

STRUCTURAL ANALYSIS AND DESIGN REPORT

PROPOSED 1 UNIT A-FRAME HOUSE

Garcia, Sta. Monica, Surigao del Norte

Owner:

MADEL A. MALAZA

Calculations Report by:

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TIN: 276-202-839

Report date: _____

Rev: 00

I. Structural Design Criteria

1.0 Codes and Standards

3.1 Governing Codes

- 1.1.1 National Structural Code of the Philippines – NSCP 2015
- 1.1.2 American Concrete Institute – ACI 318-14
- 1.1.3 American Institute of Steel Construction – AISC 9th Edition

3.2 Governing Standard

ASTM A36	Specification for Structural Steel
ASTM A53	Standard Specification for Pipe, Steel, Black and Hot-dipped, Zinc-Coated, Welded, and Seamless
ASTM A611	Specification for Steel, Sheet, Carbon, Cold Rolled, Structural Quality
ASTM A616	Specification for Deformed and Plain Billet-steel Bars for Concrete Reinforcement
PNS 49	Steel Bars for Concrete Reinforcement Specification
ASTM C33/ PNS 49	Standard Specification for Concrete Aggregates
ASTM C39	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimen
ASTM C94/ PNS 46	Standard Specification for Ready-Mix Concrete
ASTM C150/ PNS 07	Specification for Portland Cement
PNS 16	Philippine National Standard for Concrete Hollow Blocks
SG 671	Specification for the Design of Cold-formed Steel, Structural Members by AISC

2.0 Material Specifications

2.1 Normal weight concrete 28th day compressive strength (Unless indicated otherwise on the drawings)

2.1.1	Suspended slab	21 MPa (3,000 psi)
2.1.2	Beams and Girders	21 MPa (3,000 psi)
2.1.3	Slab on grade,	21 MPa (3,000 psi)
2.1.4	Columns, Stairs,	21 MPa (3,000 psi)
2.1.3	Footings	21 MPa (3,000 psi)

2.2 Reinforcing steel yield, f_y

2.2.1	For bars 16 mm diameter and smaller	276 MPa (40,000 psi)
2.2.2	For bars 20 mm diameter and larger	414 MPa (60,000 psi)

2.3 Structural steel yield, F_y

2.3.1	For rolled shapes	250 MPa (36,000 psi)
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2.4 Masonry Concrete Compressive Strength, f_m

7.8 MPa (700 psi)

2.5 Masonry Grout Compressive Strength, f_c'

13.8 MPa (2,000 psi)

2.6 Lean Concrete 28th day compressive strength, f_c'

10.4 MPa (1,500 psi)

3.0 Loading Criteria

3.1 Dead Load, DL

Concrete	24.00 kN/m ³
Steel	77.00 kN/m ³
SDL (tiles, ceiling)	1.20 kPa

3.2 Live Load, LL

Residential Area	1.90 kPa
Stairs	2.40 kPa
Exterior Balconies	2.40 kPa

3.3 Wind Load, WL

$$q_z = 47.3 \times 10^{-6} K_z K_{zt} K_d V^2 I_w \text{ (kPa)} \quad [\text{Eq. 207-15}]$$

where

q_z =velocity pressure at mean roof height, h

K_{zt} =topographic factor

K_d =wind directionality factor

V =basic wind speed

I_w =importance factor

3.4 Seismic Load, EL

Total design base shear

$$V = C_v I W / R T$$

The total design base shear need not exceed the following:

$$V = 2.5 C_a I W / R$$

$$V = 0.8 Z N_v I W / R$$

The total design base shear shall be less than:

$$V = 0.11 C_a I W$$

where:

V = total design shear at the base of the structure

C_v = seismic coefficient as set forth in Table 208-8

I = Importance factor given in Table 208-1

W = Total dead load defined in Section 208.5.1.1

R = ductility coefficient set forth in Table 208-11 or 208-13

T = fundamental period of vibration

Z = seismic zone factor as given in Table 208-3

N_v = near source factor as set forth in Table 208-5 and 208-6

II. Construction Notes

1.0 General

- 1.1 The structural drawings shall be used in conjunction with the specifications, the architectural, mechanical, electrical and civil drawings.
- 1.2 The contractor shall verify all dimensions and conditions at the site, which shall include the location and dimensions of openings, grooves, reglets, pipe sleeves, conduits, embedded or attached to concrete, etc.
- 1.3 All dimensions are in millimeters unless otherwise noted.
- 1.4 All bar diameters and spacing are in millimeters unless otherwise noted.
- 1.5 All dimensions are in millimeters unless otherwise noted.
- 1.6 All bar diameters and spacing are in millimeters unless otherwise noted.

