr 13, 2020 at 1:58 PM

louden risa model 4-9-20 3.r3d



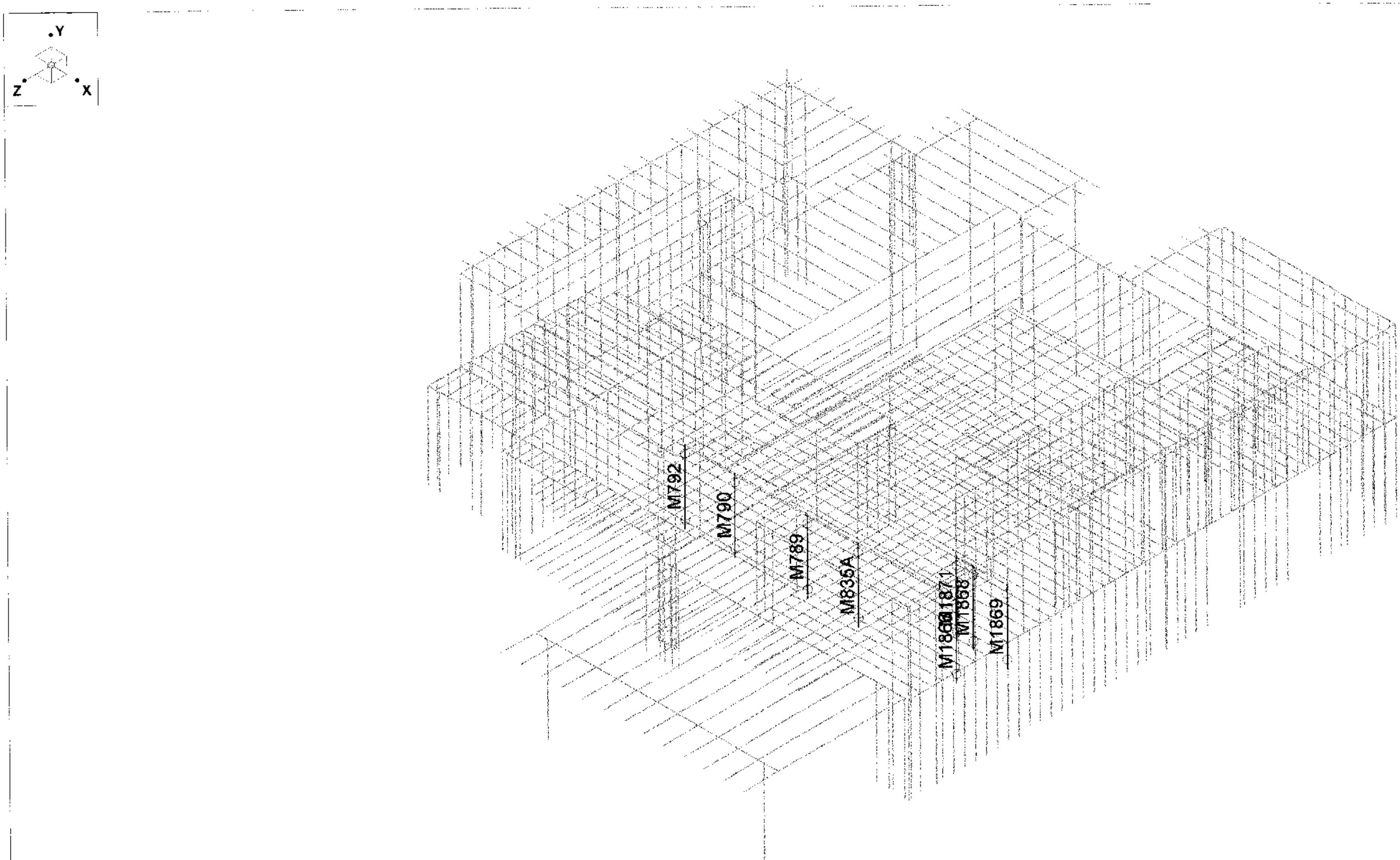
Member Code Checks Displayed
Results for LC 4, d+.75sn+.75l

