SEIZMIC INC.

MATERIAL HANDLING ENGINEERING

EST. 1985

SPECIAL PRODUCTS	*CONVEYORS*	*STORAGE RACKS*	*OTHER SERVICES*	*SHELVING*	*SPECIALS PRODUCTS*
TANK SUPPORTS	TALL SUPPORTS	SELECTIVE	SEISMIC ANALYSIS	METAL	SHUTTLES
MACHINERY	HEADER STEEL	DRIVE-IN	PERMIT ACQUISITION	METAL/WOOD	VLM
RACK BLDGS	SORT PLATFORM	PUSH BACK	EGRESS PLANS	MOVABLE	CAROUSELS
SHEDS	PICK MODULES	FLOW RACK	STATE APPROVALS	GONDOLAS	VRC
MEZZANINES	ROOF VERIFICATION	CANTILEVER	PRODUCT TESTING	LOCKERS	MODULAR OFFICES
FOOTINGS				CATWALKS	FENCES

Licensed In All 50 States

PRCTI20251723

Seismic Analysis of

Light Duty Storage Fixtures

for

AutoZone #10668

4423 South Meridian

Puyallup, WA 98373

Job No. 26-0165

Approved by:

Sal E. Fateen, P.E.

1/28/2026

City of Puyallup Building REVIEWED FOR COMPLIANCE

SKinnear

03/02/2026

8:36:25 AM

CITY OF PUYALLUP STATE OF WASHINGTON

City of Puyallup Development & Permitting Services

ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

SAL E. FATEEN STATE OF WASHINGTON #29554 REGISTERED PROFESSIONAL ENGINEER

Digitally signed by Sal Fateen
Date: 2026.01.30 06:55:45-08'00'

EXPIRES
04-12-2026

Calculations required to be provided by the Permittee on site for all Inspections

1130 E Cypress Street • Covina, CA 91724 • Tel : 909-869-0989

SEIZMIC INC.
MATERIAL HANDLING ENGINEERING
TEL: (909) 869-0989
1130 E. CYPRESS STREET, COVINA, CA 91724

PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	2
CALCULATED BY	KE
DATE	1/28/2026

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SEIZMIC
INC.

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1130 E. CYPRESS STREET, COVINA, CA 91724

PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	3
CALCULATED BY	KE
DATE	1/28/2026

SCOPE:

THE PURPOSE OF THIS ANALYSIS IS TO SHOW THAT THE FOLLOWING LIGHT DUTY STORAGE FIXTURES ARE IN COMPLIANCE WITH SECTION 1613 OF THE 2021 IBC.

PARAMETERS:

LATERAL FORCE:

$$V = Cs * Ws \quad \text{WHERE } Cs = S_{DS}/(R/I)$$
$$= [(2/3) * Fa * Ss * I/R] * Ws$$

SPECIFICATIONS:

MAIN STEEL	Fy = 36000 PSI MIN. YIELD STEEL
BOLTS	A307 (WHEN USED)
ANCHORS	3/8"Ø x 2-3/8" MIN. EMBED. HILTI KB1 (IAPMO #UES ER-678)
SLAB	4 IN (minimum required) x 2500 PSI (minimum required)
SOIL	1000 PSF

CONFIGURATIONS:

GRAVITY FEED FIXTURE

*** TYPE D/E**	**84"H x 60"W x 38"D**	**FREE STANDING**

GONDOLA FIXTURE

TYPE B1	90"H x 48"W x 16"D	FREE STANDING
*** TYPE N**	**144"H x 48"W x 16"D & 120"H x 48"W x 24"D**	**FREE STANDING**
TYPE L	144"H x 48"W x 13"D	ATTACHED
TYPE G1	114"H x 48"W x 16"D	FREE STANDING
TYPE C1	114"H x 48"W x 16"D	WALL BRACED

LIGHT DUTY STORAGE FIXTURE

*** TYPE H**	**144"H x 48"W x 12"D**	**WALL BRACED**
TYPE H-SGL	144"H x 48"W x 12"D	FREE STANDING
*** TYPE I**	**(2) 144"H x 48"W x 12"D**	**FREE STANDING**
TYPE J	144"H x 48"W x 24"D & 12"D	FREE STANDING
TYPE K	144"H x 48"W x 24"D	WALL BRACED
TYPE K-SGL	144"H x 48"W x 24"D	FREE STANDING
TYPE M	(2) 144"H x 48"W x 24"D	WALL BRACED

WIDESPAN LIGHT DUTY STORAGE FIXTURE

*** TYPE W**	**120"H x 96"W x 48"D**	**FREE STANDING**

** ONLY CRITICAL CONFIGURATIONS BEING ANALYZED*

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	4
CALCULATED BY	KE
DATE	1/28/2026

GRAVITY FEED FIXTURE ANALYSIS

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	5
CALCULATED BY	KE
DATE	1/28/2026

LOADS & DISTRIBUTION: GRAVITY FEED FIXTURE (TYPE D/E)

SEISMIC SHEAR BASED ON SECTION 1613 OF THE 2021 IBC.

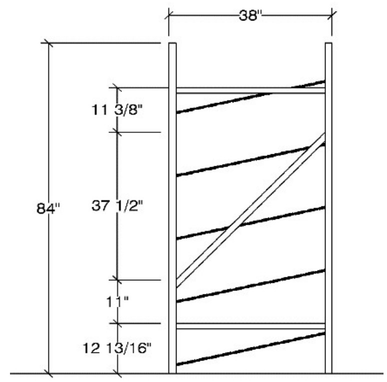
SITE CLASS = D

S_{MS} = 1.51 WHERE Sms = Fa*Ss AND Fa = 1.2, Ss = 1.26

S_{DS} = 1.01

I = **1.50** <=== IN PUBLIC ACCESS AREA

R = 4



SIDE VIEW

PRODUCT LOAD, wPL = 1,000 lb/LEVEL

DEAD LOAD, wDL = 50 lb/LEVEL

OF LVLS = 5

FULLY LOADED

E = [2/3*1.2*1.26*1.5*(0.67*5000 lb+250 lb)/4]

= 1361 lb

LATERAL FORCE DISTRIBUTION

h	LEVEL	WEIGHT	HEIGHT	W x H	Fi	Movt
6.0 in	1	1050 lb	6.0 in	6,300 in-lb	43 lb	258 in-lb
16.0 in	2	1050 lb	22.0 in	23,100 in-lb	158 lb	3,466 in-lb
16.0 in	3	1050 lb	38.0 in	39,900 in-lb	272 lb	10,342 in-lb
16.0 in	4	1050 lb	54.0 in	56,700 in-lb	387 lb	20,885 in-lb
16.0 in	5	1050 lb	70.0 in	73,500 in-lb	501 lb	35,094 in-lb
70.0 in		5250 lb	∑ =	199,500 in-lb	1361 lb	70,045 in-lb

TOP SHELF LOADED ONLY

E_{top} = [2/3*1.2*1.26*1.5*(1000 lb+250 lb)/4]

= 473 lb

LATERAL FORCE DISTRIBUTION

h	LEVEL	WEIGHT	HEIGHT	W x H	Fi	Movt
6.0 in	1	50 lb	6.0 in	300 in-lb	2 lb	11 in-lb
16.0 in	2	50 lb	22.0 in	1,100 in-lb	7 lb	144 in-lb
16.0 in	3	50 lb	38.0 in	1,900 in-lb	11 lb	429 in-lb
16.0 in	4	50 lb	54.0 in	2,700 in-lb	16 lb	867 in-lb
16.0 in	5	1050 lb	70.0 in	73,500 in-lb	437 lb	30,579 in-lb
70.0 in		1250 lb	∑ =	79,500 in-lb	473 lb	32,029 in-lb

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	6
CALCULATED BY	KE
DATE	1/28/2026

OVERTURNING ANALYSIS: GRAVITY FEED FIXTURE (TYPE D/E)

$(0.6\text{-}0.11S_{DS})$*DL + 0.75*$(0.6\text{-}0.14S_{DS})$*PL - 0.75*0.7*EL

WORKING STRESS REDUCTION = 0.7

Fully Loaded Condition:

Allowable Tension = 830 lb
Allowable Shear = 900 lb
# of Anchors / Plate = 1
Depth = 38.0 in

Movt = 0.75*0.7*∑(Fi*Hi)
= 0.75*0.7*70045 in-lb
= 36,774 in-lb

Mst = [$(0.6\text{-}0.11S_{DS})$*DL + 0.75*$(0.6\text{-}0.14S_{DS})$*PL] x Depth / 2
= 35,019 in-lb

Puplift = (Movt - Mst) / Depth
= (36774 in-lb - 35019 in-lb) / 38 in
= 46 lb

Vcol = 0.75*0.7*E/2
= 357 lb

Interaction Eqn. **[46 lb/ 830 lb] + [357 lb/ 900 lb] =** **0.45** **< 1.2 Therefore, Ok**

Top Shelf Loaded Only:

Allowable Tension = 830 lb
Allowable Shear = 900 lb
# of Anchors / Plate = 1
Depth = 38.0 in

Movt = 0.75*0.7*∑(Fi*Hi)
= 0.75*0.7*32029 in-lb
= 16,815 in-lb

Mst = [$(0.6\text{-}0.11S_{DS})$*DL + 0.75*$(0.6\text{-}0.14S_{DS})$*PL] x Depth / 2
= 8,862 in-lb

Puplift = (Movt - Mst) / Depth
= (16815 in-lb - 8862 in-lb) / 38 in
= 209 lb

Vcol = 0.75*0.7*E_{top}/2
= 124 lb

Interaction Eqn. **[209 lb/ 830 lb] + [124 lb/ 900 lb] =** **0.39** **< 1.2 Therefore, Ok**

USE (1) 3/8"Ø x 2-3/8" MIN. EMBED. HILTI KB1 (IAPMO #UES ER-678)
PER BASE PLATE.

