S. GORDIN STRUCTURAL DESIGN
& ENGINEERING SERVICES, Inc. (SGE)
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STRUCTURAL ENGINEERING REPORT

Project: Seismic Restraint for Optical Table

Location: Ss=2.50, S1=1.00

Client: MKS Instruments, Inc.

Code: 2019 CBC, 2018 IBC

SGE Job No. 520.043.139

May 2020

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Date: May 5, 2020

To: Mr. Warren Booth
Senior Product Manager
MKS Instruments, Inc.
1791 Deere Avenue
Irvine, CA 92606
Tel (949) 253-1866

Re: Structural Analysis and Design for
Optical Table Earthquake Restraint

SGE No.: 520.043.139

Dear Mr. Booth,

S. Gordin Structural Design & Engineering Services, Inc. (further referred to as "SGE") completed the engineering work on Structural Analysis and Design for the Earthquake Restraint.

This work was conducted based on MKS Instruments, Inc. PO # 1772657 dated April 20, 2020.

Please refer to the aforementioned approved proposal for all additional information, including the caveat and limitations.

1. EXISTING DOCUMENTATION

This proposal was developed upon the following documentation (ERS97):

1.1 Drawings by MKS Instruments, Inc.:

34773K	35712A	35718A
35703A	35715A	37192C
35704B	35716A	37194B
35711A	35717A	37195C
		37255B

1.2 2015 Structural Engineering Report by SGE on Seismic Restraint for Optical Table (SGE Job No. 515.052.369).

3. STRUCTURAL ANALYSIS BY SGE

3.1 The structural analysis by SGE was based on the following:

3.1.1. Governing design codes:
2018 International Building Code (IBC)
2019 California Building Code (CBC)
ASCE 7-16 (American Society of Civil Engineers)
ACI 318-14 (American Concrete Institute)
Steel Construction Manual 15th Edition (American Institute of Steel Construction)
AWS D1.3-08 Structural Welding Code – Sheet Steel (American Welding Society).

3.1.2 Design assumptions:

Light-gage (13ga) steel	ASTM A570 Grade 50
Structural steel	ASTM A36
Concrete	Normal weight concrete, 3,000 PSI strength in 28 days (minimum for California), 6” minimum uniform thickness
Tributary seismic mass	Per Item 3.2.1 below
Seismic force	Ss=2.50, S1=1.00 ap=1.0; Rp=2.50; Ω=2 (Lab Equipment, ASCE 7-16 Table.13.5-1)
Table location	At the ground floor, mid-height floor, and top floor (roof)
Table configuration	4’x6’ and 4’x20’ (4 isolators, 3 restraints) 4’x20’ (4 isolators, 4 restraints)
Restraint height	29-½” maximum from the floor.

3.1.3 Per request from MKS Instruments, Inc., only sleeve-type anchors were considered for the design of anchorage to concrete.

3.2 Commentary on some structural design issues (refer to drawings SD1 and SD2, Appendix A).

3.2.1. <u>Model.</u> The following was assumed for the purposes of this analysis/report:

a. The considered layouts are limited to the three cases presented on drawing SD1.
b. The combined center of gravity of the table and equipment is located within the height and plan limitations outlined by shaded diamond-shaped areas on drawing SD1.
c. Any conditions differing from those reflected on drawing SD1 are subject to additional structural investigation.

d. All tables are supported by vibration isolators (further referred to as "isolators," 4 per table) and earthquake restraints (or "towers," 3 or 4 per table). The isolators are assumed to resist vertical downward forces (gravity and seismic) only, while the restrains are capable of resisting only lateral and upward seismic forces.
e. Due to the deformability of the table and connections, the lateral forces on the table were assumed to be resisted by all available restraints.
f. This analysis considered only the resistance of the towers to the seismic forces specified in this report.
g. For the purposes of this analysis, the isolators were assumed as adequate for the resistance to all applicable (vertical/downward) forces at any possible location of the weight resultant force. The analysis of the isolators is beyond the scope of work by SGE.

3.2.2. <u>Codes.</u> The codes per Item 3.1.1 represent the basis for structural design as mandated by the IBC and CBC.

The seismic design basis (Ss=2.50, S1=1.00) was chosen by SGE and approved by MKS Instruments, Inc. to provide seismic forces that are conservative for most of California as well as for most of the continental United States.

3.2.3. <u>Anchors.</u> The seismic restraints experience lateral and vertical (upward only) earthquake forces due to table shifting and overturning (refer to drawings SD1 and SD2). As a result, the concrete anchors in the SGE design are subjected to pullout and shear forces. The tension forces were assumed to be resisted only by anchors along one of the tower faces, while the shear forces were assumed to be resisted by the rest of the anchors.

3.2.4. <u>Light-Gage Steel.</u> The performance of the light-gage steel components under the compression loads (for example, the faces of the 13-gage tower) is addressed in AISC Steel Design Manual. According to that code, only a certain portion of the compressed light-gage component may be considered effective in compressive resistance.

3.2.5. <u>Welding.</u> (1) Similarly to Item 3.2.4, welding of the tower to much thicker structural steel plates is only effective within the aforementioned effective portions of the tower perimeter. For example, for the 13 gage Grade 50 steel, only 3.82" of the 4"-to-10.5" of the tower face width is effective in compression.

(2) The centerlines of the holes for concrete anchors in the bottom plate (baseplate) are located at a distance of 0.75” from the tower. The effective length of the weld at each anchor is limited to the distance equal to 2x0.75”=1.5” which less than the spacing of the anchors.

(3) Welders of the light-gage tower shall be specially certified per AWS D1.3.

3.2.6. Constructability. Due to different tolerances for steel and concrete construction, the baseplate holes for steel-to-concrete connections have diameters that are larger than those for steel-to-steel connections.

3.3 The structural analysis by SGE revealed the following (refer to Appendix A).

3.3.1 The seismic restraint configured per Item 3.2.1 above and drawings SD1 and SD2 is generally adequate for the codes, loads, and assumptions per Item 3.1.2 above.

3.3.2 The resistance of the earthquake assembly appears to be limited by the strength of the anchorage to concrete.

The restraints are anchored to the floor (3,000 PSI minimum 28-day strength, normal weight concrete, minimum uniform thickness 6”) with HILTI HIT HY200 per ICC ESR 3187 (∅0.375” bolts, ∅0.65” HIS-N inserts minimum embedment - 4.38 inches.

3.3.3 Based on the capacity of the assembly, the maximum combined weight of the table and equipment per table shall be evaluated by the following formula:

W0 = 3,340*NR*KX*KZ*KH*KF [LBS]

W0 total maximum combined weight, lbs, of the table **and** of the payload secured on the table;
NR number of restraints per table (**3 or 4**);
KX coefficient for eccentric location of the resultant of the total table and payload weight - along 6' or 20' table dimension;
KZ coefficient for eccentric location of the resultant of the total table and payload weight - along 4' table dimension;
KH coefficient for hazardous payload – for installations involving quantities of toxic or explosive substances sufficient to be dangerous to the public or exceeding quantities per IBC Table 307.1.(2):

1.0 for non-hazardous payload
0.67 for hazardous payload;
KF coefficient for table location:
1.0 ground floor
0.5 mid-height floor
0.33 roof.

3.3.4 The findings of this report appear applicable for all tables measuring at least 4’x4’ and up to 5’x20’ with isolator/restraint height of 29 ½” maximum and configurations per Item 3.1.2 above.

3.3.5 Installation on floor slabs constructed over the corrugated decks and/or of the light-weight concrete may considerably limit the capacity of the anchors (to be considered on an individual basis).

3.3.6 The design earthquake was assumed to be generated by a site with Ss=2.50 and S1=1.00. For some sites, this high value may be too conservative, meaning that the payload on tables at such sites may be increased (to be considered on an individual basis).

3.3.7 All individual-basis analyses per, and similar to, Items 3.3.5 and 3.3.6, shall be requested from, and conducted by, MKS Instruments, Inc. and/or SGE.

We appreciate this and any other opportunity to be of service to you. Should you have any questions or need other assistance, please call SGE.

Respectfully submitted,
S. Gordin Structural Design & Engineering Services

REGISTERED PROFESSIONAL ENGINEER VYACHESLAV "STEVE" GORDIN, Ph.D. No. S-4311 STRUCTURAL STATE OF CALIFORNIA

Vyacheslav "Steve" Gordin, Ph.D.
Principal
Registered Structural Engineer
CA License S4311

Appendix A: Schematic Drawings
Appendix B: Structural Calculations

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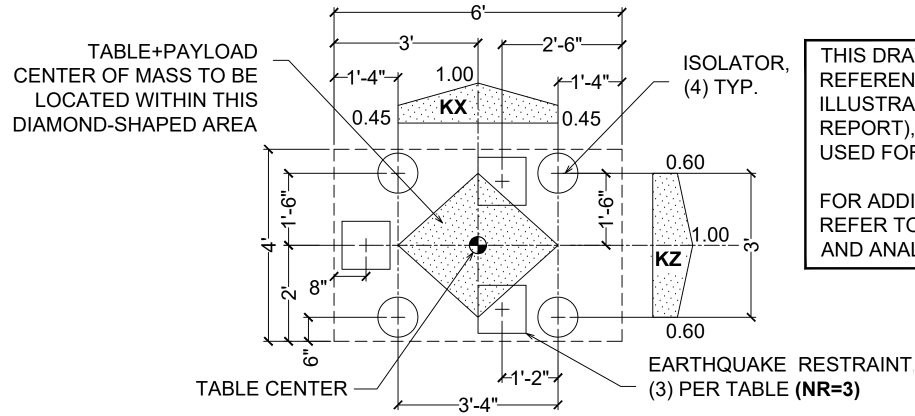
STRUCTURAL ENGINEERING REPORT