2.0 Concrete and Reinforcing Steel

- 2.1 Minimum cover to all reinforcing bars shall be as follows:

2.1.1 Concrete cast against and permanently
exposed to earth 75 mm

2.1.2 Formed surfaces exposed to earth or weather
Diameter 16 mm bars or smaller 40 mm
Other bars 50 mm

2.1.3 Formed surfaces not exposed directly to weather or earth
Slabs and walls 20 mm
Beams 40 mm
Columns 50 mm

- 2.2 Reinforcing bars shall be free of rust, grease or other materials likely to impair bond.

- 2.3 All reinforcing bars shall be accurately and securely placed before pouring concrete or applying mortar or grout.
- 2.4 Bar splices shall be securely wired together. Splices in reinforced concrete beams, columns and walls, shall be as shown in the details. For Non-structural walls, masonry walls and slabs, splices shall lap a minimum of 40 bar diameters and shall be staggered whenever possible.
- 2.5 Splices required in the reinforcement of beams/girders framing into columns shall not be located within the column or within a distance of twice the beam/girder depth from the face of the column.
- 2.6 Lap splices shall be provided within the center half of column height, and the splice length shall not be less than 1.3 times the required development length.
- 2.7 Contractor shall not be allowed to start placement/installation of reinforcing bars for footings, beams walls, columns, slabs, and other reinforced-concrete structural elements without submittal and approval of placing drawings. Only the structural engineer on record and/or the owner's engineer are authorized to approve placing drawings which should be submitted and received by the office of the structural engineer on record at least two (2) days prior to start of structural concrete works. Placing drawings must follow the same drawing standards as used in the working drawings of this project and only certified by the contractor's registered civil or structural engineer.
- 2.8 Definition of placing drawings: Placing drawings are working drawings for fabrication and placing of reinforcing steel. These drawing must comprise the following: bar lists, schedules, bending details, placing details, placing plans and elevations, grade, size, spacing, length of each bar, splices and their locations and any necessary additional information that must be supplied by the contractor concerning field conditions, field measurements, construction joints, and sequence of placing concrete.

3.0 Structural Steel

- 3.1 All materials and workmanship shall conform to the ninth edition of the American Institute of Steel Construction (AISC) Manual unless otherwise shown or noted.
- 3.2 Contractor shall furnish all plates, clip angles, connectors, etc. required for completion of the structure even if every such item is not shown on the contract drawings.
- 3.3 Welding shall be in accordance with the American Welding Society Code AWS D1.1 unless indicated otherwise. Welding electrodes shall be E70XX.
- 3.4 All bolts and threaded fasteners shall be ASTM A307 unless indicated otherwise.

4.0 Masonry

- 4.1 All concrete hollow blocks masonry walls shall be laid back in running bond. (interlocking course) with full mortar bedding. Stack bond shall be used only when specified.
- 4.2 All cells shall be solidly filled with concrete grout.

5.0 Foundation

- 5.1 All foundations are spread footings with tie beams.
- 5.2 Footings for CHB walls and other minor structures shall be embedded at least 600 mm from the finish grade line unless indicated otherwise.
- 5.3 All foundations should have compacted gravel course 100 mm thick or 50 mm thick lean concrete unless indicated otherwise.