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LOAD COMBINATIONS FOR SLAB ANALYSIS

		Resultant Load combination
1	1.4DL + 1.2PL	1.4DL + 1.2PL
2	1.2DL + 1.4PL + 1.6LL + 0.5(SL or RL)	1.2DL + 1.4PL
3	1.2DL + 0.85PL + 0.5LL + 1.6(SL or RL)	1.2DL + 0.85PL
4	1.2DL + 0.85PL + 0.5LL + 1.3WL + 0.5(SL or RL)	1.2DL + 0.85PL
5	$(1.2+0.2S_{DS})DL + (0.85+0.2S_{DS})PL + EL$	

DL-total/col = 125 lb
PL-total/col = 2,500 lb

EL = Movt/Depth
= 1,843 lb

Load combination 1

Pmax = 1.4DL + 1.2PL
= 1.4 x 125 lb + 1.2 x 2500 lb
= 3,175 lb

Load combination 2

Pmax = 1.2DL + 1.4PL
= 1.2 x 125 lb + 1.4 x 2500 lb
= 3,650 lb

Load combination 3 & 4

Pmax = 1.2DL + 0.85PL
= 1.2 x 125 lb + 0.85 x 2500 lb
= 2,275 lb

Load combination 5

Pmax = $(1.2+0.2S_{DS})DL + (0.85+0.2S_{DS})PL + EL$
= 1.4 x 125 lb + 1.05 x 2500 lb + 1843 lb
= 4,647 lb

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SLAB AND SOIL ANALYSIS

The slab will be checked for puncture and bearing stress. If no puncture occurs, the slab is assumed to distribute the load over a larger area of the slab.

a) Puncture:

Pmax = **4,647 lb**

Fpunct = 2.66 x (F'c^0.5)
= 2.66 x (2500 psi)^0.5
= 133 psi
Apunct = [(Weff.+t/2)+(Deff.+t/2)] x 2 x t
= [(2 in + 4 in) + (3.25 in + 4 in)] x 2 x 4 in
= 106 in^2

fv/Fv = P/[Apunct x Fpunct]
= 4647 lb/[106 in^2 x 133 psi x 0.55] — **0.60 < 1.0 OK**

b) Bearing:

ØBn = 0.85 x Ø x f'c x A1
= 7,597 lb
Pu / ØBn = 4647 lb / 7597 lb — **0.61 < 1.0 OK**

c) Slab Tension

Asoil = P/[1.0 x fs]
= 4647 lb/[1.0 x 1000 psf/(144 in^2/ft^2)]
= 669 in^2
L = Asoil^0.5
= (669.24 in^2)^0.5
= 25.9 in
B = [Weff. x Deff.]^0.5 + t
= [2 in x 3.25 in]^0.5 + 4 in
= 6.6 in
b = (L-B)/2
= (25.87 in - 6.6 in)/2
= 9.6 in

Mconc = (w)(b^2)/2 = [(1.0)(fs)(b^2)]/[144 (in^2/ft^2) x 2]
= [1.0 x 1000 psi x (9.64 in)^2]/[144 (in^2/ft^2) x 2]
= 322 in-lb
Sconc = 1 in x (t^2)/6
= 1 in x (4 in)^2/6
= 2.67 in^3
Fconc = 5 x Ø x f'c^0.5
= 5 x 0.55 x (2500 psi)^0.5
= 137.5 psi
fb/Fb = Mconc/[Sconc x Fconc]
= 322.39 in-lb/[(2.67 in^3)(137.5 psi)] — **0.88 < 1.0 OK**

Base Plate:

Weff. = 2.00 in
Deff. = 3.25 in
A1 = 6.5 in^2

Concrete:

Thickness = 4.00 in
f'c = 2,500 psi

Soil:

fs = 1,000 psf

Ø= 0.55

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GONDOLA FIXTURES ANALYSIS

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	10
CALCULATED BY	KE
DATE	1/28/2026

LOADS & DISTRIBUTION: TYPE N (144"Hx48"Wx16"D/24"D)

SEISMIC SHEAR BASED ON SECTION 1613 OF THE 2021 IBC.

SITE CLASS = D

S_{MS} = 1.512 WHERE Sms = Fa*Ss AND Fa = 1.2, Ss = 1.26

S_{DS} = 1.008

I = 1.00

R = 3 <=== TABLE 15.4-2 (STEEL DISTRIBUTED MASS CANTILEVER STRUCTURE)

WORKING STRESS REDUCTION = 0.7

PRODUCT LOAD, wPL = 60 lb/LEVEL

DEAD LOAD, wDL = 10 lb/LEVEL

OF LEVELS = 7

LATERAL FORCE

E = [(2/3)*1.512*1*(0.67*840 LB+140 LB)/3]

= 236 lb

LATERAL FORCE DISTRIBUTION: DOUBLE SIDE LOADED

h	LEVEL	WEIGHT	HEIGHT	W x H	Fi (trans)	Movt
18.0 in	1	140 lb	18.0 in	2,520 in-lb	8 lb	139 in-lb
18.0 in	2	140 lb	36.0 in	5,040 in-lb	15 lb	554 in-lb
18.0 in	3	140 lb	54.0 in	7,560 in-lb	23 lb	1,247 in-lb
30.0 in	4	140 lb	84.0 in	11,760 in-lb	36 lb	3,018 in-lb
18.0 in	5	140 lb	102.0 in	14,280 in-lb	44 lb	4,451 in-lb
18.0 in	6	140 lb	120.0 in	16,800 in-lb	51 lb	6,160 in-lb
18.0 in	7	140 lb	138.0 in	19,320 in-lb	59 lb	8,147 in-lb
138 in			∑ =	77,280 in-lb	236 lb	23,717 in-lb

$Pcol_{DL}$ = 10 lb x 7 LEVELS x 2

= 140 lb

$Pcol_{PL}$ = 60 lb x 7 LEVELS x 2

= 840 lb

Mcol(seismic) = Movt

= 23,717 in-lb

SEIZMIC INC.

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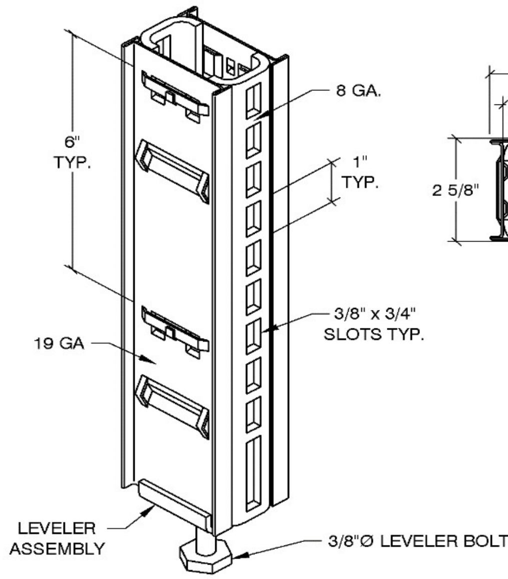
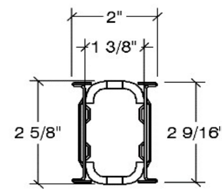
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	11
CALCULATED BY	KE
DATE	1/28/2026

COLUMN ANALYSIS: TYPE N (144"Hx48"Wx16"D/24"D)

Pmax = $(1+0.11S_{DS})$*DL + 0.75*$(1+0.14S_{DS})$*PL
= **874 lb**
Mcol(max) = 0.75*0.7*Mcol(seismic)
= **12,451 IN-LB**
KxLx/rx = 2.1 x 96 IN/1.13 IN
= 178.4 <=== (Kl/r)max
KyLy/ry = 1 x 6 IN/0.83 IN
= 7.2
ro= (rx^2 + ry^2 + xo^2)^0.5
= 1.402 in
ß= 1-(xo/ro)^2
= 1.000





Fe is taken as the smaller of Fe1 and Fe2:
Fe1= π^2E/(KL/r)max^2
= 9.15 KSI
σex= π^2E/(KxLx/rx)^2
= 9.15 KSI
σt= 1/Aro^2[GJ+(π^2ECw)/(KtLt)^2]
= 57.26 KSI
Fe2= 1/(2ß) x {(σex+σt)-[(σex+σt)^2-(4 x ß x σex x σt)]^0.5}
= 9.15 KSI
Fe= 9.15 KSI
Fy/2= 18.00 KSI
Since, Fe < Fy/2
Then, Fn= Fe
= 9.15 KSI
Pn= Aeff x Fn
= 8,873 LB
Ωc= 1.8
Pa= Pn/Ωc
= 4,929 LB
P/Pa= 0.18 > 0.15
Thus, check: P/Pa + (Cmx*Mx)/(Max*µx) ≤ 1.0
P/Pao + Mx/Max ≤ 1.0
Pno= Ae x Fy
= 34,920 LB
Pao= Pno/Ωc
= 19,400 LB
Me= Cb x ro x Aeff x (σey x σt)^0.5
= 768 IN-KIP
My= Sx * Fy
= 31,680 IN-LB
Mc= My[1-My/(4Me)]
= 31,353 IN-LB
Max=Maxo= Mc/Ωf
= 18,774 IN-LB
µx= {1/[1-(Ωc*P/Pcr)]}^-1
= 0.82