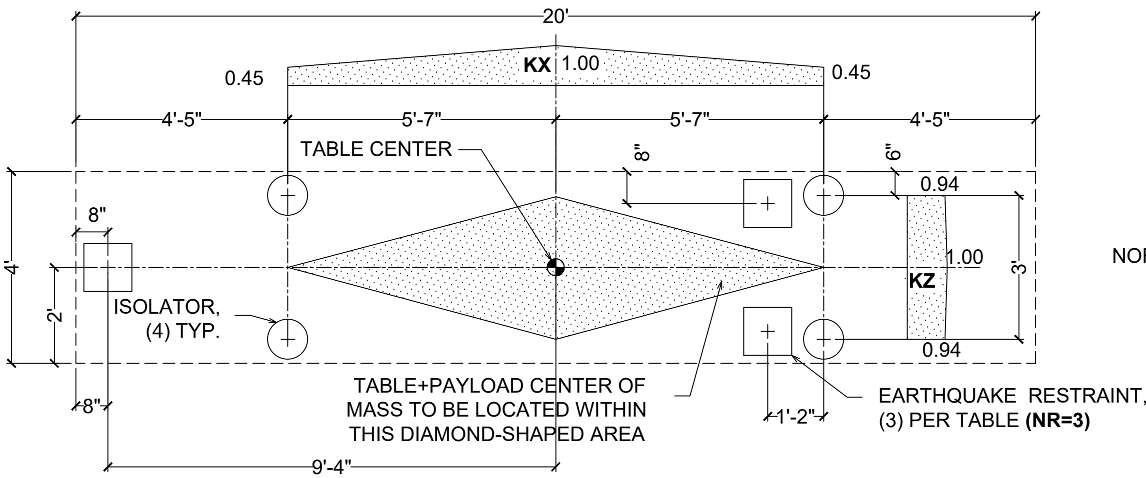
APPENDIX A:

SCHEMATIC DRAWINGS

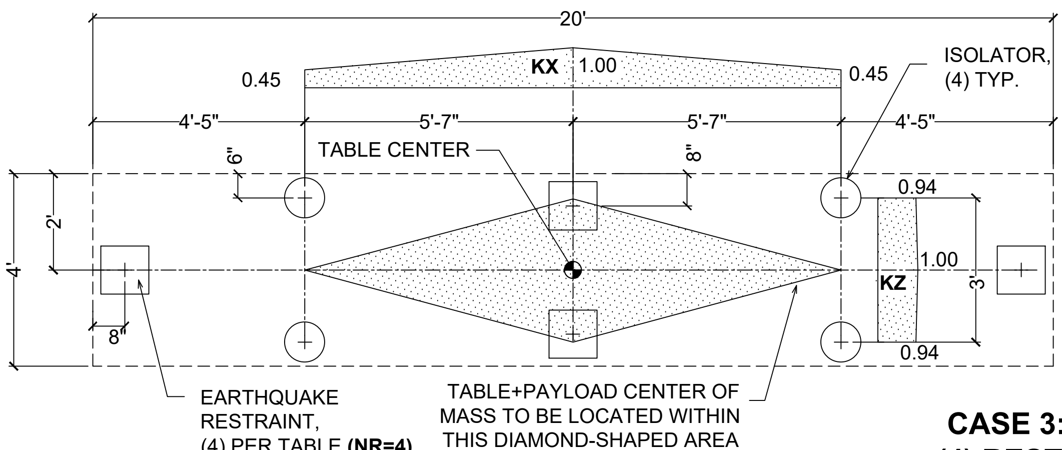
Project:	Seismic Restraint for Optical Table
Location:	Ss=2.50, S1=1.00
Client:	MKS Instruments, Inc.
Code:	2019 CBC, 2018 IBC
SGE Job No.	520.043.139



CASE 1:SHORT TABLE - (3) RESTRAINTS, NR=3



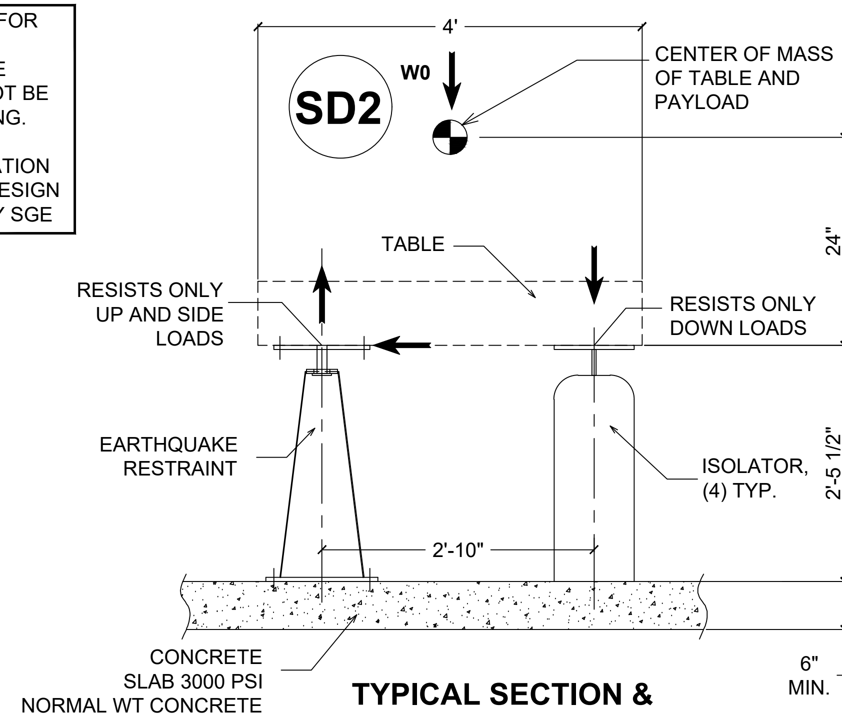
CASE 2: LONG TABLE - (3) RESTRAINTS (NR=3)



CASE 3: LONG TABLE (4) RESTRAINTS (NR=4)

THIS DRAWING IS ISSUED FOR REFERENCE ONLY (AS AN ILLUSTRATION TO THE SGE REPORT), AND SHOULD NOT BE USED FOR MANUFACTURING.

FOR ADDITIONAL INFORMATION REFER TO STRUCTURAL DESIGN AND ANALYSIS REPORT BY SGE



TYPICAL SECTION & LOADING DIAGRAM

May 5, 2020

SGE

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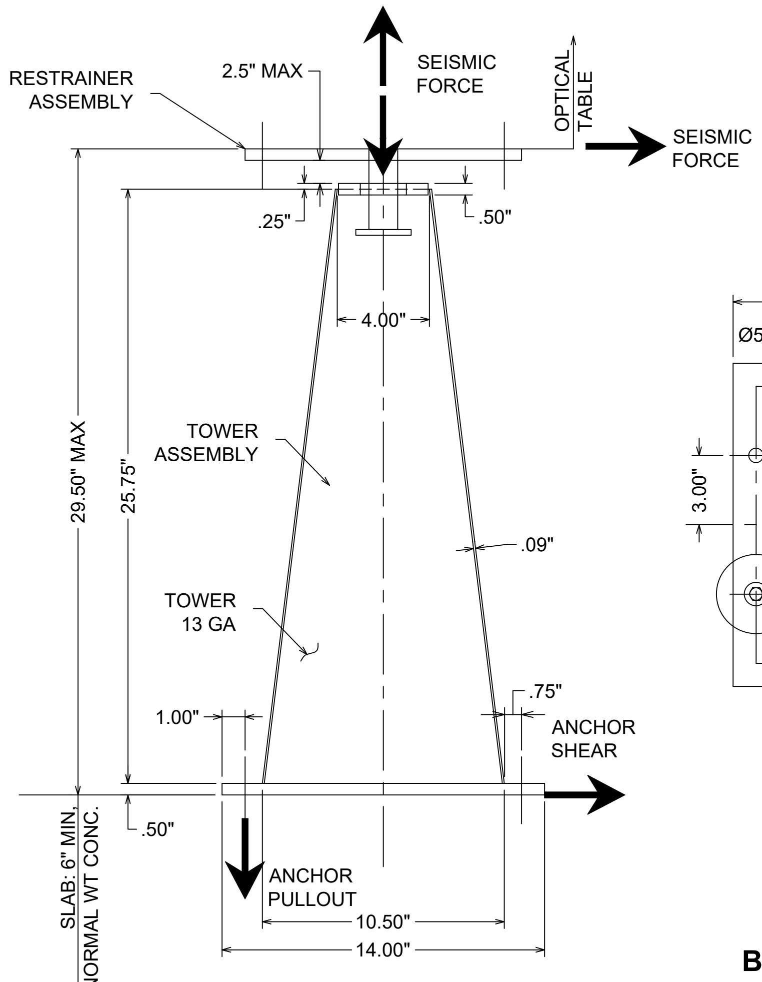
MKS INSTRUMENTS, INC.
IRVINE CA