6.0 Load Combinations

6.1 Steel (Design)

$U = 1.4DL$	(DSTL1)
$U = 1.2DL + 1.6LL$	(DSTL2)
$U = 1.2DL + 0.5LL + 1.3WX$	(DSTL3)
$U = 1.2DL + 0.5LL + 1.3WY$	(DSTL5)
$U = 0.9DL + 1.3WX$	(DSTL7)
$U = 0.9DL + 1.3WY$	(DSTL9)
$U = 1.2DL + 0.5LL + EX$	(DSTL11)
$U = 1.2DL + 0.5LL + -EX$	(DSTL12)
$U = 1.2DL + 0.5LL + EY$	(DSTL13)
$U = 1.2DL + 0.5LL + -EY$	(DSTL14)
$U = 0.9DL + EX$	(DSTL15)
$U = 0.9DL + -EX$	(DSTL16)
$U = 0.9DL + EY$	(DSTL17)
$U = 0.9DL + -EY$	(DSTL18)

6.2 Concrete (Design)

$U = 1.4DL$
$U = 1.2DL + 1.6LL$
$U = 1.2DL + 1.0LL + 1.6WX$
$U = 1.2DL + 1.0LL + -1.6WX$
$U = 1.2DL + 1.0LL + 1.6WY$
$U = 1.2DL + 1.0LL + -1.6WY$
$U = 1.2DL + 0.8WX$
$U = 1.2DL + -0.8WX$
$U = 1.2DL + 0.8WY$
$U = 1.2DL + -0.8WY$
$U = 0.9DL + 1.6WX$
$U = 0.9DL - 1.6WX$
$U = 0.9DL + 1.6WY$
$U = 0.9DL - 1.6WY$
$U = 1.2DL + 1.0LL + 1.0EX$
$U = 1.2DL + 1.0LL + -1.0EX$
$U = 1.2DL + 1.0LL + 1.0EY$
$U = 1.2DL + 1.0LL + -1.0EY$
$U = 0.9DL + 1.0EX$
$U = 0.9DL + -1.0EX$
$U = 0.9DL + 1.0EY$
$U = 0.9DL + -1.0EY$