SECTION PROPERTIES

A =	2.000 IN
B =	2.563 IN
C =	0.563 IN
t1 =	0.045 IN
t2 =	0.170 IN
Aeff =	0.970 IN^2
Ix =	1.230 IN^4
Sx =	0.880 IN^3
rx =	1.130 IN
Iy =	0.670 IN^4
Sy =	0.540 IN^3
ry =	0.830 IN
J =	0.007 IN^4
Cw =	0.004 IN^6
xo =	0.000 IN
Kx =	2.1
Lx =	96.00 IN
Ky =	1.0
Ly =	6.00 IN
Kt =	1.0
Lt =	6.00 IN
Fy=	36 KSI
G=	11,300
E=	29,500 KSI
Cmx=	0.85
Cb=	1.0
Ωf=	1.67

Thus, (874 lb/4929 lb) + (0.85 x 12451 in-lb)/(18774 in-lb x 0.82) = ***0.86*** ***< 1.0, OK***
(874 lb/19400 lb) + (12451 in-lb/18774 in-lb) = ***0.71*** ***< 1.0, OK***

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OVERTURNING ANALYSIS: TYPE N (144"Hx48"Wx16"D/24"D)

$(0.6\text{-}0.11S_{DS})$*DL + 0.75*$(0.6\text{-}0.14S_{DS})$*PL - 0.75*0.7*EL

Full Loaded Condition:

Allowable Tension = 830 lb
Allowable Shear = 900 lb
\# of Anchors / Plate = 2
Depth = 40.00 in

Movt = 0.75*0.7*(Fi x hi)
= 12,451 in-lb

Mst = [$(0.6\text{-}0.11S_{DS})$*DL + 0.75*$(0.6\text{-}0.14S_{DS})$*PL] x Depth / 2
= 7,151 in-lb

OCCURRENCE OF ANCHORS = 2

Puplift = (Movt - Mst) / Depth
= (12451 in-lb - 7151 in-lb) / 40 in
= 132 lb

Vcol = 0.75*0.7*E
= 124 lb

Interaction Eqn. **[265 lb/ 1660 lb] + [248 lb/ 3600 lb] =** **0.23** **< 1.2 Therefore, Ok**

USE 3/8"Ø x 2-3/8" MIN. EMBED. HILTI KB1 (IAPMO #UES ER-678)
(2) ANCHORS PER BASE PLATE AT FRONT ONLY, EVERY OTHER BAY.
(1) ANCHOR PER BASE PLATE AT END.

LIGHT DUTY STORAGE FIXTURE ANALYSIS

SEIZMIC INC.
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TEL: (909) 869-0989
1130 E. CYPRESS STREET, COVINA, CA 91724

PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	14
CALCULATED BY	KE
DATE	1/28/2026

LOADS & DISTRIBUTION: TYPE I

SEISMIC SHEAR BASED ON SECTION 1613 OF THE 2021 IBC.

SITE CLASS = D

S_{MS} = 1.51 WHERE Sms = Fa*Ss AND Fa = 1.2, Ss = 1.26

S_{DS} = 1.01

I = 1.00 <=== IN STOCKROOM AREA, NO PUBLIC ACCESS

R = 4

WORKING STRESS REDUCTION = 0.7

PRODUCT LOAD, wPL = 70 lb/LEVEL

DEAD LOAD, wDL = 10 lb/LEVEL

OF LVLS= 10 <=== NOTE: NO LOAD ON TOP LEVEL

FULLY LOADED

E = [2/3*1.26*1.2*(0.67*630 lb+100 lb)/4]

= 132 lb

LATERAL FORCE DISTRIBUTION

LEVEL	wx	hx	wxhx	Fi	Movt
1	80 lb	14.0 in	1,120 in-lb	3 lb	40 in-lb
2	80 lb	28.0 in	2,240 in-lb	6 lb	159 in-lb
3	80 lb	42.0 in	3,360 in-lb	9 lb	358 in-lb
4	80 lb	56.0 in	4,480 in-lb	11 lb	637 in-lb
5	80 lb	70.0 in	5,600 in-lb	14 lb	995 in-lb
6	80 lb	84.0 in	6,720 in-lb	17 lb	1,433 in-lb
7	80 lb	98.0 in	7,840 in-lb	20 lb	1,950 in-lb
8	80 lb	112.0 in	8,960 in-lb	23 lb	2,547 in-lb
9	80 lb	126.0 in	10,080 in-lb	26 lb	3,223 in-lb
10	10 lb	144.0 in	1,440 in-lb	4 lb	605 in-lb
		∑ =	51,840 in-lb	132 lb	11,947 in-lb

TOP LOADED ONLY

E_{top} = [2/3*1.26*1.2*(70 lb+100 lb)/4]

= 43 lb

LATERAL FORCE DISTRIBUTION

LEVEL	wx	hx	wxhx	Fi	Movt
1	10 lb	14.0 in	140 in-lb	0 lb	5 in-lb
2	10 lb	28.0 in	280 in-lb	1 lb	20 in-lb
3	10 lb	42.0 in	420 in-lb	1 lb	46 in-lb
4	10 lb	56.0 in	560 in-lb	1 lb	81 in-lb
5	10 lb	70.0 in	700 in-lb	2 lb	127 in-lb
6	10 lb	84.0 in	840 in-lb	2 lb	183 in-lb
7	10 lb	98.0 in	980 in-lb	3 lb	248 in-lb
8	10 lb	112.0 in	1,120 in-lb	3 lb	325 in-lb
9	80 lb	126.0 in	10,080 in-lb	26 lb	3,286 in-lb
10	10 lb	144.0 in	1,440 in-lb	4 lb	617 in-lb
		∑ =	16,560 in-lb	43 lb	4,937 in-lb

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INC.
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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	15
CALCULATED BY	KE
DATE	1/28/2026

TRANSVERSE SEISMIC LOADS: TYPE I

THE FRAME SHALL BE ANALYZED AS A BRACED FRAME IN THE LONGITUDINAL DIRECTION, AND A MOMENT RESISTING FRAME IN THE TRANSVERSE DIRECTION, UTILIZING MOMENT RESISTANT SPREADER TO COLUMN CONNECTIONS.

V = 132 lb

Vcol= 66 lb

TRANSVERSE SEISMIC

$M_{1\text{-}1}$= Vcol * h1
= 65.8 lb * 5.25 in
= 345 in-lb

$M_{2\text{-}2}$= [Vcol-(F1+F2)/2] * h2/2
= [65.8 lb - (2.84 lb + 5.69 lb)/2]*30 in/2
= 923 in-lb

$M_{3\text{-}3}$= [Vcol-(F1+F2+F3+F4)/2] * h3/2
= 774 in-lb

CONNECTION MOMENT

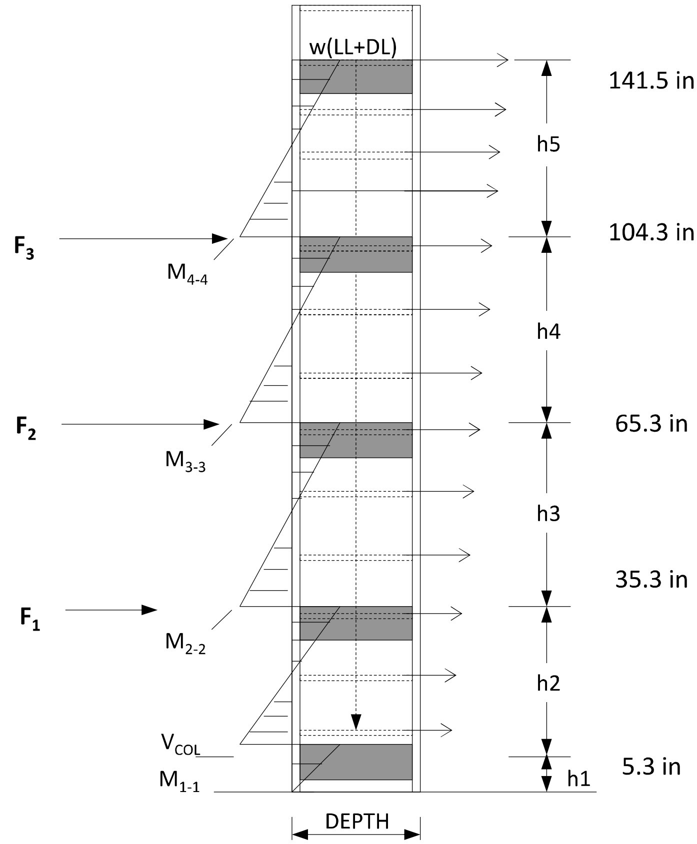
Mconn$_{1\text{-}1}$= [($M_{1\text{-}1}$)+($M_{2\text{-}2}$)]
= 1,268 in-lb