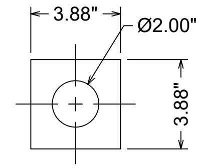
OPTICAL TABLE EARTHQUAKE RESTRAINT

GENERIC LAYOUTS & SECTION

SD1



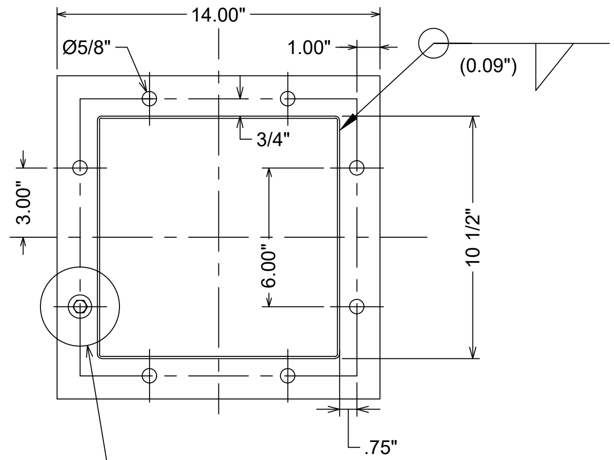
RESTRAINT ASSEMBLY



TOWER ASSEMBLY TOP PLATE

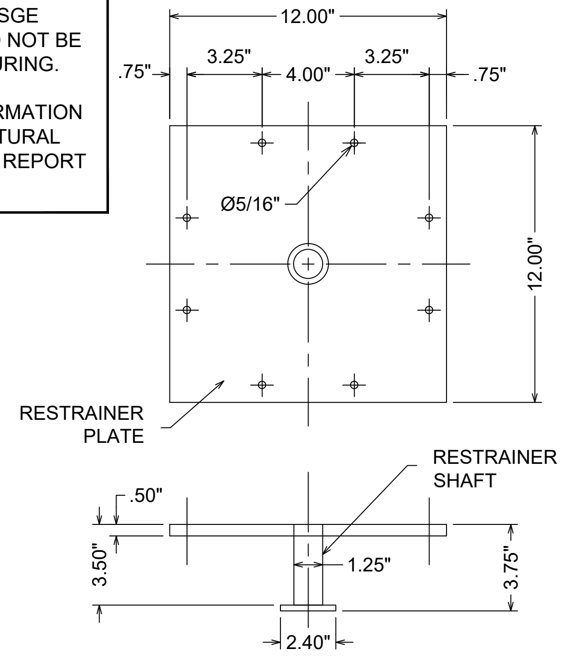
THIS DRAWING IS ISSUED FOR REFERENCE ONLY (AS AN ILLUSTRATION TO THE SGE REPORT), AND SHOULD NOT BE USED FOR MANUFACTURING.

FOR ADDITIONAL INFORMATION REFER TO 2015 STRUCTURAL DESIGN AND ANALYSIS REPORT BY SGE

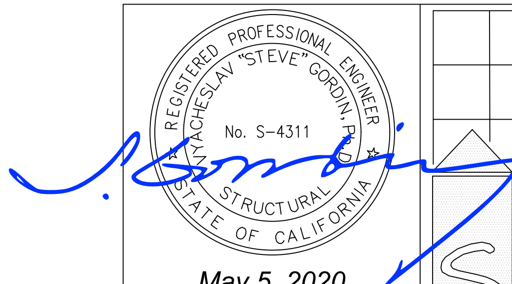


CONCRETE ANCHORS (8) PER RESTRAINT:
HILTI HIT HY 200 PER ICC ESR 3187
Ø3/8" ASTM A307 STEEL BOLT
TYPE N INSERT 4 3/8" MIN. EMBEDMENT
TYPE 1 CIRCULAR WASHER ASTM F436

RESTRAINT ASSEMBLY - BOTTOM PLATE (BASEPLATE)



RESTRAINER ASSEMBLY



May 5, 2020

W0 TOTAL MAXIMUM COMBINED WEIGHT, LBS, OF THE TABLE **AND** PAYLOAD
NR NUMBER OF RESTRAINTS PER TABLE (**3 OR 4**)
KX COEFFICIENT FOR ECCENTRIC LOCATION OF THE RESULTANT OF THE TOTAL TABLE AND PAYLOAD WEIGHT - ALONG 6' OR 20' TABLE DIMENSION
KZ COEFFICIENT FOR ECCENTRIC LOCATION OF THE RESULTANT OF THE TOTAL TABLE AND PAYLOAD WEIGHT - ALONG 4' TABLE DIMENSION
KH COEFFICIENT FOR HAZARDOUS PAYLOAD
- **1.0** FOR NON-HAZARDOUS PAYLOAD
- **0.67** FOR HAZARDOUS PAYLOAD

KF COEFFICIENT FOR TABLE LOCATION:
- **1.0** GROUND FLOOR
- **0.5** MID-HEIGHT FLOOR
- **0.33** ROOF

SD1

W0 = 3,340*NR*KX*KZ*KH*KF [LBS]

TOTAL MAXIMUM COMBINED WEIGHT (TABLE+PAYLOAD)

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MKS INSTRUMENTS, INC.
IRVINE CA

OPTICAL TABLE EARTHQUAKE RESTRAINT

GENERIC DETAILS | **SD2**

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STRUCTURAL ENGINEERING REPORT

APPENDIX B:

STRUCTURAL CALCULATIONS

Project: Seismic Restraint for Optical Table

Location: Ss=2.50, S1=1.00

Client: MKS Instruments, Inc.

Code: 2019 CBC, 2018 IBC

SGE Job No. 520.043.139

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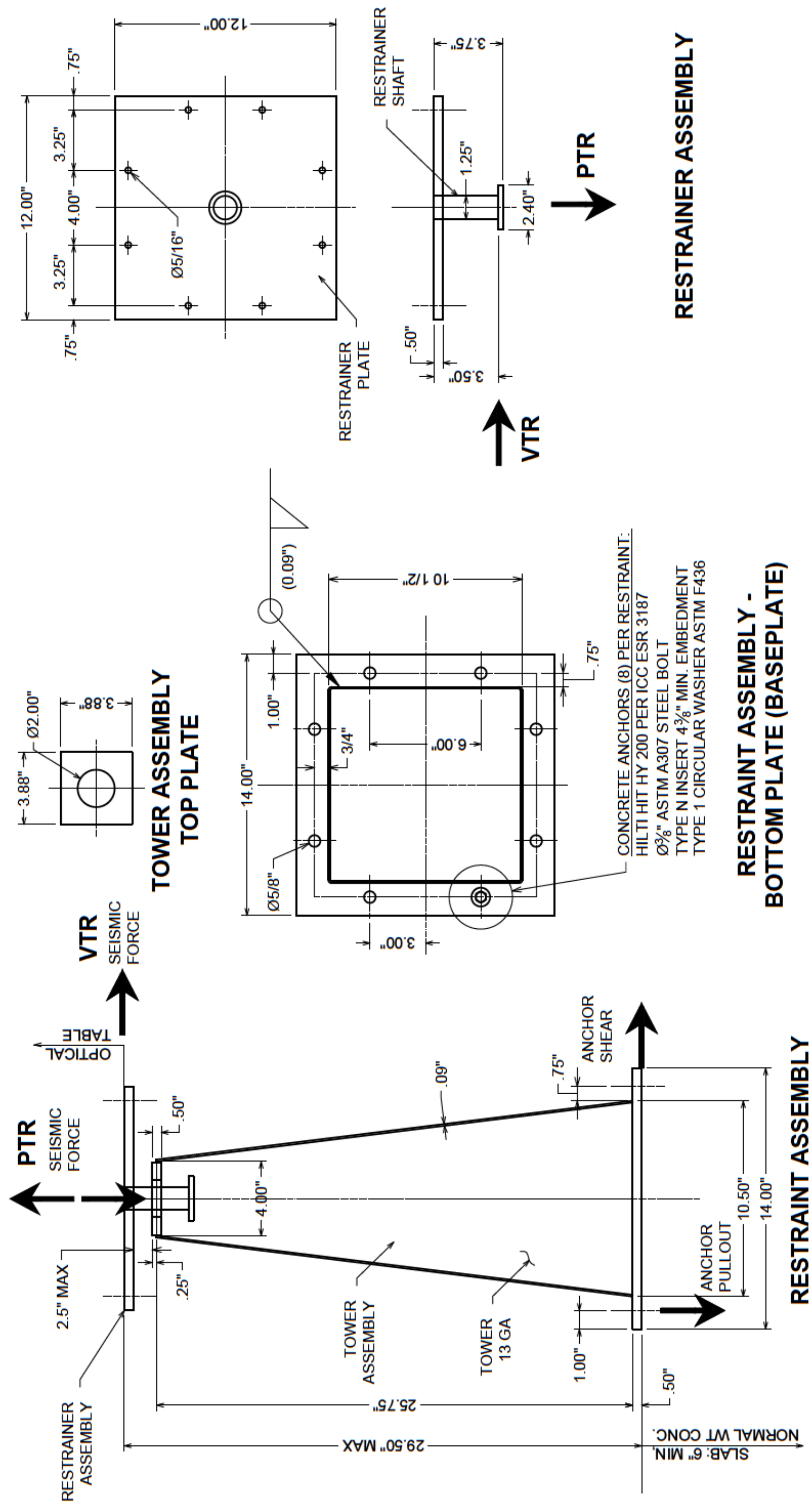
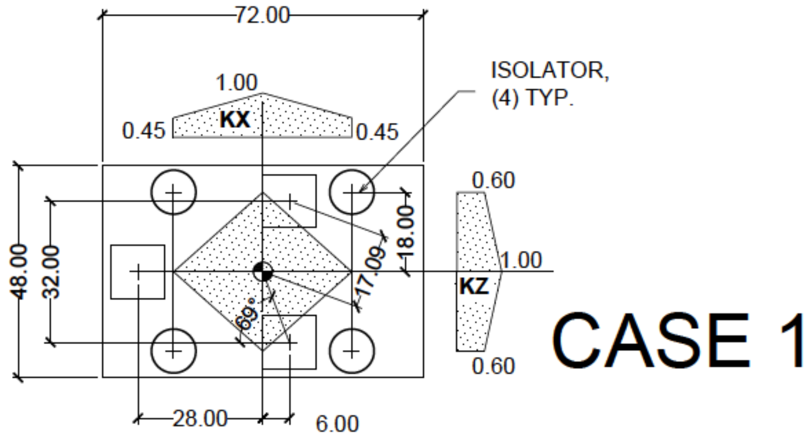
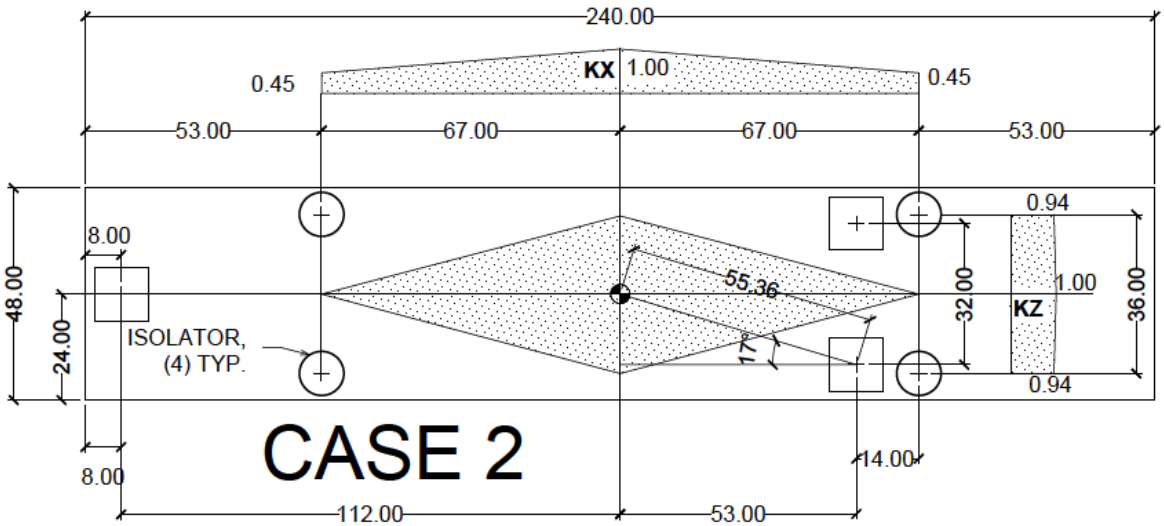
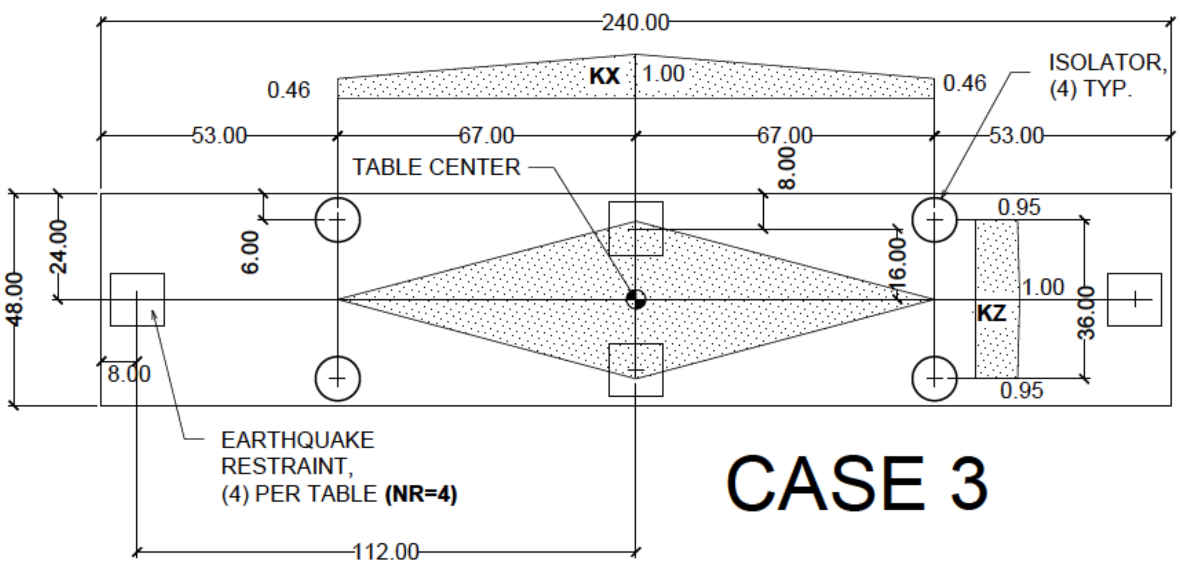
XX - Referenced page number of structural calculations