6.3 Steel (Serviceability)

$$U = 1.0DL$$

$$U = 1.0DL + 1.0LL$$

$$U = 1.0DL + 0.6WX$$

$$U = 1.0DL + -0.6WX$$

$$U = 1.0DL + 0.6WY$$

$$U = 1.0DL + -0.6WY$$

$$U = 1.0DL + 0.75LL + 0.45WX$$

$$U = 1.0DL + 0.75LL + -0.45WX$$

$$U = 1.0DL + 0.75LL + 0.45WY$$

$$U = 1.0DL + 0.75LL + -0.45WY$$

$$U = 0.6DL + 0.6WX$$

$$U = 0.6DL + -0.6WX$$

$$U = 0.6DL + 0.6WY$$

$$U = 0.6DL + -0.6WY$$

$$U = 1.0DL + 0.75LL + 0.53EX$$

$$U = 1.0DL + 0.75LL + -0.53EX$$

$$U = 1.0DL + 0.75LL + 0.53EY$$

$$U = 1.0DL + 0.75LL + -0.53EY$$

$$U = 0.6DL + 0.7EX$$

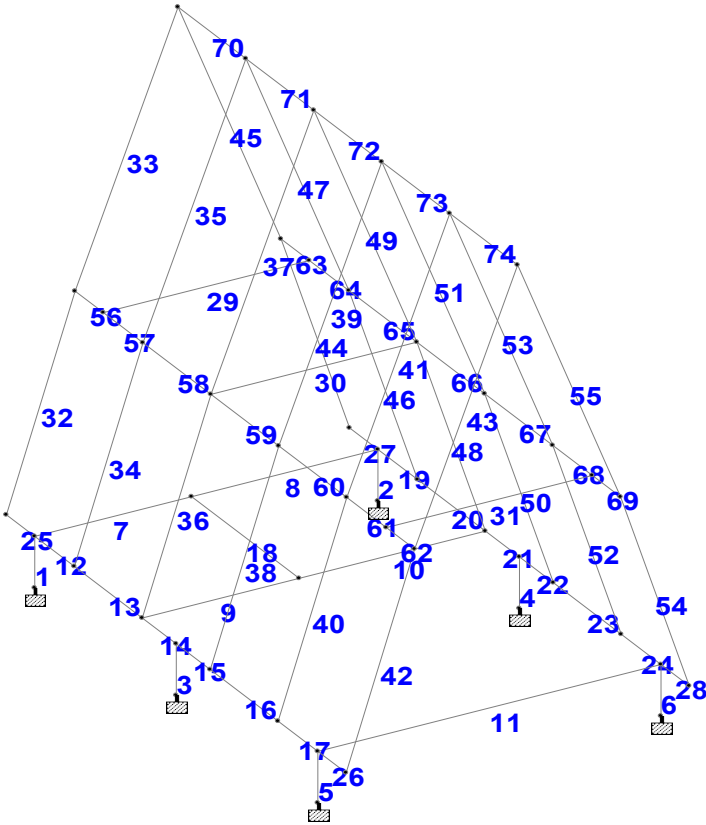
$$U = 0.6DL + -0.7EX$$

$$U = 0.6DL + 0.7EY$$

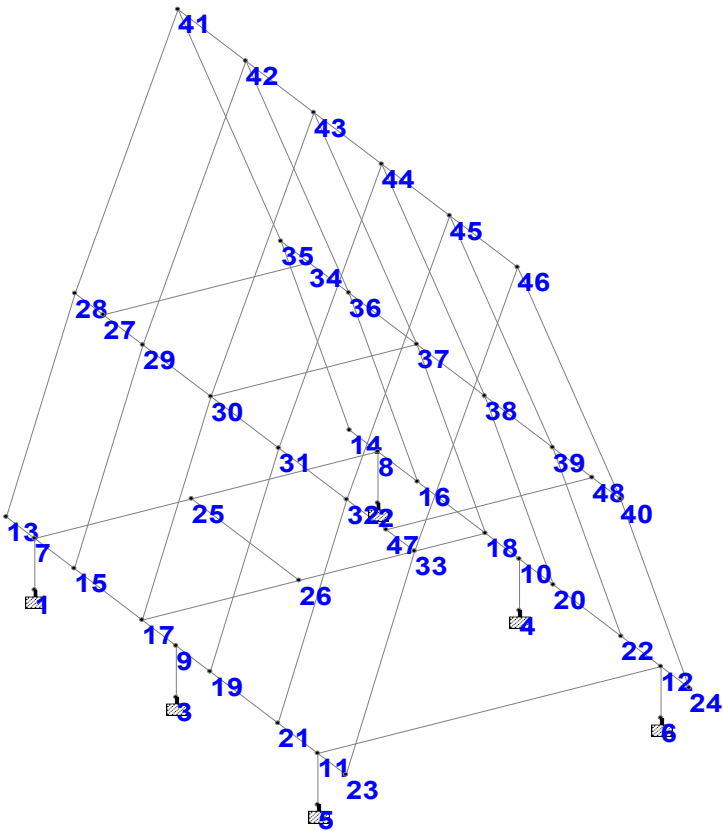
$$U = 0.6DL + -0.7EY$$

III.1 ANALYSIS MODEL : PROPOSED 1 UNIT A-FRAME HOUSE

BEAM NUMBER



NODE NUMBER



III.2 DESIGN OF STEEL MEMBERS: PROPOSED 1 UNIT A-FRAME HOUSE

III.2.1 MEMBER STRENGTH CHECK

ALL UNITS ARE - KN METE (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
=====					
25 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.976	104
		0.16 C	2.08	6.43	0.50
26 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.918	104
		0.20 C	2.07	5.85	0.00
27 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.969	103
		0.16 C	-2.07	6.39	0.50
28 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.917	103
		0.20 C	-2.07	5.85	0.00
29 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.423	102
		6.36 C	0.10	-4.00	1.05
30 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.368	103
		12.40 C	0.06	3.33	2.10
31 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.398	101
		6.74 C	-0.35	-3.30	1.05
32 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.280	104
		13.39 C	0.67	-1.02	0.00
33 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.214	103
		1.06 T	-0.50	-1.31	0.00
34 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1a	0.491	104
		22.12 C	-0.43	-2.35	0.00
35 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.165	103
		2.37 T	0.18	-1.34	0.00
36 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1a	0.702	104
		21.10 C	0.60	-4.62	0.00
37 ST	HSST6X4X0.125		(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.104	104
		4.36 C	0.10	0.56	0.00