Mconn$_{2\text{-}2}$= [($M_{2\text{-}2}$)+($M_{3\text{-}3}$)]
= 1,696 in-lb

Mconn.max.= 1,696 in-lb

VERIFY ADEQUACY OF COLUMN

Mcol.max.= 923 in-lb



TRANSVERSE ELEVATION

h1= 5.3 in
h2= 30.0 in
h3= 30.0 in
h4= 39.0 in
h5= 37.3 in
depth= 12.0 in

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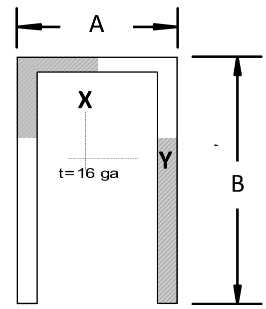
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	16
CALCULATED BY	KE
DATE	1/28/2026

COLUMN ANALYSIS: TYPE I
TRANSVERSE DIRECTION



Pmax = $(1+0.11S_{DS})DL + 0.75*(1+0.14S_{DS})PL + 0.75*0.7*Movt/D$
= 848 lb

Mmax = 0.75*0.7*Mcol.max
= 484 in-lb

KxLx/rx = 43.3
KyLy/ry = 35.1
→ 43.3 (GOVERNS)

PROPERTIES

A = 0.75 in
B = 1.66 in
t = 0.06 in
AREA = 0.217 in^2
Sx = 0.06 in^3
rx = 0.323 in
Sy = 0.06 in^3
ry = 0.479 in
Fy = 36,000 psi
Kx = 1.0
Ky = 1.2
Cm = 1.00

Cc = (2π^2E/Fy)^0.5
= 126.1

SINCE Kl/r < Cc USE Fa = .522Fy-[(Kl/r*Fy)/1494]^2

Fa = .522Fy-[(Kl/r*Fy)/1494]^2
= 17,701 psi

fa = Pmax/A
= 3,907 psi

fb = Mmax/Sy
= 8,353 psi

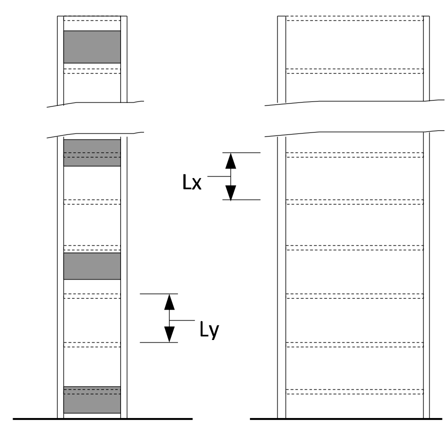
F'e = (12π^2E)/((23(KL/r)^2)
= 79,488 PSI

Fb = 0.6*Fy
= 21,600 psi

fa/Fa = 0.22 > 0.15

(1-fa/F'e)= 0.951

fa/Fa + Cm*fb/(Fb*(1-fa/F'e))= **0.63** **< 1.0 OK**



SIDE VIEW **FRONT VIEW**

Lx = 14.0 in
Ly = 14.0 in

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	17
CALCULATED BY	KE
DATE	1/28/2026

TRANSVERSE MOMENT CONNECTION ANALYSIS: TYPE I

Mconn= 0.75*0.7*Mconn.max
= 891 in-lb

(A) SHEAR CAPACITY OF CONNECTOR SCREWS:

Fu= 58,000 PSI

SHEAR IS TAKEN AS LEAST OF Pns:

Pns= 4.2*(t2^3*d)^0.5*Fu2
= 4.2 * [(0.0598 in)^3 * 0.164 in]^0.5 * 58000 psi
= 1,443 lb

t1=t-top plate= 0.060 in
t2=bottom plate= 0.060 in
Fu1=Fu of top plate= 58,000 PSI
Fu2=Fu of bottom plate= 58,000 PSI
d=screw diam= 0.164 in
OK TO USE E4 FORMULA
t2/t1= 1
Ω = 3

Pns= 2.7*t1*d*Fu1
= 2.7 * 0.0598 in * 0.164 in * 58000 psi
= 1,536 lb

Pns= 2.7*t2*d*Fu2
= 2.7 * 0.0598 in * 0.164 in * 58000 psi
= 1,536 lb

Pns-eff= 1,443 lb

Pns-eff/Ω= 1443 lb / Ω
= 481 lb

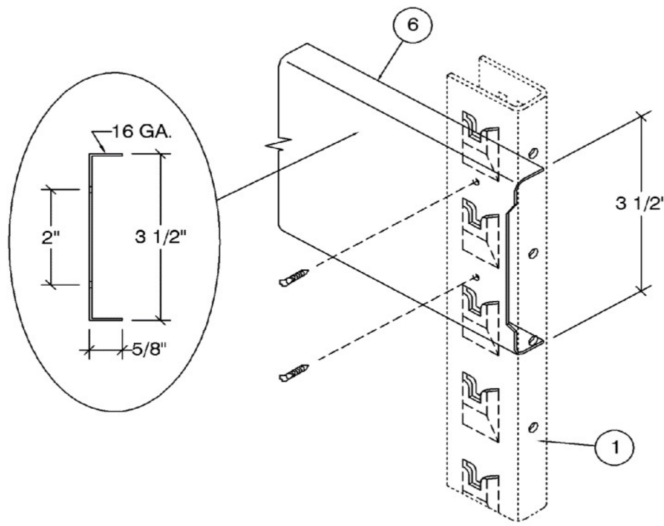
(B) MOMENT CAPACITY OF BATTEN:

SxHOR. BRACE = .22 in^3

fb = Mmax/Sx
= 891 in-lb/0.22 in^3
= 4,048 psi

Fb = 0.6*Fy
= 0.6 * 36000 psi
= 21,600 psi

fb/Fb = 4048 psi/21600 psi
= **0.19** **< 1.0 O.K.**



(C) MOMENT CAPACITY OF CONNECTION:

Mconn. cap.= Pns-eff/Ω * 2 in
= (481 lb)*(2.0 in)
= **962 in-lb** **> 891 in-lb O.K.**

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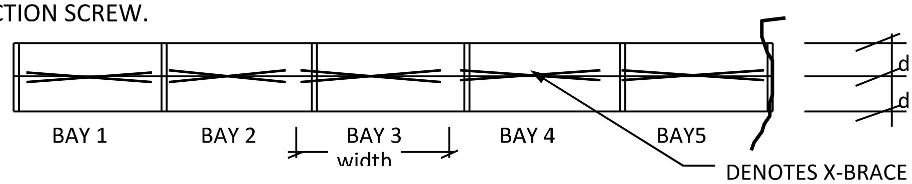
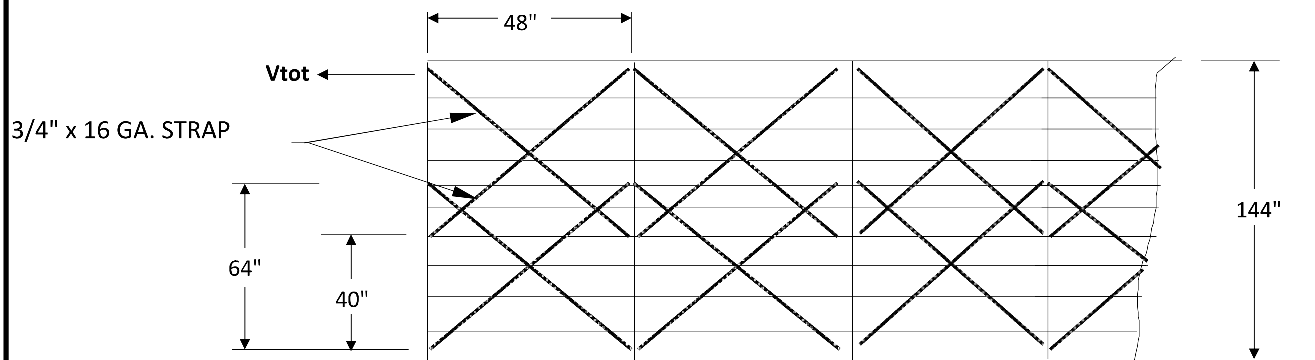
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	18
CALCULATED BY	KE
DATE	1/28/2026

BRACING ANALYSIS: TYPE I

BRACING CAPACITY IS GOVERNED BY TENSION CAPACITY OF CROSS BRACING AND SHEAR CAPACITY OF CONNECTION SCREW.