Isolator
Earthquake Restraint


RESTRAINER ASSEMBLY
RESTRAINER SHAFT
RESTRAINER PLATE
12.00"
.75"
3.25"
4.00"
Ø5/16"
1.25"
2.40"
3.75"
3.50"
.50"
PTR
VTR
TOWER ASSEMBLY TOP PLATE
Ø2.00"
3.88"
RESTRAINT ASSEMBLY - BOTTOM PLATE (BASEPLATE)
(0.09")
10 1/2"
14.00"
1.00"
3/4"
6.00"
Ø5/8"
3.00"
CONCRETE ANCHORS (8) PER RESTRAINT:
HILTI HIT HY 200 PER ICC ESR 3187
Ø3/8" ASTM A307 STEEL BOLT
TYPE N INSERT 4 3/8" MIN. EMBEDMENT
TYPE 1 CIRCULAR WASHER ASTM F436
RESTRAINT ASSEMBLY
VTR SEISMIC FORCE
PTR SEISMIC FORCE
OPTICAL TABLE
ANCHOR SHEAR
ANCHOR PULLOUT
TOWER ASSEMBLY
TOWER 13 GA
RESTRAINER ASSEMBLY
2.5" MAX
.25"
.09"
10.50"
25.75"
29.50" MAX
SLAB: 6" MIN. NORMAL WT CONC.


72.00
1.00
KX
0.45
0.45
ISOLATOR,
(4) TYP.
0.60
1.00
KZ
0.60
48.00
32.00
18.00
17.09
69°
28.00
6.00
CASE 1


240.00
0.45
KX
1.00
0.45
53.00
67.00
67.00
53.00
8.00
0.94
55.36
1.00
KZ
32.00
36.00
48.00
24.00
ISOLATOR,
(4) TYP.
17°
0.94
8.00
CASE 2
14.00
112.00
53.00


240.00
0.46
KX
1.00
0.46
ISOLATOR,
(4) TYP.
53.00
67.00
67.00
53.00
TABLE CENTER
8.00
0.95
6.00
16.00
1.00
KZ
36.00
48.00
24.00
8.00
0.95
EARTHQUAKE
RESTRAINT,
(4) PER TABLE **(NR=4)**
CASE 3
112.00

Determine SDS

Ss = 2.50 g

S1 = 1.00 g

Assuming site class D, by default:

Fa = 1.0

SMS = (Fa)(Ss) = 2.50 g

SDS = 2/3(SMS) = 1.67 g

ASCE 7-16
Table 11.4-1
Eq. 11.4-1
Eq. 11.4-3

Section 15.3.1 of ASCE 7-16 -
"for the condition where the weight of the nonbuilding structure is less than 25% of the combined effective seismic weights of the nonbuilding structure and supporting structure, the design seismic forces of the nonbuilding structure shall be determined in accordance with chapter 13 where the values of Rp and Ap shall be determined in accordance with section 13.1.6."

Therefore, the analysis will be conducted per chapter 13 of ASCE 7-16, as for nonstructural component, i.e. "lab equipment" for this project.

Seismic Lateral Force on Tributary Weight

FP =

$$\frac{0.4(ap)(SDS)(W0)\left(1+\frac{2Z}{h}\right)}{Rp/Ip}$$

ASCE 7-16 §13.3.1.1 Eq. 13.3-1

4

VS = FP x Ω   AP = 1.0   RP=2 ½   Ω = 2   SDS = 1.67 g

VS=K1*(IP)*(W0), where K1=

ASCE 7-16 Table 13.5-1

$$\frac{0.4(ap)(SDS)\left(1+\frac{2Z}{h}\right)(\Omega)}{Rp} = 0.5344 * \left(1+\frac{2Z}{h}\right)$$

Ground floor: Z/H = 0 → K1 = 0.5344

Mid height floor: Z/H = ½ → K1 = 1.0688

Top floor (roof): Z/H = 1 → K1 = 1.6032

Factor KF (Installation Floor)

KF =1.0
= (1+0) / (1+2x½) = 0.5
= (1+0) / (1+2x1) = 0.33

Factor KH (Hazardous Condition)

IP = 1.0 (non-hazardous) or 1.5 (hazardous)

KH = 1/1.0 = 1.0 (non-hazardous)
= 1/1.5 = 0.67 (hazardous)

Seismic Vertical Force

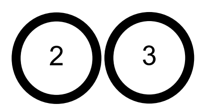
EV = ±0.2(SDS)(W0)
= ±(0.2)(1.67)(W0) = ±0.334(W0) = K2(W0) total — ASCE 7-16 §13.3.1.2

TVS = EV/NR = (K2)(W0)/NR, where K2 = 0.334 per restraint

Uplift on Restraints Due to Overturning

Tributary weight to each restraint:

WTR= W0/NR (NR = # of restraints per table)



NR = 3 (Case 1, 2)
= 4 (Case 3)

Lateral seismic force, total:

VS = (K1)(IP)(W0)

Lateral seismic force, tributary to, and applied on top of, each restraint:

VTR=(K1)(IP)(W0)/NR

Additional uplift on anchors from overall overturning of the table:

TOT=(VTR*H)/(R*NRT)

H = 53.5” height of center of mass above floor, TYP

R = 34” design distance between restraints and isolator

NRT = 1 #of restraints participating in overturning resistance

TOT = (K1*IP*W0*H)/(NR*R*NRT) =
= (K1*IP*W0*53.5) /(NR*34*1) =1.574(K1)(IP)(W0)/NR
= 0.52*K1*(IP)*(W0) Case 1, 2 (NR=3)
= 0.39*K1*(IP)*(W0) Case 3 (NR=4)

Seismic vertical uplift per restraint:

TVS = (K2)(W0)/NR

= 0.111(W0) Case 1, 2 (NR=3)

= 0.083(W0) Case 3 (NR=4)

Restraint Strength Based on Anchor Capacity

PA = MTR/LE + PTR/N ≤ 8.0 Kips

MTR = VTR*HR, IN-K moment at bottom of each restraint = V*HR

= 9.833(K1)(IP)(W0) CASE 1, 2 (NR=3)

= 7.375(K1)(IP)(W0) CASE 3 (NR=4)

PTR = TOT+TVS total uplift on restraint

HR = 29.5" height of restraint

LE = 7.5” effective moment arm for anchors

N = 4 # of anchors per side (in anchor groups)

(2) anchors per side = (1) anchor group

8 Kips LRFD capacity of anchor group in tension

Only (2) anchors out of (8) considered effective for moment resistance.