38	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.259	103	
		6.46 C	0.02	-2.30	2.50	
39	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.234	103	
		1.51 T	0.13	-2.16	0.00	
40	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.268	103	
		9.36 C	0.20	-1.92	2.50	
41	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.257	103	
		2.79 T	-0.30	-2.06	0.00	
42	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.287	104	
		12.29 C	-0.69	-1.12	0.00	
43	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.177	103	
		1.31 T	0.39	-1.10	0.00	
44	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.276	103	
		13.30 C	-0.67	-0.98	0.00	
45	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.216	104	
		1.08 T	0.50	-1.33	0.00	
46	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1a	0.475	103	
		22.14 C	0.41	-2.17	0.00	
47	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.175	104	
		2.42 T	-0.18	-1.43	0.00	
48	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1a	0.687	103	
		23.02 C	-0.59	-4.23	0.00	
49	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.102	104	
		2.86 T	-0.09	-0.83	0.00	
50	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.267	104	
		6.36 C	-0.02	-2.39	2.50	
51	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.243	104	
		1.63 T	-0.12	-2.26	0.00	
52	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.269	104	
		9.36 C	-0.20	-1.95	2.50	
53	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.261	104	
		2.81 T	0.31	-2.09	0.00	

54	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.286	103	
		12.28 C	0.68	-1.11	0.00	
55	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.177	104	
		1.30 T	-0.39	-1.11	0.00	
56	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.510	104	
		0.43 T	-1.43	-2.74	0.50	
57	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.502	104	
		0.56 T	-1.38	-2.75	0.00	
58	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.430	103	
		0.01 C	-1.58	1.67	1.20	
59	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.498	103	
		0.06 T	-2.11	1.43	0.00	
60	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.269	103	
		0.10 C	1.25	0.59	0.00	
61	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.564	104	
		0.61 T	-1.95	-2.39	0.70	
62	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.514	104	
		0.42 T	-1.67	-2.37	0.00	
63	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.505	103	
		0.43 T	1.42	-2.71	0.50	
64	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.509	103	
		0.49 T	1.44	-2.72	0.00	
65	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.396	104	
		0.08 C	1.43	1.58	1.20	
66	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.466	104	
		0.01 C	1.97	1.35	0.00	
67	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.270	104	
		0.16 C	-1.25	0.59	0.00	
68	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.570	103	
		0.56 T	1.99	-2.38	0.70	
69	ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	Eq. H1-1b	0.512	103	
		0.41 T	1.67	-2.36	0.00	

70	ST	HSST6X4X0.125	(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.055	102
		0.57 C	-0.01	0.54	0.00
71	ST	HSST6X4X0.125	(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.037	102
		0.43 C	-0.00	-0.36	0.60
72	ST	HSST6X4X0.125	(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.109	103
		0.19 C	-0.05	1.04	1.20
73	ST	HSST6X4X0.125	(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.081	104
		0.05 C	0.02	0.80	0.00
74	ST	HSST6X4X0.125	(AISC SECTIONS)		
		PASS	Eq. H1-1b	0.032	101
		0.43 C	-0.00	0.32	1.20

III.2.2 DEFLECTION CHECK

ALL UNITS ARE - KN METE (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
=====					
25 ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	DEFLECTION	0.262	2007
		0.08	-1.46	-4.94	0.50

DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.167					
Span/Deflection : 1.14E+03 Deflection : 0.044					
L/C : 2007 LOC : 0.500					
Ratio : 0.262 (PASS)					

26 ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	DEFLECTION	0.256	2003
		0.13	0.14	0.48	0.50

DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.167					
Span/Deflection : 1.17E+03 Deflection : 0.043					
L/C : 2003 LOC : 0.500					
Ratio : 0.256 (PASS)					

27 ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	DEFLECTION	0.259	2006
		0.09	1.45	-4.90	0.50

DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.167					
Span/Deflection : 1.16E+03 Deflection : 0.043					
L/C : 2006 LOC : 0.500					
Ratio : 0.259 (PASS)					

28 ST	HSST6X4X0.125	(AISC SECTIONS)			
		PASS	DEFLECTION	0.256	2002
		0.13	-0.14	0.47	0.50

DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.167					
Span/Deflection : 1.17E+03 Deflection : 0.043					
L/C : 2002 LOC : 0.500					
Ratio : 0.256 (PASS)					