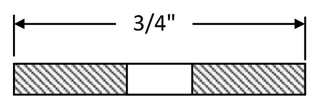


PLAN VIEW



GENERAL CONFIGURATION: LONGITUDINAL BRACE

Vlongit(eff)= 0.75*0.7*Vtot *2 BAYS/1 X-BRACES <=== 2 BAYS TRIB. TO X-BRACES.
= **138 lb**
Vdiag= Vlongit(eff)*(Ldiag/Lhoriz)
= 138 lb * (80 in/48 in)
= **230 lb** <=== SEISMIC LOAD IN TENSION



"X" BRACE (3/4" x 16 ga STRAP)

SHEAR CAPACITY OF #8x3/4" SHEET METAL SCREW
PER AISI COLD FORMED MANUAL.
Ω= 3

SHEAR IS TAKEN AS LEAST OF Pns:

Pns= 4.2*(t2^3*d)^0.5*Fu2
= 4.2 * [(0.06 in)^3 * 0.164 in]^0.5 * 58000 psi
= 1450 lb

Pns= 2.7*t1*d*Fu1
= 2.7 * 0.06 in * 0.164 in * 58000 psi
= 1541 lb

Pns= 2.7*t2*d*Fu2
= 2.7 * 0.06 in * 0.164 in * 58000 psi
= 1541 lb

Pns-eff/Ω= (1450 lb/3)
= **483 lb** **> Vdiag OK IN SHEAR**

t (GAUGE) = 16 GA.
AREA(gross)= 0.045 in^2
AREA(net)= 0.036 in^2
Lhoriz= 48.00 in
Lvert= 64.00 in
Ldiag= 80.00 in
Fy= 36,000 psi
Fu= 58,000 psi
Fv= 10,000 psi
SCREW DIAM.= 0.164 IN
SCREW TYPE = #8x3/4" SHEET METAL SCREW
t1=t-top plate= 0.06 in
t2=bottom plate= 0.06 in
Fu1=Fu of top plate= 58,000 psi
Fu2=Fu of bottom plate= 58,000 psi
d=screw diam= 0.164 in

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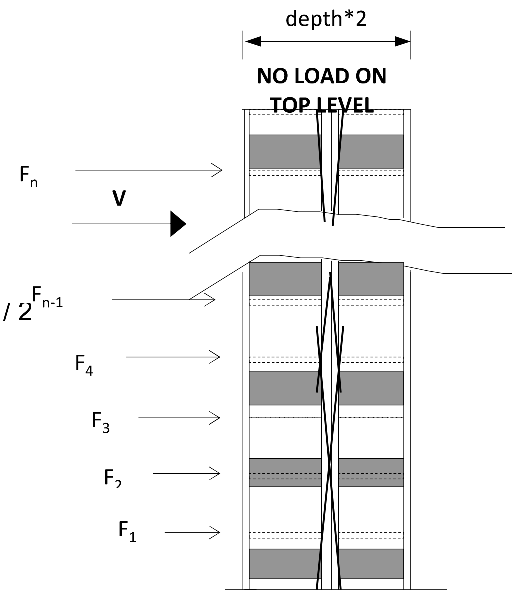
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	19
CALCULATED BY	KE
DATE	1/28/2026

OVERTURNING ANALYSIS: TYPE I

$(0.6-0.11S_{DS})DL + 0.75*(0.6-0.14S_{DS})PL - 0.75*0.7EL$



TRANSVERSE ELEVATION

Fully Loaded Condition:

Movt = 0.75*0.7*∑(Fi * Hi)*2
= 12,544 in-lb

Mst = $[(0.6-0.11S_{DS})*DL + 0.75*(0.6-0.14S_{DS})*PL]$ x Depth*2 / 2
= 6,378 in-lb

Puplift = [Movt - Mst] / Depth
= 257 lb <= UPLIFT

Vcol = 0.75*0.7*E*2/2
= 69 lb

Interaction Eqn.

(2*257 lb/790 lb) + (2*69 lb/900 lb) = 0.80 < 1.2 OK

Top Shelf Loaded Only:

TOTAL DEPTH =	24.0 in
DEPTH OF UNIT (d) =	12.0 in
TOP SHELF HT =	126.00 in
# LEVELS=	10

Movt = 0.75*0.7*∑(Fi * Hi)*2
= 5,184 in-lb

Mst = $[(0.6-0.11S_{DS})*DL + 0.75*(0.6-0.14S_{DS})*PL]$ x Depth*2 / 2
= 1,752 in-lb

Puplift = [Movt - Mst] / Depth
= 143 lb <= UPLIFT

Vcol = 0.75*0.7*E_{top}*2/2
= 22 lb

ANCHOR

QUANTITY =	1
PULLOUT =	790 lb
SHEAR =	900 lb
OCCURRENCE =	2

Interaction Eqn.

(2*143 LB/790 lb) + (2*22 lb/900 lb) = 0.41 < 1.2 OK

FOR TYPE I UNITS:

(1) 3/8"Ø x 2-3/8" NOM. EMBED. HILTI KB1 (IAPMO #UES ER-678)
PER BASE PLATE; ANCHOR PERIMETER COLUMNS, STAGGERED AND AT ENDS.

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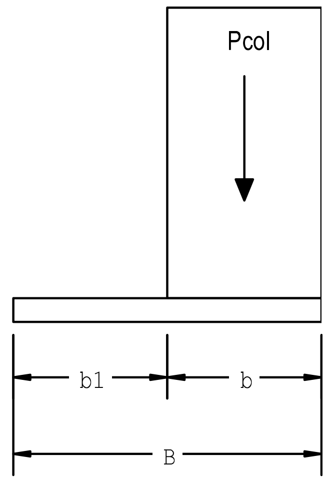
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	20
CALCULATED BY	KE
DATE	1/28/2026

BASE PLATE ANALYSIS: TYPE I

Pcol.max. = 848 lb <== SEE COLUMN ANALYSIS



BASE PLATE WIDTH (D)= 2.25 in

EFF.BASE PLATE DEPTH (B)= 1.75 in

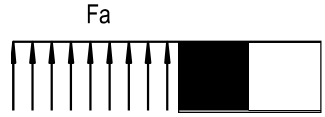
b = 1.50 in

b1 = 0.25 in

THICKNESS (t) = 0.06 in

Fy = 36,000 psi

fa =P/A = Pcol/((D)(B))= 215 psi



Mbase= (W/in)(L^2)/2
= (fa*1"STRIP)(b1^2)/2
= 6.7 in-lb

Sbase= (1)(t^2)/6
= 0.0006 in^3

Fbase = (0.75)(Fy)
= 27,000 psi

fb/Fb = Mbase/((Sbase)(Fbase))
= **0.42** **OK**

ACTUAL BASE PLATE SIZE IS 2.25 in x2 in x0.0598 in, ONLY AN EFFECTIVE BASE PLATE SIZE OF 2.25 in x1.75 in x0.0598 in IS USED IN THE ANALYSIS.

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LOAD COMBINATIONS FOR SLAB ANALYSIS

		Resultant Load combination
1	1.4DL + 1.2PL	1.4DL + 1.2PL
2	1.2DL + 1.4PL + 1.6LL + 0.5(SL or RL)	1.2DL + 1.4PL
3	1.2DL + 0.85PL + 0.5LL + 1.6(SL or RL)	1.2DL + 0.85PL
4	1.2DL + 0.85PL + 0.5LL + 1.3WL + 0.5(SL or RL)	1.2DL + 0.85PL
5	$(1.2+0.2S_{DS})DL + (0.85+0.2S_{DS})PL + EL$	
	$_{DS})DL + (0.9-0.2S_{DS})PLapp - EL$	$_{DS})DL + (0.9-0.2S_{DS})PLapp - EL$

DL-total/col = 50 lb
PL-total/col = 345 lb

EL = Movt/Depth
= 996 lb

Load combination 1

Pmax = 1.4DL + 1.2PL
= 1.4 x 50 lb + 1.2 x 345 lb
= 484 lb

Load combination 2

Pmax = 1.2DL + 1.4PL
= 1.2 x 50 lb + 1.4 x 345 lb
= 543 lb

Load combination 3 & 4

Pmax = 1.2DL + 0.85PL
= 1.2 x 50 lb + 0.85 x 345 lb
= 353 lb

Load combination 5

Pmax = $(1.2+0.2S_{DS})DL + (0.85+0.2S_{DS})PL + EL$
= 1.4 x 50 lb + 1.05 x 345 lb + 996 lb
= 1,428 lb

$_{DS})DL + (0.9-0.2S_{DS})PLapp - EL$

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SLAB & SOIL ANALYSIS: TYPE I

A) PUNCTURE

Pmax = 1428 lb

Fpunct = 2.66*sqrt(f'c)*Ø
= 120 psi

Apunct = (B+D)*2*t
= 32.0 in^2

fv/Fv = Pmax/(Apunct * Fpunct)
= **0.37**

SINCE NO PUNCTURE OCCURS, THE SLAB WILL DISTRIBUTE THE LOAD OVER A LARGER AREA OF SOIL & ACT AS A FOOTING.

B) SLAB TENSION

Asoil = Pmax * 144/fsoil
= 205.7 in^2

L = (Asoil)^.5
= 14.3 in

B = sqrt(B*D) + t
= 6.0 in

l= (L-B)/2
= 4.2 in

Mconc = wl^2/8 = (fsoil*l^2) / (144*2)
= 61 in-lb

Sconc = 1 in * t^2 / 6
= 2.67 in^3

fconc = 5Ø* sqrt(f'c)
= 138 psi

fb/Fb = Mconc/(Sconc*fconc)
= **0.17**

BASE PLATE:

BASE PLATE DEPTH (B)= 1.75 in
BASE PLATE WIDTH (D)= 2.25 in

SLAB:

t = 4.0 in
(minimum required)
fc' = 2,500 psi
(minimum required)

Ø= 0.55

SOIL:

fs = 1,000 psf

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	23
CALCULATED BY	KE
DATE	1/28/2026

LOADS & DISTRIBUTION: TYPE H

ANALYSIS BASED ON SECTION 1613 OF THE 2021 IBC.