Shear is resisted by the rest of the anchors (in compression zone).

PA = 5.25(W0)/7.5 + 0.389(W0)/4 ≤ 8 K

Assuming:

IP = 1 (non-hazardous)

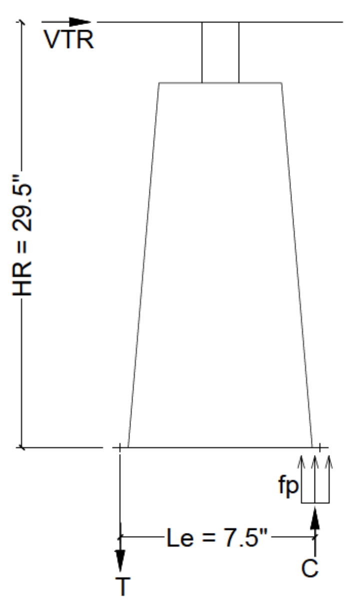
NR = 3

Z/H = 0

Therefore:

W0 ≤ 10.03 K, or

WTRA = W0/NR ≤ 3.34 K



SUMMARY

ANALYSIS - CENTERED FORCE

NA=number of anchors in group
TA=Total number of anchors
WTRA= weight per restraint anchor prespective
WTRS=weight per restraint from steel perspective
WTRW= weight per restraint from weld perspective
W TBL = total max weight of table and load

TOT=tension from overturning
P=Tension from Vertical Seismic and Overturning consideration
V=Shear on restraint
M=Moment on restraint
$vw=\sqrt{(P^2+V^2)}$ weld shear

	CASE	IP	R	H	HR	NR	K1	K2	PER RESTRAINT TOT/(W0*IP)	P/(W0*IP)	V/(W0*IP)	M/(W0*IP)	vw
			IN	IN	IN				K	K	K	IN-K	K
GROUND FLOOR	1, 2	1.00	34.00	53.50	29.50	3.00	0.5344	0.331	0.28	0.39	0.178	5.255	0.429
MID-HEIGHT FLOOR	1, 2	1.00	34.00	53.50	29.50	3.00	1.0688	0.331	0.56	0.67	0.356	10.510	0.760
TOP FLOOR	1, 2	1.00	34.00	53.50	29.50	3.00	1.6032	0.331	0.84	0.95	0.534	15.765	1.091

RETAIN SHAFT: TTR=P VTR=V

	CASE	NA	TA	WTRA	WTRS	WTRW	WTR (MIN)	W TBL	TTR	VTR
			K	K	K	K	K	K	K	K
GROUND FLOOR	1, 2	2.000	8.00	3.34	17.84	7.21	**3.34**	**10.02**	1.31	1.79
MID-HEIGHT FLOOR	1, 2	2.000	8.00	1.70	8.92	3.68	**1.70**	**5.10**	1.14	1.82
TOP FLOOR	1, 2	2.000	8.00	1.14	5.95	2.47	**1.14**	**3.42**	1.08	1.83

	CASE	IP	R	H	HR	NR	K1	K2	PER RESTRAINT TOT/(W0*IP)	P/(W0*IP)	V/(W0*IP)	M/(W0*IP)	vw
			IN	IN	IN				K	K	K	IN-K	K
GROUND FLOOR	3	1.00	34.00	53.50	29.50	4.00	0.5344	0.331	0.21	0.29	0.134	3.941	0.322
MID-HEIGHT FLOOR	3	1.00	34.00	53.50	29.50	4.00	1.0688	0.331	0.42	0.50	0.267	7.882	0.570
TOP FLOOR	3	1.00	34.00	53.50	29.50	4.00	1.6032	0.331	0.63	0.71	0.401	11.824	0.818

RETAIN SHAFT

	CASE	NA	TA	WTRA	WTRS	WTRW	WTR (MIN)	W TBL	TTR	VTR
			K	K	K	K	K	K	K	K
GROUND FLOOR	3	2.000	8.00	3.34	17.84	7.21	**3.34**	**13.36**	0.98	1.79
MID-HEIGHT FLOOR	3	2.000	8.00	1.70	8.92	3.68	**1.70**	**6.80**	0.86	1.82
TOP FLOOR	3	2.000	8.00	1.14	5.95	2.47	**1.14**	**4.56**	0.81	1.83

S. Gordin Structural Design &
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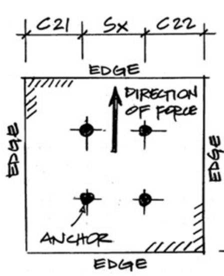
Irvine CA
Tel. (949) 552-5244

ANCHORAGE TO CONCRETE ~ EPOXY ANCHOR ~ HILTI HIT-HY 200

REFERENCES		
	ACI	ACI 318-14
	ESR	ICC ESR 3187

DESIGN PARAMETER	NAME	FORMULA OR SWITCH	VALUE	UNIT	?	COMMENT	REFERENCE
FORCES & CONDITIONS							
FACTORED PULLOUT FORCE	Nn1		**8.00**	K			7
FACTORED SHEAR FORCE	Vn1		**0.00**	K			
OPTIONAL FORCE FACTOR	KF		***1.00***				
TEMPERATURE (ºF) AND TEMPERATURE RANGE	T		130				ESR TBL 14
DESIGN PULLOUT FORCE	Nan	Nn1*KF	***8.00***	K			
DESIGN SHEAR FORCE	Vn	Vn1*KF	***0.00***	K		SDC C-F	ACI 17.2.3.4.4
SEISMIC COEFF (TENSION, CONCRETE ONLY)	ksdc		***0.75***				
FACTOR TENSION FORCE BY **Ω** Y/N	Ω	**N**	***1.00***		OK		ACI 17.2.3.4.3 (d)
FACTOR SHEAR FORCE BY **Ω** Y/N	Ω	**N**	***1.00***		OK		
CONCRETE STRENGTH (NWC)	f'c		***3,000***	PSI			
INSTALLATION CONDITION		DRY = "D"; WET/SATURATED="W"	**D**				ACI 17.5.1.3
GROUT PADS (SHEAR STEEL ONLY)	kg	**N**	***1.00***				
CRACKED CONCRETE Y/N		**N**					
GEOMETRY							
# OF ANCHORS IN THE GROUP, EFFECTIVE							
STEEL & CONCRETE, TENSION	nt		***2.00***			<=4	
CONCRETE, SHEAR	nv		***2.00***				
STEEL, SHEAR	ns		***2.00***				
ALONG LOADED EDGE	NALE		***2.00***				
DIAMETER							
ANCHOR	da		***0.375***	IN			
INSERT	d		***0.650***	IN			
SPECIFIED STRENGTH OF STEEL							
ANCHOR, TENSILE	fut		***75***	KSI		F1554 GR 55 OR SIM	
ANCHOR, YIELD	fy		***55***	KSI	OK		
fyt<=125,000 PSI; fyt<=1.9fy			105	KSI			
	futa		75	KSI			
INSERT, TENSILE	fut		***75***	KSI			
ANCHOR, YIELD	fy		***55***	KSI			
INSERT/ANCHOR(S) EMBEDMENT, ASSUMED	hef		***4.33***	IN	OK		
INSERT/ANCHOR EMBEDMENT, MINIMUM	hef min		2.38	IN			ESR TBL 14
PAD THICKNESS, MINIMUM	tp*		5.63	IN			ESR TBL 12
PAD THICKNESS, ASSUMED	tp		***6.00***	IN	OK		



DESIGN PARAMETER	NAME	FORMULA OR SWITCH	VALUE	UNIT	?	COMMENT	REFERENCE
ANCHOR SPACING							
DIRECTION 1 (MINIMUM)	sx	ACROSS SHEAR FORCE	***6.00***	IN			
DIRECTION 2 (MAXIMUM)	sa	ALONG SHEAR FORCE	***12.00***	IN			
ALONG LOADED EDGE	SL		***6***	IN			ESR TBL 12
MIN. ANCHOR SPACING	smin		1.88	IN	OK		
		3hef	12.99	IN			
AVAIL. WIDTH OF HALF-PYRAMID BASE	wpa		***12.00***	FT			
ANCHOR EDGE DISTANCE							
DIRECTION 1	c11	MIN ALONG SHEAR FORCE	***12.00***	IN	OK		
	c12	MAX ALONG SHEAR FORCE	***12.00***	IN	OK		
DIRECTION 2	c21	MIN ACROSS SHEAR FORCE	***12.00***	IN	OK		
	c22	MAX ACROSS SHEAR FORCE	***12.00***	IN	OK		
		1.5hef	**6.50**	IN			
MIN. EDGE DIST	cmin	6*d	1.88	IN	OK		17.7.3, 17.7.4