Main Level Beam Stress Check

Framing - 15

Apr 13, 2020 at 1:59 PM

louden risa model 4-9-20 3.r3d

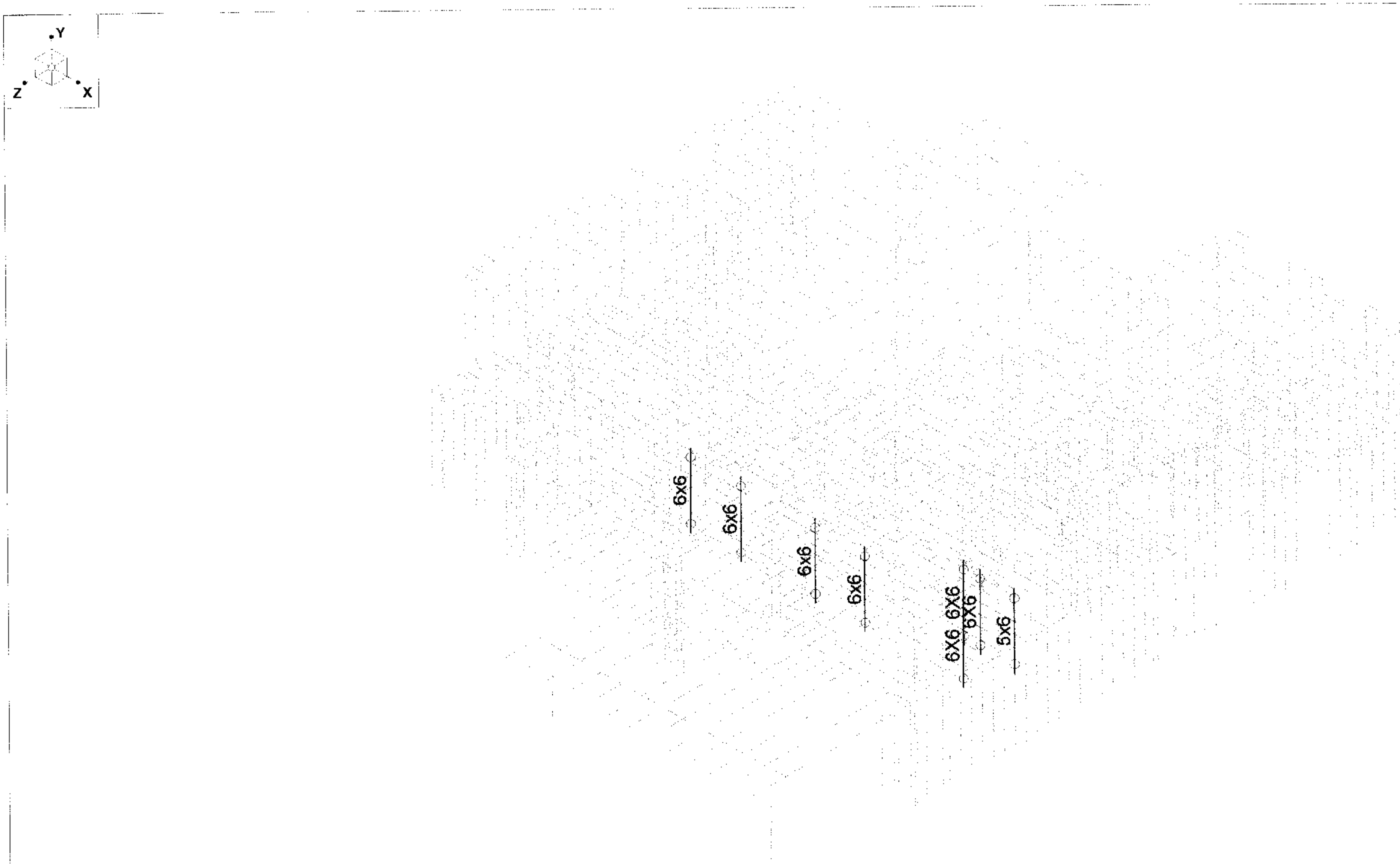


Basement Coumn Labels

Framing - 16

Apr 13, 2020 at 2:00 PM

louden risa model 4-9-20 3.r3d

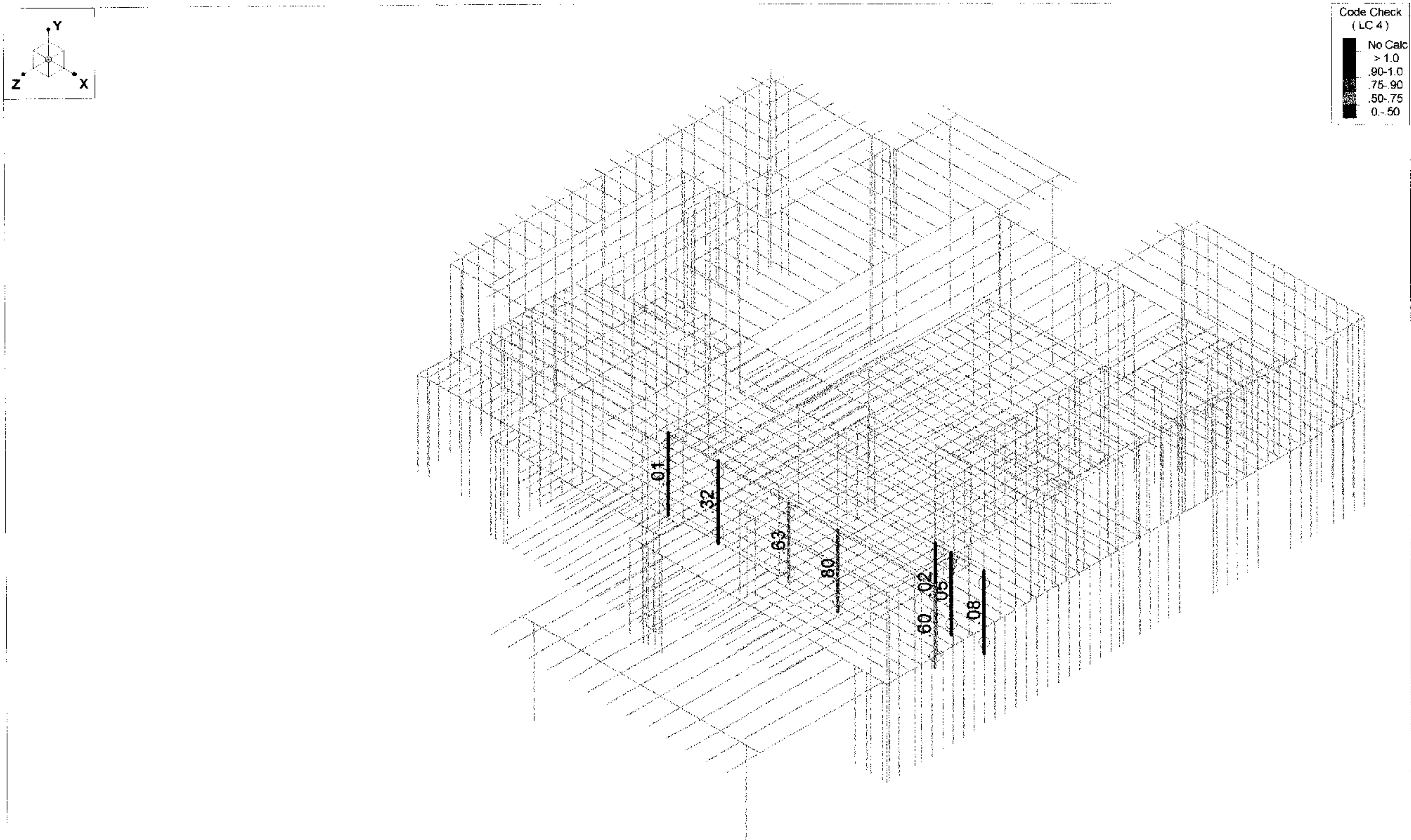


Basement Level Column Sizes

Framing - 17

Apr 13, 2020 at 2:01 PM

louden risa model 4-9-20 3.r3d



Member Code Checks Displayed
Results for LC 4, d+.75sn+.75l

Basement Column Stress Check

Framing - 18

Apr 13, 2020 at 2:02 PM

louden risa model 4-9-20 3.r3d