SITE CLASS = D
Sms = 1.51 WHERE Sms = Fa*Ss AND Fa = 1.2, Ss = 1.26
Ip = 1.00 S_{DS} = 1.01
Rp = 2.5
ap = 1.0

Fp = [0.4 * ap * (2/3*Fa*Ss) *Ip/(Rp)] * Wp = 0.161 Wp

EXCEPT WHERE:

Fp shall not be taken as less than 0.3*(2/3*Fa*Ss)*Ip*Wp = 0.302 Wp

AND

not required to be taken as greater than 1.6*(2/3*Fa*Ss)*Ip*Wp = 1.613 Wp

Fp (GOVERNS) = 0.302 Wp

OF LEVELS = 10
LIVE LOAD = 70 lb
DEAD LOAD = 10 lb

NOTE: NO LOAD ON TOP LEVEL

LONGITUDINAL DIRECTION & TRANSVERSE DIRECTION

Fp = 0.3*SDS*Ip*Wp
= [0.3*((2/3)*1.2*1.26)*1] * (0.67*70 lb + 10 lb)
= 17.2 lb

Vtotal = 158 lb

LATERAL FORCE DISTRIBUTION

h	LEVEL	WEIGHT	HEIGHT	Fp	Movt
14.0 in	1	80 lb	14.0 in	17 lb	241 in-lb
14.0 in	2	80 lb	28.0 in	17 lb	482 in-lb
14.0 in	3	80 lb	42.0 in	17 lb	723 in-lb
14.0 in	4	80 lb	56.0 in	17 lb	964 in-lb
14.0 in	5	80 lb	70.0 in	17 lb	1204 in-lb
14.0 in	6	80 lb	84.0 in	17 lb	1445 in-lb
14.0 in	7	80 lb	98.0 in	17 lb	1686 in-lb
14.0 in	8	80 lb	112.0 in	17 lb	1927 in-lb
14.0 in	9	80 lb	126.0 in	17 lb	2168 in-lb
18.0 in	10	10 lb	144.0 in	3 lb	435 in-lb
			∑ WH =	158 lb	11276 in-lb

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	24
CALCULATED BY	KE
DATE	1/28/2026

WALL ADEQUACY ANLAYSIS: TYPE H

WALL LATERAL LOAD = 5. psf

WALL AREA/UNIT = (144 in) * (48 in)
= 6912. in^2
= 48. ft^2

NOTE: IT IS UNDERSTOOD THAT THE DESIGN OF THE WALL IS UNDER THE BUILDING PERMIT. CALCULATIONS WILL SHOW THAT PER SECTION 1607.14 INTERIOR/PARTITION WALLS>THAN 6 FT IN HEIGHT MUST BE DESIGNED FOR A MINIMUM OF 5 PSF LATERAL LOADING AND SINCE THE UNITS ARE SUPPORTED AT THE TOP AND BOTTOM, HALF OF THE SEISMIC SHEAR IS TRANSMITTED TO THE TOP AND THE BALANCE TO THE BOTTOM.

Vwall = Vtotal / WALL AREA
= 157.9/48
= 3.3 psf < 5 psf OK; THUS, WALL IS ADEQUATE TO RESIST SEISMIC LOADS.

CONNECTION TO WALL ANLAYSIS:

WITHDRAWAL FORCE ON WALL CONNECTION = T
CONNECTION HEIGHT = 90 in

Mot = 0.75*0.7*∑(Fi * Hi)
= 5,920 in-lb

Mst = [(0.6-0.11S_{DS})DL + (0.6-0.14S_{DS})PL] x Depth / 2
= 2,028 in-lb

T = (Mot - Mst)/(HEIGHT OF CONNECTION)
= [Mot - Mst] / Height
= 43 lb <= WITHDRAWAL

CHECK TENSION CAPACITY OF #12 TEK SCREW PER CHAPTER E4 OF THE 1996 AISI COLD FORMED MANUAL.

Tpullout= 0.85*tc*d*Fu2/Ω <=== sec E4.4.1-1
= (0.85 * 0.036 in * 0.216 in * 65000 psi/3)
= 143 lb

Ω = 3
\# SCREWS/CONN = 2
SCREW TYPE = #12 TEK SCREW
P(allow) = 143 lb
Pallow.CONN = 286 lb <== (2) screws per connection

SINCE 43 lb < 286 lb THEN WALL ATTACHMENT IS SUFFICIENT TO RESIST SEISMIC FORCES.

WIDE SPAN FIXTURE ANALYSIS

Wide Span Fixtures

SCOPE:

The purpose of this analysis is to show that the following light duty storage fixtures complies with the specifications set forth in Section 1613 of the 2021 IBC and Section 15.5.3 of ASCE 7-16.

The storage racks are prefabricated and are to be field installed only without any field welding.

PARAMETERS:

The fixtures will be analyzed as a braced steel storage rack system.
The system will be analyzed for seismic loading utilizing the following equation:

$V = Cs \times W$
$Cs = S_{DS} \times I / R$
$S_{DS} = 2/3 \times Fa \times Ss$
$I = 1$
$R = 4$
$\rho = 1$
$\beta = 1$

Fa = **1.2**
Ss = **1.26**

SPECIFICATIONS:

Steel - Fy = 50,000 psi

Bolts - ASTM A307 unless otherwise noted

Anchors - 3/8"Ø x 2-3/8" NOM. EMBED. HILTI KB1 (IAPMO #UES ER-678)

Slab - 4" x 2,500 psi

Soil - 1000 psf

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	27
CALCULATED BY	KE
DATE	1/28/2026

Loads & Distribution

Fa = 1.20
Ss = 1.26
S_{DS} = 2/3*Fa*Ss
= 1.01
I = 1 <=== IN AREA NOT OPEN TO THE PUBLIC
R = 4
Cs = S_{DS} * I / R
= 0.253

Depth = 48.0 IN

# of Levels= 3
Shelf PL = 350 lb/Level
Shelf DL = 50 lb/Level

Seismic Shear:

Vtotal = Cs*Ws
= 0.253 x 1200 LB
= 216 LB

Level	Ws	hx	wx hx	Fi	Movt
1	400.0 LB	6.0 IN	2,400 IN-LB	7.0 LB	42 IN-LB
2	400.0 LB	60.0 IN	24,000 IN-LB	69.7 LB	4,179 IN-LB
3	400.0 LB	120.0 IN	48,000 IN-LB	139.3 LB	16,718 IN-LB
	1,200.0 LB		74,400 IN-LB	216 LB	20,939 IN-LB

Vtop = Cs*Ws,top
= 0.253 x 500 LB
= 127 LB

Level	Ws	hx	wx hx	Fi	Movt
1	50 LB	6.0 IN	300 IN-LB	0.7 LB	4 IN-LB
2	50 LB	60.0 IN	3,000 IN-LB	7.4 LB	444 IN-LB
3	400 LB	120.0 IN	48,000 IN-LB	118.4 LB	14,204 IN-LB
	500 LB		51,300 IN-LB	127 LB	14,652 IN-LB

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Longitudinal & Transverse Analysis:

Longitudinal Column Forces:

Mbase = 0 IN-LB <== Based Assumed to be pinned

Level	Pcol-static	Pcol-seismic	Pcol-total	Mcol	Mconn-seismic	
1	600 LB	0 LB	600 LB	648 IN-LB	1,734 IN-LB	
2	400 LB	0 LB	400 LB	2,821 IN-LB	2,455 IN-LB	
3	200 LB	0 LB	200 LB	2,090 IN-LB	1,045 IN-LB	

Transverse Loads:

Level	Pcol-static	Pcol-seismic				
1	600 LB	436 LB				
2	400 LB	435 LB				
3	200 LB	348 LB				

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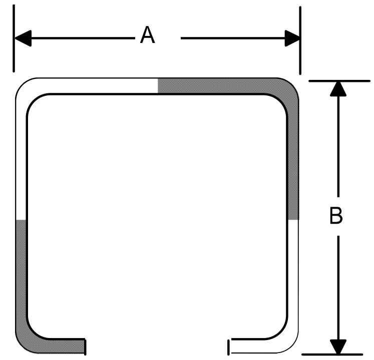
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	29
CALCULATED BY	KE
DATE	1/28/2026

Longitudinal Column Analysis:

Analyzed per AISI. Section properties are based on net effective sections.