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DESIGN PARAMETER	NAME	FORMULA OR SWITCH	VALUE	UNIT	?	COMMENT	REFERENCE
STEEL STRENGTH, TENSION			**ANCHOR**			**INSERT**	
THREADS PER INCH	ntr		16.00			11	
EFFECTIVE AREA	Ase=	π/4(d0-.9743/ntr)^2	0.0775	IN^2		0.2476	
NOM. STRENGTH OF ANCHOR GROUP - STEEL	Ns	nt*(Ase)futa	**11.62**	K		37.13	
STEEL STRENGTH REDUCTION FACTOR	***φS***		0.75			0.75	ACI 17.4.1.2
DESIGN STRENGTH, STEEL		***φS*Ns***	8.72	K		27.85	
	NS1		**8.72**	K			
	NS2	1.2NS1	**13.95**	K			
CONCRETE BREAKOUT STRENGTH, TENSION							
PROJ. AREA OF TENSION FAILURE SURFACE FOR ANCHOR GROUP							
nt=1 CLOSE TO EDGE	AN1c	(c11+c12)(c21+c22)	-	IN^2		cij≤1.5hef	ACI 17.4.5.2
nt=1 AWAY FROM EDGE	AN0	9hef^2	169	IN^2			
nt=2 CLOSE TO EDGE	AN2c	(c11+c12)(c21+sx+c22)	-	IN^2			
nt=2 AWAY FROM EDGE	AN2a		247			cij≤1.5hef,	
nt=4 CLOSE TO EDGE	AN4c	(c11+sa+c12)(c21+sx+c22)	-	IN^2		si≤3hef	
nt=4 AWAY FROM EDGE	AN4a		-	IN^2			
		n*AN0	337				
	AN	<=n*AN0	**247**	IN^2			
	kc		**24**			UNCRACKED	ESR TBL 12
BASIC BREAKOUT STRENGTH IN CONCRETE	Nb	kc*(f'c)$^{1/2}$*(hef)$^{3/2}$	**11.84**	K			ACI 17.4.2.2a
ECCENTRICITY OF PULLOUT FORCE	e'N1		***0.00***	IN			
	e'N2		***0.00***	IN			
MODIFICATION FACTOR FOR ECCENTRICITY	$\Psi 11$	[1+2e'N/(3hef)]$^{-1}$	1.00				ACI 17.4.2.4
	$\Psi 12$	[1+2e'N/(3hef)]$^{-1}$	1.00				
	$\Psi 1$	$\Psi 11 * \Psi 12$	1.00				
MODIFICATION FACTOR FOR EDGE EFFECT			1.00			c1>=1.5hef	ACI 17.4.2.5a
			-			c1<1.5hef	ACI 17.4.2.5b
	$\Psi 2$		1.00				
MODIF FACTOR FOR CRACKED TENSION ZONE	$\Psi 3$	IF (f_t < f_r) = 1.25, 1.00	1.25			NO TENSION CRACKS	
NOMINAL CONCRETE BREAKOUT STRENGTH							
FOR SINGLE ANCHOR	Ncb	(AN/AN0) ($\Psi 2$)($\Psi 3$)Nb	14.81	K			ACI 17.4.2.1a
FOR GROUP OF ANCHORS	Ncbg	(AN/AN0) (Ψ**1**) ($\Psi 2$)($\Psi 3$)Nb	21.67	K			ACI 17.4.2.1b
STRENGTH REDUCTION FACTOR	***φC1***	DUCTILE FAILURE	0.75				ACI 17.3.3(b)
DESIGN BREAKOUT STRENGTH		***φC1*Ncbg***	**16.25**	K			
CONCRETE PULLOUT STRENGTH, TENSION							
MIN. EMBEDMENT	hefm		3	IN			ESR TBL 12
MINIMUM SPACING	smin		1.88	IN			ESR TBL 12
BOND STRENGTH IN CONCRETE							
FACTOR FOR f'c>2500 PSI	kfc		1.02				ESR TBL 14^2
UNCRACKED	τ		2,261	PSI			ESR TBL 14
CRACKED (IF APPLICABLE)	$\tau 1$		2,261	PSI		UNCRACKED	ESR TBL 14
	kcc	3.1-0.7h/hef, h/hef<=2.4	2.13				ESR 4.1.10.2
CRITICAL EDGE DISTANCE	cac	hef*($\tau 1$/1,160)$^{0.4}$*kcc	12.04	IN			ESR 4.1.10.2
	cna	10da*(τ *uncr*/1,100)$^{0.5}$	9.23	IN			ACI 17.4.5.1d
	cc1	MIN(cac, cna)	**9.23**	IN			
MODIFICATION FACTORS FOR:							
POST INSTALLED ANCHORS	Ψ_{CPNA}		**1.00**			cmin≥cc1	ACI 17.4.5.5a
		cmin/cc1	**-**			cmin<cc1	ACI 17.4.5.5b
EDGE EFFECTS	Ψ_{EDNA}		**1**			cmin≥cc1	ACI 17.4.5.4a
		0.7+0.3*cmin/cc1	**N/A**			cmin<cc1	ACI 17.4.5.4b
FOR ECCENTRICITY	Ψ_{ECNA}		**1.00**			NO ECCENTRICITY	ACI 17.4.5.3
STRENGTH REDUCTION FACTORS:							
FOR BOND IN SEIS. CATEGORIES C-F	α_{NS}		**0.88**				ESR TBL 14
STRENGTH REDUCTION FACTOR	$\phi 1$		**0.65**				ESR TBL 14

DESIGN PARAMETER	NAME	FORMULA OR SWITCH	VALUE	UNIT	?	COMMENT	REFERENCE
							PULLOUT, CONTINUED
PROJ. AREA OF PULLOUT FAILURE SURFACE FOR ANCHOR GROUP							
nt=1 CLOSE TO EDGE	AN1c 1	(c11+c12)(c21+c22)	-	IN^2		c1<cc1	ACI 17.4.5.1
nt=1 AWAY FROM EDGE	AN0 1	$(2*cac)^2$	341	IN^2		c1>cc1	
nt=2 CLOSE TO EDGE	AN2c 1	(c11+sx+c12)(c21+c22)	452	IN^2		c1<cc1; sx<2cc1	
nt=2 AWAY FROM EDGE	AN2a 1		-	IN^2		c1>cc1; sx<2cc1	
nt=4 CLOSE TO EDGE	AN4c 1	(c11+sx+c12)(c21+sa+c22)	-	IN^2		c1<cc1; c2<cc1; sx<2cc1; sa<2cc1	
nt=4 AWAY FROM EDGE	AN4a 1		-	IN^2		c1>cc1; c2>cc1; sx<2cc1; sa<2cc1	
		n*AN0	682	IN^2			
	AN 1	<=n*AN0 1	**452**	IN^2			
	Na0	$\tau 1 * \pi$ *d*hef*α_{NS}	17.6	K			ACI 17.4.5.2
NOMINAL STATIC PULLOUT (BOND) STRENGTH							
FOR SINGLE ANCHOR	Na	(AN1/AN01)* Ψ_{EDNA}*Ψ_{CPNA}*Na0	**17.6**	K			ACI 17.4.5.1a
FOR GROUP OF ANCHORS	Ncbg	(AN1/AN01)* Ψ_{EDNA}*Ψ_{ECNA}*Ψ_{CPNA}*Na0	**23.3**	K			ACI 17.4.5.1b
DESIGN PULLOUT STRENGTH		**$\phi 1$ * Ncbg**	**15.2**	K			
ANCHOR GROUP TENSION STRENGTH							
STEEL Ns			***8.7***	K			
CONCRETE Nc			***15.2***	K			
DUCTILE STEEL ANCHOR Y/N		**Y**					
STEEL STRENGTH GOVERNS Y/N		**Y**					
CONSERV., NO SUPPL REINF. , COND B, Y/N		**Y**					
FACT'D TENSILE STRENGTH, ANCHOR GROUP		***MIN(Ns, Nc*ksds)***	***8.72***	K	***OK***		
SHEAR							
STEEL STRENGTH IN SHEAR	Vs	ns*kg*n*0.6*Ase*fut	**6.97**	K			ACI 17.5.1.2b
REDUCTION, SEISMIC SHEAR (STEEL ONLY)	α_{VS}		0.70				ESR TBL 11
STRENGTH REDUCTION FACTOR	$\phi 2$		0.70				ESR TBL 11
CONCRETE BREAKOUT STRENGTH (SHEAR)							
SHEAR FORCE PARALLEL TO EDGE Y/N	ksd	**N**	1.00				
SHEAR FORCE ECCENTRICITY	e'V		***0.00***		OK		
MODIFICATION FACTORS FOR SHEAR STRENGTH:							
FOR ECCENTRICITY	Ψ_{ECV}	$[1+2*e'v/(3*C1)]^{-1} \leq 1$	1.00			NO ECC	ACI 17.5.2.5
EDGE EFFECTS	Ψ_{EDV}		**-**			ca2/ca1≥1.5	ACI 17.5.2.6a
		0.7+0.3*cmin/cc1	0.90			ca2/ca1<1.5	ACI 17.5.2.6b
FOR TENSION IN THE ANCHORING ZONE							
CRACKING IN THE TENSION ZONE		**N**					
SUPPLEMENTARY REBAR >=#4		**Y**					
	Ψ_{CV}		1.40				ACI 17.5.2.7
			-			ha/c1≥1.5	
	Ψ_{HV}		1.73			ha/c1<1.5	ACI 17.5.2.8
			1.73				
LOAD BEARING ANCHOR LENGTH, SHEAR	Le		4.33	IN		L<=8d0	ACI 17.5.2.2
	1.5c1		18.00	IN			
PAD THICKNESS	tp		6.00	IN			
DEPTH OF SHEAR FAILURE HALF-PYRAMID BASE	dp	MIN(1.5c1,tp)	6.00	IN			
ANCHOR SPACING ALONG LOADED EDGE	SL		6.00	IN			
	cef						
EDGE DISTANCE ACROSS SHEAR FORCE	ca		12.00	IN			
	cd	MIN(1.5c1,c21, tp)	6.00	IN			
BASIC BREAKOUT STRENGTH, SINGLE ANCHOR	Vb	$7(Le/d)^{0.2}(d)^{1/2}(f'c)^{1/2}(c1)^{1.5}$	18.78	K			ACI 17.5.2.2a
		$9(f'c)^{1/2}(c1)^{1.5}$	20.49	K			ACI 17.5.2.2b
			18.78	K			
# OF ANCHORS ALONG LOADED EDGE	NALE		2.00				