56 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.091	2007
	0.31	1.02	2.26	0.50

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 3.31E+03 Deflection : 0.036				
L/C : 2007 LOC : 0.500				
Ratio : 0.091 (PASS)				

57 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.091	2007
	0.38	-1.02	-2.27	0.00

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 3.31E+03 Deflection : 0.036				
L/C : 2007 LOC : 0.000				
Ratio : 0.091 (PASS)				

58 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.016	2002
	0.05	1.07	-1.26	0.90

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 1.83E+04 Deflection : 0.007				
L/C : 2002 LOC : 0.900				
Ratio : 0.016 (PASS)				

59 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.027	2006
	0.22	-0.98	-0.69	0.40

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 1.11E+04 Deflection : 0.011				
L/C : 2006 LOC : 0.400				
Ratio : 0.027 (PASS)				

60 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.065	2002
	0.02	-0.65	-0.87	0.60

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 4.59E+03 Deflection : 0.026				
L/C : 2002 LOC : 0.600				
Ratio : 0.065 (PASS)				

61 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.095	2003
	0.39	1.41	1.91	0.70

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 3.16E+03 Deflection : 0.038				
L/C : 2003 LOC : 0.700				
Ratio : 0.095 (PASS)				

62 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.095	2003
	0.30	-1.17	-1.90	0.00

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 3.16E+03 Deflection : 0.038				
L/C : 2003 LOC : 0.000				
Ratio : 0.095 (PASS)				

63 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.091	2006
	0.31	-1.01	2.24	0.50

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 3.29E+03 Deflection : 0.036				
L/C : 2006 LOC : 0.500				
Ratio : 0.091 (PASS)				

64 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.091	2006
	0.30	1.09	-2.24	0.00

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 3.29E+03 Deflection : 0.036				
L/C : 2006 LOC : 0.000				
Ratio : 0.091 (PASS)				

65 ST	HSST6X4X0.125	(AISC SECTIONS)		
	PASS	DEFLECTION	0.014	2003
	0.02	-0.98	-1.22	0.90

DEFLECTION CHECK: (UNIT: CM)				
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400				
Span/Deflection : 2.15E+04 Deflection : 0.006				
L/C : 2003 LOC : 0.900				
Ratio : 0.014 (PASS)				

66 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.023	2003	
	0.12	1.01	-0.56	0.30	

DEFLECTION CHECK: (UNIT: CM)					

Limit Span/Deflection (DFF)	:	300.000	Limit	:	0.400
Span/Deflection	:	1.28E+04	Deflection	:	0.009
L/C	:	2003	LOC	:	0.300
Ratio	:	0.023	(PASS)		

67 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.064	2003	
	0.01	0.62	-0.88	0.60	

DEFLECTION CHECK: (UNIT: CM)					

Limit Span/Deflection (DFF)	:	300.000	Limit	:	0.400
Span/Deflection	:	4.66E+03	Deflection	:	0.026
L/C	:	2003	LOC	:	0.600
Ratio	:	0.064	(PASS)		

68 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.095	2002	
	0.37	-1.43	1.90	0.70	

DEFLECTION CHECK: (UNIT: CM)					

Limit Span/Deflection (DFF)	:	300.000	Limit	:	0.400
Span/Deflection	:	3.14E+03	Deflection	:	0.038
L/C	:	2002	LOC	:	0.700
Ratio	:	0.095	(PASS)		

69 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.095	2002	
	0.30	1.17	-1.89	0.00	

DEFLECTION CHECK: (UNIT: CM)					

Limit Span/Deflection (DFF)	:	300.000	Limit	:	0.400
Span/Deflection	:	3.14E+03	Deflection	:	0.038
L/C	:	2002	LOC	:	0.000
Ratio	:	0.095	(PASS)		

70 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.003	2017	
	0.42	-0.00	-0.08	0.40	

DEFLECTION CHECK: (UNIT: CM)					

Limit Span/Deflection (DFF)	:	300.000	Limit	:	0.400
Span/Deflection	:	8.84E+04	Deflection	:	0.001
L/C	:	2017	LOC	:	0.400
Ratio	:	0.003	(PASS)		