P= (1+0.11S_{DS})DL+(1+0.14S_{DS})PL
= 803 LB
M= 0.7*Mcol
= 1,975 IN-LB
KxLx/rx = 1.7*54 IN/0.75 IN
= 122.4 <=== (Kl/r)max
KyLy/ry = 1*54 IN/0.622 IN
= 86.8
ro= (rx^2 + ry^2 + xo^2)^0.5
= 1.808 IN
ß= 1-(xo/ro)^2
= 0.290



Fe IS TAKEN AS THE SMALLER OF Fe1 AND Fe2:
Fe1= π^2E/(KL/r)max^2
= 19.4 KSI
σex= π^2E/(KxLx/rx)^2
= 19.4 KSI
σt= 1/Aro^2[GJ+(π^2ECw)/(KtLt)^2]
= 20.91 KSI
Fe2= 1/(2ß)*{(σex+σt)-[(σex+σt)^2-(4*ß*σex*σt)]^0.5}
= 10.9 KSI
Fe= 10.9 KSI
Fy/2= 25.0 KSI
SINCE, Fe < Fy/2
THEN, Fn= Fe
= 10.9 KSI
Pn= Aeff*Fn
= 3,389 LB
Ωc= 1.92

Pa= Pn/Ωc
= 1,765 LB

P/Pa= 0.45 > 0.15

THUS, CHECK: P/Pa + (Cmx*Mx)/(Max*µx) ≤ 1.0
Pno= Ae*Fy
= 15,500 LB
Pao= Pno/Ωc
= 8,073 LB
Me= Cb*ro*Aeff*(σey*σt)^0.5
= 16 IN-K
My= Sx*Fy
= 9,700 IN-LB
Mc= My[1-My/(4Me)]
= 8,223 IN-LB
Max=Maxo= Mc/Ωf
= 4,924 IN-LB
µx= {1/[1-(Ωc*P/Pcr)]}^-1
= 0.75

(803 LB/1765 LB) + (0.85*1975 IN-LB)/(4924 IN-LB*0.75) = **0.91** **< 1.0, OK**
(803 LB/8073 LB) + (1975 IN-LB/4924 IN-LB) = **0.50** **< 1.0, OK**

SECTION PROPERTIES

A =	1.813 IN
B =	1.625 IN
t =	0.075 IN
Aeff =	0.310 IN^2
Ix =	0.175 IN^4
Sx =	0.194 IN^3
rx =	0.750 IN
Iy =	0.120 IN^4
Sy =	0.141 IN^3
ry =	0.622 IN
J =	0.001 IN^4
Cw =	0.138 IN^6
xo =	-1.523 IN
Kx =	1.7
Lx =	54.00 IN
Ky =	1.00
Ly =	54.00 IN
Kt =	1.00
Lt =	54.00 IN
Fy =	50 KSI
G =	11,300
E =	29,500 KSI
Cmx =	0.85
Cb =	1.0
Ωf =	1.67

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INC.
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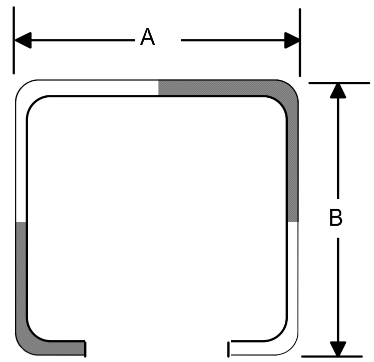
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	30
CALCULATED BY	KE
DATE	1/28/2026

Transverse Column Analysis:

Analyzed per AISI. Section properties are based on net effective sections.

Pcol= (1+0.11S_{DS})DL+(1+0.14S_{DS})PL+0.7*EL
= **1,108 LB**
KxLx/rx = 1.7*54 IN/0.75 IN
= 122.4
KyLy/ry = 1*54 IN/0.622 IN
= 86.8
(KL/r)max = 122.4



ro= (rx^2 + ry^2 + xo^2)^0.5
= 1.808 IN

ß= 1-(xo/ro)^2
= 0.290

Fe IS TAKEN AS THE SMALLER OF Fe1 AND Fe2:

Fe1= π^2E/(KL/r)max^2
= 19.4 KSI

σex= π^2E/(KxLx/rx)^2
= 19.4 KSI
σt= 1/Aro^2[GJ+(π^2ECw)/(KtLt)^2]
= 20.91 KSI
Fe2= 1/(2ß)*{(σex+σt)-[(σex+σt)^2-(4*ß*σex*σt)]^0.5}
= 10.9 KSI

Fe= 10.9 KSI
Fy/2= 25.0 KSI
SINCE, Fe < Fy/2
THEN, Fn= Fe
= 10.9 KSI

Pn= Aeff*Fn
= 3,389 LB

Ωc= 1.92

Pa= Pn/Ωc
= 1,765 LB

Pcol/Pa= **0.63** **< 1.0, OK**

SECTION PROPERTIES

A =	1.813 IN
B =	1.625 IN
t =	0.075 IN
Aeff =	0.310 IN^2
Ix =	0.175 IN^4
Sx =	0.194 IN^3
rx =	0.750 IN
Iy =	0.120 IN^4
Sy =	0.141 IN^3
ry =	0.622 IN
J =	0.001 IN^4
Cw =	0.138 IN^6
xo =	-1.523 IN
Kx =	1.7
Lx =	54.00 IN
Ky =	1.00
Ly =	54.00 IN
Kt =	1.00
Lt =	54.00 IN
Fy =	50 KSI
G =	11,300
E =	29,500 KSI

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	31
CALCULATED BY	KE
DATE	1/28/2026

Beam Analysis

Beam to column connections provide adequate moment capacity to stabalize the system, although it does not provide full fixity. Thus, the beams shall be analyzed assuming partial end fixity. In justifying the beam to column moment connection, the partial end fixity moment will be added to the Longitudinal frame moment for the analysis of the connection.

% END FIXITY = 10 %

Effective Moment for Partially Fixed Beam

For simply supported beams, the max imum moment at the center is given by wL^2/8. An assumption of partial fixity will decrease the maximum moment by the following method.

Ø = 0.1

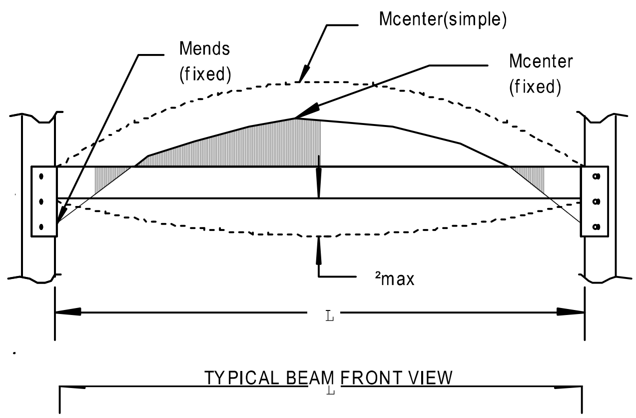
Mcenter = Mcenter(simple ends) - Ø*Mcenter(fixed ends)
= wL^2/8 - (0.1 * wL^2/12)
= wL^2/8 - wL^2/120
= 0.117 * wL^2

Reduction
COEFF ß = 0.117/0.125
= 0.936

THUS,

Mcenter = ß*(wL^2/8)
= **0.936*(wL^/8)**

Mends = Ø*Mmax(fixed ends)
= (wL^2/12)*0.1
= **0.0083*wL^2**



TYPICAL BEAM FRONT VIEW

Efective Deflection for Partially Fixed Beam

For simply supported beam conditions, the maximum deflection at the center is given by 5wL^4/384EI. An assumption of partial fixity will decrease this maximum deflection by the following method:

Dmax=	**ß * [5w(L)^4/(384*E*Ix)]**

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	32
CALCULATED BY	KE
DATE	1/28/2026

Beam Analysis cont.

Live Load = 350 LB
Dead Load = 50 LB

End fixity = 10 %

Mcenter = 0.117 * wL^2
= 2,246 IN-LB
Mends = 0.0083*wL^2
= 159 IN-LB

Lmax = [1950+1200(M1/M2)]b/Fy
M1/M2 = 0.070940171 <=== SINCE Mcenter > Mends
Lmax = (1950 + 85.1)*1/50
= 40.7 IN < L
Thus, Fb = 0.6 * Fy
= 30,000 PSI

Maximum Static Load per Level depends on:

1) Moment Capacity

If, fb = M/Sx
Then, 0.6 * Fy = (ß*wL^2/8)/Sx
Thus,
Max Wt./Level = [9.6*Fy*Sx/(ß * L)]*a
= (9.6* 50 KSI * 0.49 IN^3)/(0.936*96 IN)*0.875
= 2,327 LB/LVL **<=== GOVERNS**
OR,
2) Allowable Deflection

If, Dallow = L/180
= 0.533 IN
And, D = [5w(L)^4/(384*E*Ix)]*ß
Then,
Max Wt./Level = 2*[(384*E*Ix*Δallow)/(5*L^3 * ß)]
= 3,758 LB/LVL

Beam Properties
Ix = 1.310 IN^4
Sx = 0.490 IN^3
Fy = 50,000 PSI
Length=L = 96.0 IN
t = 0.060 IN
b = 1.000 IN

a = Impact Coefficient = 0.875
ß = 0.936

Thus, *Maximum* Allowable Live Load / Level = **2,277 LB/LVL**

LEVEL	Mstatic	Mimpact	Mallow(static)	Mseismic	Mseismic(allow)	Result
1	2,246 IN-LB	2,567 IN-LB	14,700 IN-LB	1,894 IN-LB	19,600 IN-LB	**Good**
2	2,246 IN-LB	2,567 IN-LB	14,700 IN-LB	2,615 IN-LB	19,600 IN-LB	**Good**
3	2,246 IN-LB	2,567 IN-LB	14,700 IN-LB	1,204 IN-LB	19,600 IN-LB	**Good**

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Beam To Column Connection: 3 TABS

Mconn= 0.7*Mconn-seismic
= 1,719 in-lb

CAPACITY OF CONNECTOR:

a) SHEAR CAPACITY OF TAB

Tab height = .625 in
Tab thickness= .125 in
Area = (0.625 in)*(0.125 in)
= .078 in^2

Fy= 50,000 psi

Pmax= A*Fv
= 1,563 lb

b) BEARING ON COLUMN:

Col. thickness= .075 in (14 GA.)
Bearing length= .625 in
Fu= 65,000 psi
Abrg.= .0469 in^2