DESIGN PARAMETER	NAME	FORMULA OR SWITCH	VALUE	UNIT	?	COMMENT	REFERENCE
							SHEAR, CONTINUED
WIDTH OF SHEAR FAILURE HALF-PYRAMID BASE							
GROUP	wp	2*1.5c11+(NALE-1)*SL	42.00	IN			
		c21+1.5c11+(NALE-1)*SL	36.00	IN			
		c21+c22+(NALE-1)*SL	30.00	IN			
			30.00	IN			
SINGLE	wp1	MIN [3c11,(c21+c22)]	24.00	IN			
DESIGN WIDTH OF HALF-PYRAMID BASE	wpd	Choose from (wp,wp1)	**30.00**	IN			
AREA OF SHEAR FAILURE HALF-PYRAMID BASE							
SINGLE			144	IN^2			
ACTUAL	AV	dp*wpd	180	IN^2			
SINGLE, DEEP CONCRETE	AV0	$4.5c11^2$	648	IN^2			ACI 17.5.2.1c
NOMINAL CONCRETE BREAKOUT STRENGTH							
		AV/AV0(Ψ_{EDV}*Ψ_{ECV}*Ψ_{HV})Vb	9	K			ACI 17.5.2.1a
ANCHOR GROUP		AV/AV0(Ψ_{EDV}*Ψ_{ECV}*Ψ_{HV}*Ψ_{HV})Vb	**43**	K			ACI 17.5.2.1b
		IF MIN(c11,c21,c22>=tp) -> PRYOUT STRENGTH CONTROLS					
CONCRETE PRYOUT STRENGTH IN SHEAR							ACI 17.5.3
	kcp		***2.00***			hef>=2.5 IN	ACI 17.5.3.1a
PRYOUT STRENGTH, SINGLE ANCHOR	Vcp	kcp*Ncb	**29.61**	K			ACI 17.5.3.1b
PRYOUT STRENGTH, ANCHOR GROUP	Vcpg	kcp*Ncbg	**43.34**	K			
ANCHOR GROUP NOMINAL STRENGTH, SHEAR							
STEEL Vs		$\phi 2$ ***Vs*** α ***vs**	***3.42***	K			
CONCRETE Vc		$\phi 2$ ***Vc***	***30.34***	K		***	
DUCTILE STEEL ANCHOR Y/N		***Y***					
STEEL STRENGTH GOVERNS Y/N		***N***					
CONSERV., NO SUPPL REINF. , COND B, Y/N		***Y***					
FACTORED SHEAR STRENGTH, GROUP	***ϕV***	***MIN(Ns, Nc)***	***3.42***	K	***OK***		

STRENGTH DESIGN INTERACTION SUMMARY

					ACI 17.6
KN	Ω^* (Nu/FNn)<=1.0	**0.92**	OK		ACI 17.6.1
KV	Ω^* (Vu/FVn)<=1.0	**0.00**	OK		ACI 17.6.2
	$(KN)^{5/3}+(KV)^{5/3}\leq 1$	**-**	OK		R17.6

DUCTILITY CHECK		N/A for "x Ω" cases [D 17.2.3.4.3(d)]			
PER ANCHOR GROUP (na ≥ 1)					
NOMINAL SHEAR STRENGTH, STEEL	VS		6.97	K	
NOMINAL SHEAR STRENGTH, CONCRETE	VC		43.34	K	
SHEAR DEMAND	V		0.00	K	
NOMINAL TENSILE STRENGTH					
STEEL	TSU		13.95	K	ACI 17.2.3.4.3a
CONCRETE, BREAKOUT	TCU1		21.67	K	
CONCRETE, PULLOUT	TCU2		26.49	K	
CONCRETE, MIN	TCU		21.67	K	
TENSILE DEMAND	T		8.00	K	
		UTILIZATION RATIOS			
SHEAR, STEEL	kvs	V/VS	0.000		ACI R17.2.3.4.3
SHEAR, CONCRETE	kvc	V/VC	0.000		
TENSION, STEEL	kts	T/TS	0.574		
TENSION, CONCRETE	ktc	T/TC	0.369		
TOTAL, STEEL	KS	kvs+kts	0.574		
TOTAL, CONCRETE	KC	kvc+ktc	0.369		

Restraint Strength Based on Tower Capacity

By inspection, compression governs over tension.

Effective properties of restraint tower:

BT/TT = 10.5”/0.09” = 116

$$1.40 * \sqrt{\frac{E}{FY}} = 1.4 * \sqrt{\frac{29000}{50}} = 33.7$$

AISC 360-16
Table B4.1a

33.7 << 116 → slender element

Effective width of compressed flange:

AISC 360-16
Eq. C-E7-1

BE =1.92*0.09*24*[1-(0.38*24)/116]

=3.82” <10.5” O.K.

14

BE/2 = 1.92”

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Rev: 580006
User: KW-0602158, Ver 5.8.0, 1-Nov-2006
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Built-Up Section Properties

restraint 2011.ecw:Calculations

Description TOWER BTTM

General Information

	Type...					X cg	Y cg
#1	Rectangular	Height	0.0900 in	Width	10.5000 in	5.2500 in	0.0000 in
#2	Rectangular	Height	3.5400 in	Width	0.0900 in	0.0000 in	1.7700 in
#3	Rectangular	Height	3.5400 in	Width	0.0900 in	10.5000 in	1.7700 in
#4	Rectangular	Height	1.9200 in	Width	0.0900 in	0.0000 in	9.5400 in
#5	Rectangular	Height	1.9200 in	Width	0.0900 in	10.5000 in	9.5400 in
#6	Rectangular	Height	0.0900 in	Width	1.9200 in	0.9600 in	10.5000 in
#7	Rectangular	Height	0.0900 in	Width	1.9200 in	9.5400 in	10.5000 in

Summary

Total Area	2.2734 in2	Ixx	43.794 in4	r xx	4.3890 in
		Iyy	42.238 in4	r yy	4.3104 in
X cg Dist.	5.2500 in	Edge Distances from CG...			
Y cg Dist.	3.5426 in	+X	5.2950 in	S left	7.9769 in3
		-X	-5.2950 in	S right	7.9769 in3
		+Y	7.0024 in	S top	6.2541 in3
		-Y	-3.5876 in	S bottom	12.2072 in3

S MIN, AS EFFECTIVE IN COMPRESSION

13

Steel strength of fully effective portion of tower wall, LRFD

MTR/SEFF ≤ 0.9*50 KSI = 45 KSI

SEFF = 6.25 IN^3

For ground floor, Case 1: NR = 3, IP = 1

MTR = 9.833(K1)(IP)(W0) = 6.25*45 = 281.25 IN-K

W0 = 53.52 K

WTRA = 3.34 K < WTRS = W0/NR = 17.84 K

Anchor-based capacity governs.

Restraint Strength Based on Weld Capacity

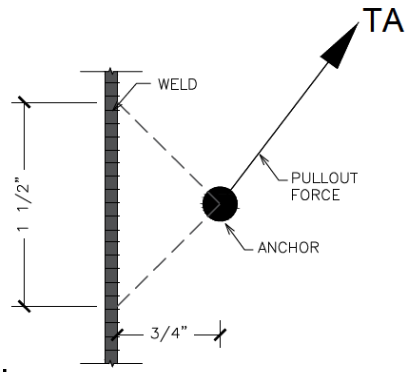
Capacity based on overall weld strength, LRFD:

AW = 2.27 IN^2

SW = 6.25 IN^3 (MIN)

TW = 0.09 IN fillet weld leg & effective throat, light-gage steel

$\frac{\sqrt{PTR^2+VTR^2}}{Aw} + \frac{MTR}{Sw} \leq 0.75*0.6*70$ KSI=31.5 KSI



For ground floor, Case 1 IP=1:

NR = 3, MTR = 9.833(K1)(IP)(W0)

PTR = 0.389(IP)(W0), VTR = 0.5344(IP)*(W0)/NR

1.03(W0) ≤ 31.5 KSI

W0 = 30.6 K

Weight tributary to each per restraint based on weld strength:

WTRW = W0/NR = 10.2 K > WTRA = 3.34 K

Anchor-based capacity governs.

Capacity based on effective weld at each anchor:

Effective weld – tension

LW = 2*0.75" = 1.5" per anchor, TW = 0.09"

NA = 2 # of anchors per side/ anchor group

N = 4 # of anchor groups

For tension force, per anchor, case 1, ground floor, IP = 1:

TA = 5.25(W0)/(7.5*N) + 0.389(W0)/(4*N) = 0.399(W0)

Fw = TA/(LW*TW) ≤ 31.5 KSI,

WWA ≤ 10.67 K

WTRA = 3.34 K < WWA = 10.67 K

Anchor-based capacity governs.

Restraint Strength Based On Baseplate Capacity

Maximum (governing) anchor force: 2

TA = 8K /2=4K (LRFD)

MPL = 4K*0.75”=3 IN-K per anchor 7

ZPL = 1.5”*TPL^2/4=0.375TPL^2

fb = MPL/ZPL ≤ 0.9*36 KSI 8

TPL ≥ 0.5”, ∴½” PLATE O.K.

Restraint Strength Based On Retaining Shaft Capacity

Based on anchor capacity, KH = KF = 1, Case 1, 2 (NR=3), ground floor:

WTR = W0/NR = 3.34K

VTR = K1*(IP)*(W0)/NR=0.5344*(1)*(3.34) = 1.79 K (LRFD)

PTR = 1.3K

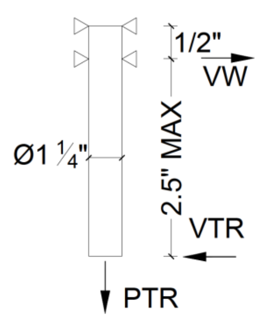
MMAX = 1.79 K * 3” = 5.4 IN-K

D =1.25” SHAFT DIAMETER

Z $=1.25^3/6 = 0.33\ IN^2$ $A = 1.23\ IN^2$

f $= 5.4\ IN\text{-}K/(0.33\ IN^3) + 1.3K/(1.23\ IN^2) = 17.4\ KSI$

<0.9 (36 KSI) = 32.4 KSI ∴O.K



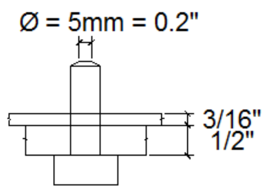
WELD

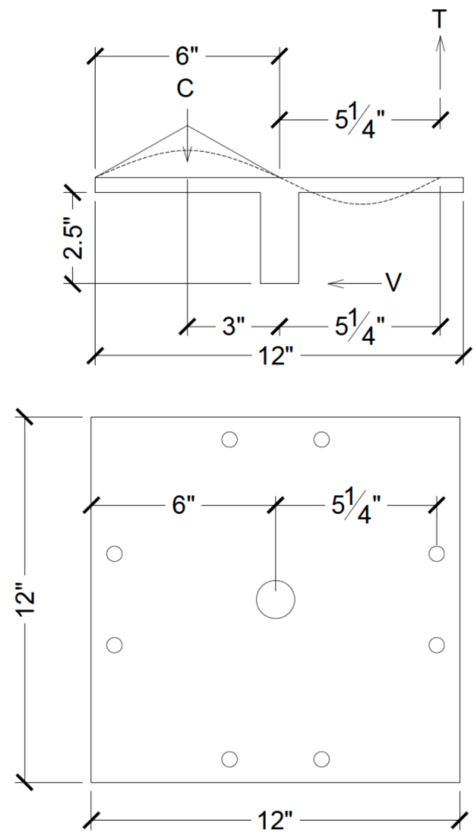
VW = 1.79K*3”/0.5” = 10.7 K MAX. REACTION AT WELD

AW $= 0.7071 * (1.25\text{”}+0.25\text{”})*3.14*0.25\text{”}=0.83\ IN^2$

Fw $= \frac{\sqrt{10.7^2+1.3^2}}{0.833}$ = 12.94 KSI < 31.5 KSI, ∴ ¼” WELD OK

Retaining Plate Design





ASD:

V = VTR/1.4 = 1.79K/1.4 = 1.27 K
PTR = 1.3K/1.4 = 0.93 K

T = 1.27K* 2.5” /(3” + 5.25”) + 0.93K/(4 SIDES) = 0.37K+0.23K=0.6 K

FS = 33 KSI (assumed)

DM = 0.2” (5mm)

L = 3T/(3.14*DM*FS) = 3*0.6K/[3.14*(0.2”)*(33KSI)] = 0.087” < 3/16”

∴ plate OK

Anchor stress

V = 1.27K/8 = 0.16 K

T = 0.6K/2 = 0.3 K (2) anchors in tension

A = 0.2^2*3.14/4 = 0.03 IN^2

f = (0.16K + 0.30K)/0.03 = 15.3 KSI – anchors OK

ECCENTRIC POSITION OF RESULTANT OF LATERAL FORCE
CAUSING TRANSLATION AND ROTATION IN THE PLANE OF THE TABLE

CASE 1

3

RESTRAINTS EFFECTIVE ***3 of 3***

EX

i			1	2	3	4
Ai		IN	28	17.09	17.09	0
$\sum AI^2$				1368		
EX		IN	20	20	20	
L		IN		6	6	
B		IN		32	32	
α	ATAN(B/2/L)	RAD	0	1.212	1.212	
		DEG		69.4	69.4	
M	E*(V0=1)	IN-#	20	20	20	
RM	M*Ai/∑Ai		0.409	0.250	0.250	
RMX	RM*SIN α		0.000	0.234	0.234	
RVX	1/3		0.000	0.000	0.000	
RX	**RMX+RVX**	#	**0.000**	**0.234**	**0.234**	
RMZ	RM*COS α		0.409	0.088	0.088	
RVZ	1/3		0.330	0.330	0.330	
RZ	RMZ+RVZ		**0.739**	**0.418**	**0.418**	
R0	**$(RX^2+RZ^2)^{0.5}$**		**0.739**	**0.479**	**0.479**	
KX	**V0/(3*R0)**		**0.45**	**0.69**	**0.69**	
KX MIN (@ ±EX)				**0.45**		

EZ

i			1	2	3	4
Ai		IN	28	17.09	17.09	0
$\sum AI^2$				1368		
EZ		IN	18	18	18	
L		IN		6	6	
B		IN		32	32	
α	ATAN(B/2/L)	RAD	0	1.212	1.212	
		DEG		69.4	69.4	
M	E*(V0=1)	IN-#	18	18	18	
RM	M*Ai/∑Ai		0.368	0.225	0.225	
RMX	RM*SIN α		0.000	0.211	0.211	
RVX	1/3		0.333	0.333	0.333	
RX	**RMX+RVX**	#	**0.333**	**0.544**	**0.544**	
RMZ	RM*COS α		0.368	0.079	0.079	
RVZ	1/3		0.000	0.000	0.000	
RZ	RMZ+RVZ		**0.368**	**0.079**	**0.079**	
R0	**$(RX^2+RZ^2)^{0.5}$**		**0.497**	**0.549**	**0.549**	
KZ	**V0/(3*R0)**		**0.66**	**0.60**	**0.60**	
KZ MIN (@ ±EZ)				**0.60**		

ECCENTRIC POSITION OF RESULTANT OF LATERAL FORCE
CAUSING TRANSLATION AND ROTATION IN THE PLANE OF THE TABLE

3

CASE 2

RESTRAINTS EFFECTIVE ***3 of 3***

EX

i			1	2	3	4
Ai		IN	112	55.36	55.36	0
$\sum AI^2$				18673		
EX		IN	67	67	67	
L		IN		53	53	
B		IN		32	32	
α	ATAN(B/2/L)	RAD	0	0.293	0.293	
		DEG		16.8	16.8	
M	E*(V0=1)	IN-#	67	67	67	
RM	M*Ai/∑Ai		0.402	0.199	0.199	
RMX	RM*SIN α		0.000	0.057	0.057	
RVX	1/3		0.000	0.000	0.000	
RX	**RMX+RVX**	#	**0.000**	**0.057**	**0.057**	
RMZ	RM*COS α		0.402	0.190	0.190	
RVZ	1/3		0.330	0.330	0.330	
RZ	RMZ+RVZ		**0.732**	**0.520**	**0.520**	
R0	**$(RX^2+RZ^2)^{0.5}$**		**0.732**	**0.523**	**0.523**	
KX	**V0/(3*R0)**		**0.45**	**0.63**	**0.63**	
KX MIN (@ ±EX)				**0.45**		

EZ

i			1	2	3	4
Ai		IN	112	55.36	55.36	0
$\sum AI^2$				18673		
EZ		IN	18	18	18	
L		IN		53	53	
B		IN		32	32	
α	ATAN(B/2/L)	RAD	0	0.293	0.293	
		DEG		16.8	16.8	
M	E*(V0=1)	IN-#	18	18	18	
RM	M*Ai/∑Ai		0.108	0.053	0.053	
RMX	RM*SIN α		0.000	0.015	0.015	
RVX	1/3		0.333	0.333	0.333	
RX	**RMX+RVX**	#	**0.333**	**0.348**	**0.348**	
RMZ	RM*COS α		0.108	0.051	0.051	
RVZ	1/3		0.000	0.000	0.000	
RZ	RMZ+RVZ		**0.108**	**0.051**	**0.051**	
R0	**$(RX^2+RZ^2)^{0.5}$**		**0.350**	**0.352**	**0.352**	
KZ	**V0/(3*R0)**		**0.94**	**0.94**	**0.94**	
KZ MIN (@ ±EZ)				**0.94**		

ECCENTRIC POSITION OF RESULTANT OF LATERAL FORCE
CAUSING TRANSLATION AND ROTATION IN THE PLANE OF THE TABLE

CASE 3

RESTRAINTS EFFECTIVE ***4 of 4***

3

EX

i			1	2	3	4
Ai		IN	112	112	16	16
$\sum AI^2$				25600		
EX		IN	67	67	67	67
L		IN		0	0	0
B		IN		32	32	32
α	ATAN(B/2/L)	RAD	0	0.000	1.570	1.57
		DEG		0.0	90.0	90.0
M	E*(V0=1)	IN-#	67	67	67	67
RM	M*Ai/∑Ai		0.293	0.293	0.042	0.042
RMX	RM*SIN α		0.000	0.000	0.042	0.042
RVX	1/4		0.000	0.000	0.000	0.000
RX	**RMX+RVX**	#	**0.000**	**0.000**	**0.042**	**0.042**
RMZ	RM*COS α		0.293	0.293	0.000	0.000
RVZ	1/4		0.250	0.250	0.250	0.250
RZ	RMZ+RVZ		**0.543**	**0.543**	**0.250**	**0.250**
R0	**$(RX^2+RZ^2)^{0.5}$**		**0.543**	**0.543**	**0.254**	**0.254**
KX	**V0/(4*R0)**		**0.46**	**0.46**	**0.99**	**0.99**
KX MIN (@ ±EX)				**0.46**		

EZ

i			1	2	3	4
Ai		IN	112	112	16	16
$\sum AI^2$				25600		
EZ		IN	18	18	18	18
L		IN		0	0	0
B		IN		32	32	32
α	ATAN(B/2/L)	RAD	0	0.000	1.570	1.570
		DEG		0.0	90.0	90.0
M	E*(V0=1)	IN-#	18	18	18	18
RM	M*Ai/∑Ai		0.079	0.079	0.011	0.011
RMX	RM*SIN α		0.000	0.000	0.011	0.011
RVX	1/4		0.250	0.250	0.250	0.250
RX	**RMX+RVX**	#	**0.250**	**0.250**	**0.261**	**0.261**
RMZ	RM*COS α		0.079	0.079	0.000	0.000
RVZ	1/4		0.000	0.000	0.000	0.000
RZ	RMZ+RVZ		**0.079**	**0.079**	**0.000**	**0.000**
R0	**$(RX^2+RZ^2)^{0.5}$**		**0.262**	**0.262**	**0.261**	**0.261**
KZ	**V0/(4*R0)**		**0.95**	**0.95**	**0.96**	**0.96**
KZ MIN (@ ±EZ)				**0.95**		