Pmax brg.= Fu*Abrg
= 3,047 lb

c) MOMENT CAPACITY OF BRACKET:

SINCE TAB GOVERNS P1= 1,563 lb

Mconn. cap.= P1*4.5 + P2*2.5 + P3*0.5
= 1563 lb*4.5 + 868 lb*2.5 + 174 lb*0.5
= 9,288 in-lb > Mconn max, OK

C = P1 + P2 + P3
= P1 + (2.5/4.5)P1 + (0.5/4.5)P1
= 1.667 P1

P1 = 1,563 lb
P2 = (2.5/4.5)P1
= 868 lb
P3 = (0.5/4.5)P1
= 174 lb

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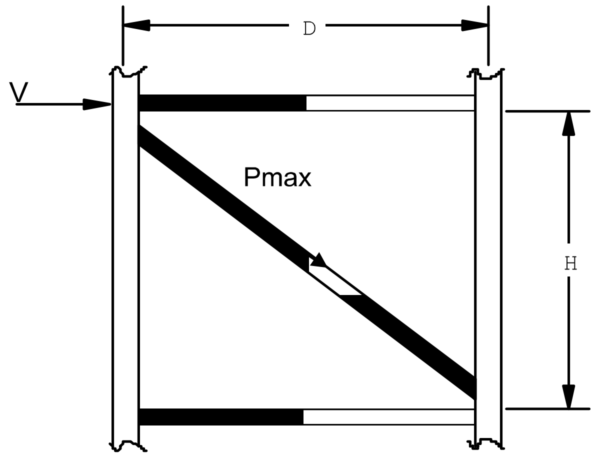
PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	34
CALCULATED BY	KE
DATE	1/28/2026

Transverse Brace Analysis

It is assumed that the transverse braces resist the seismic shear in tension and compression, with compression being critical in the design.



LOWER BRACE PANEL ELEVATION

H = 54.0 IN
D = 48.0 IN

Diagonal Member

Vtransv= 0.7*V
= 151 LB

Ldiag= [(D-3")^2 + (H-6")^2]^0.5
= 65.8 IN

Pmax= V*(Ldiag/D)
= 207.2 LB

(kl/r)max= (k * Ldiag)/r min
= 1 x 65.8 IN /0.3 IN
= 219.3 IN

Fe= $\pi^2 E/(kl/r)^2$
= 5,950 PSI

SINCE Fe<Fy/2,
Fn= Fe
= 5,950 PSI

Pn= AREA*Fn
= 0.12IN^2 * 5950 PSI
= 714 LB

Ωc= 1.92

Pallow= Pn/Ωc
= 714 LB /1.92
= 372 LB

fa/Fa= 0.56 < 1.0 OK

Horizontal Member

Similarly for horizontal members.

Pmax= 151 LB

fa/Fa= 0.14 < 1.0 OK

Horizontal Member

t = 0.035 IN
AREA = 0.120 IN^2
r min = 0.300 IN
Fy = 50,000 PSI

Diagonal Member

t = 0.035 IN
AREA = 0.120 IN^2
r min = 0.300 IN
Fy = 50,000 PSI

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Overturning Analysis

Fully loaded:

Movt = 20,939 IN-LB

M_{DL} = 150 LB x 48 IN / 2
= 3,600 IN-LB

d = 48.0 IN

M_{PL} = 350 LB x 48 IN / 2
= 8,400 IN-LB

Allowable Tension = 790 LB
Allowable Shear = 900 LB
\# of Anchors/Plate = 1

Vcol = 0.7*Vtotal/2
= 76 LB

Puplift = [0.7 x Movt - (0.6-0.14S_{DS}) x M_{DL} - (0.6-0.14S_{DS}) x M_{PL}] / d
= [0.7*20939 - (0.6-0.14*1.01)*3600 - (0.6-0.14*1.01)*8400] IN-LB / 48 IN
= 191 LB <=== UPLIFT

Interaction Equation

[191 LB/ 790 LB] + [76 LB/ 900 LB] = 0.33 < 1.2 Therefore OK

Top shelf loaded only:

Movt = 14,652 IN-LB

M_{DL} = 3,600 IN-LB

d = 48.0 IN

M_{PL} = 350 LB x 48 IN / 2
= 8,400 IN-LB

Allowable Tension = 790 LB
Allowable Shear = 900 LB
\# of Anchors/Plate = 1

Vcol = 0.7*Vtop/2
= 44 LB

Puplift = [0.7 x Movt - (0.6-0.14S_{DS}) x M_{DL} - (0.6-0.14S_{DS}) x M_{PLapp}] / d
= [0.7*14652 - (0.6-0.14*1.01)*3600 - (0.6-0.14*1.01)*8400] IN-LB / 48 IN
= 99 LB <=== UPLIFT

INTERACTION EQN.

[99 LB/790 LB] + [44 LB/900 LB] = 0.17 < 1.2 Therefore OK

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	36
CALCULATED BY	KE
DATE	1/28/2026

LRFD Load Combination - For Slab and Soil Analysis

		Resultant Load combination
1	1.4DL + 1.2PL	1.4DL + 1.2PL
2	1.2DL + 1.6PL + 1.6LL + 0.5(Lr or SL or RL)	1.2DL + 1.6PL
3	1.2DL + 0.85PL + (0.5LL or 0.8WL) + 1.6(Lr or SL or RL)	1.2DL + 0.85PL
4	1.2DL + 0.85PL + 0.5LL + 1.6WL + 0.5(Lr or Sl or RL)	1.2DL + 0.85PL
5	$(1.2+0.2S_{DS})DL + (1.2+0.2S_{DS})PL + 0.5LL + EL + 0.2SL$	$(1.2+0.2S_{DS})DL + (1.2+0.2S_{DS})PL + EL$
6	$(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})PLapp - EL$	$(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})PLapp - EL$

DL-total/col = 75 LB

LL-total/col = 525 LB

E = 436 LB

Load combination 1

Pmax = 1.4DL + 1.2PL
= 1.4 x 75 LB + 1.2 x 525 LB
= 735 LB

Load combination 2

Pmax = 1.2DL + 1.6PL
= 1.2 x 75 LB + 1.6 x 525 LB
= 930 LB

Load combination 3 & 4

Pmax = 1.2DL + 0.85PL
= 1.2 x 75 LB + 0.85 x 525 LB
= 536 LB

Load combination 5

Pmax = $(1.2+0.2S_{DS})DL + (1.2+0.2S_{DS})PL + EL$
= (1.2+0.2*1.01) x 75 LB + (1.2+0.2*1.01) x 525 LB + 436 LB
= 1,277 LB

Load combination 6

Pmax = $(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})PLapp - EL$
= (0.9-0.2*1.01) x 75 LB + (0.9-0.2*1.01) x 352 LB - 436 LB
= -138 LB

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PROJECT	AutoZone #10668
FOR	AutoZone
SHEET NO.	37
CALCULATED BY	KE
DATE	1/28/2026

Slab & Soil Analysis

The slab will be checked for puncture and bearing stress. If no puncture occurs, the slab is assumed to distribute the load over a larger area of the slab.

a) Puncture:

Pmax = **1,277 LB**

Fpunct = 2.66 x (F'c^0.5)
= 2.66 x (2500 PSI)^0.5
= 133 PSI
Apunct = [(Weff.+t)+(Deff.+t)] x 2 x t
= [(5 IN + 4 IN) + (2.25 IN + 4 IN)] x 2 x 4 IN
= 122 IN^2

fv/Fv = P/[(Apunct)(Fpunct)]
= 1277 LB/[122 IN^2 x 133 PSI x 0.55]
= **0.14** **< 1.0 OK**

b) Bearing:

ØBn = 0.85 x Ø x f'c x A1
= 13,148 LB

Pu / ØBn = 1277 LB / 13148 LB
= **0.10** **< 1.0 OK**

c) Slab Tension

Asoil = P/[1.0 x fs]
= 1277 LB/[1.0 x 1000 PSF/(144IN^2/FT^2)]
= 184 IN^2
L = Asoil^0.5
= (183.92 IN^2)^0.5
= 13.6 IN
B = [(Weff.)(Deff.)]^0.5 + t
= [5 IN x 2.25 IN]^0.5 + 4
= 7.4 IN
b = (L-B)/2
= (13.56 IN - 7.45 IN)/2
= 3.1 IN
Mconc = (w)(b^2)/2 = [(1.0)(fs)(b^2)]/[144 (IN^2/FT^2) x 2]
= [1.0 x 1000 PSI x (3.06 IN)^2]/[144 (IN^2/FT^2) x 2]
= 32 IN-LB
Sconc = 1 IN x (t^2)/6
= 1 IN x (4 IN)^2/6
= 2.67 IN^3
Fconc = 5 x Ø x f'c^0.5
= 5 x 0.55 x (2500 PSI)^0.5
= 137.5 PSI

fb/Fb = Mconc/[(Sconc)(Fconc)]
= 32.47 IN-LB/[(2.67 IN^3)(137.5 PSI)]
= **0.09** **< 1.0 OK**

Base Plate:

Weff. = 5.00 IN
Deff. = 2.25 IN
A1 = 11.3 IN^2

Concrete:

Thickness = 4.00 IN
f'c = 2,500 PSI

Soil:

fs = 1,000 PSF

Ø= 0.55