71 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.012	2001	
	0.33	-0.01	0.07	0.60	

DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400					
Span/Deflection : 2.60E+04 Deflection : 0.005					
L/C : 2001 LOC : 0.600					
Ratio : 0.012 (PASS)					

72 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.006	2006	
	0.18	0.02	-0.84	0.90	

DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400					
Span/Deflection : 4.87E+04 Deflection : 0.002					
L/C : 2006 LOC : 0.900					
Ratio : 0.006 (PASS)					

73 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.004	2006	
	0.06	-0.02	0.21	0.30	

DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400					
Span/Deflection : 7.52E+04 Deflection : 0.002					
L/C : 2006 LOC : 0.300					
Ratio : 0.004 (PASS)					

74 ST	HSST6X4X0.125	(AISC SECTIONS)			
	PASS	DEFLECTION	0.001	2017	
	0.34	0.00	-0.23	0.60	

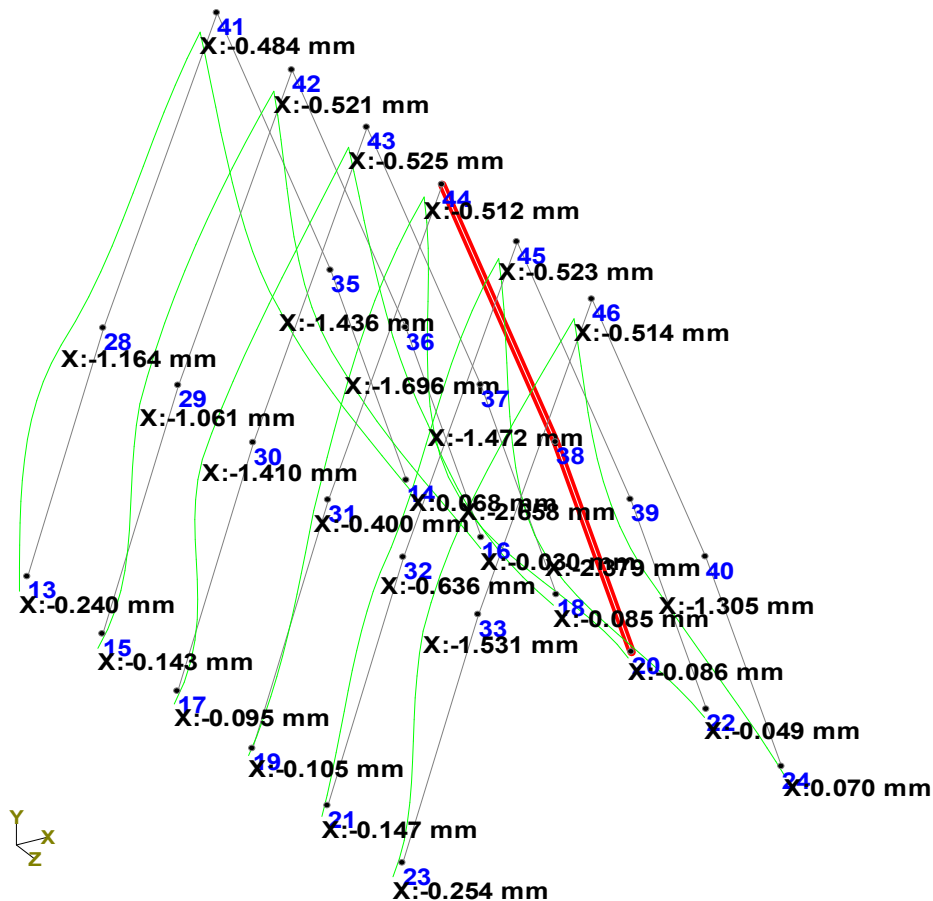
DEFLECTION CHECK: (UNIT: CM)					
Limit Span/Deflection (DFF) : 300.000 Limit : 0.400					
Span/Deflection : 2.04E+05 Deflection : 0.001					
L/C : 2017 LOC : 0.600					
Ratio : 0.001 (PASS)					

III.2.3 DISPLACEMENT CHECK PROPOSED 1 UNIT A-FRAME HOUSE

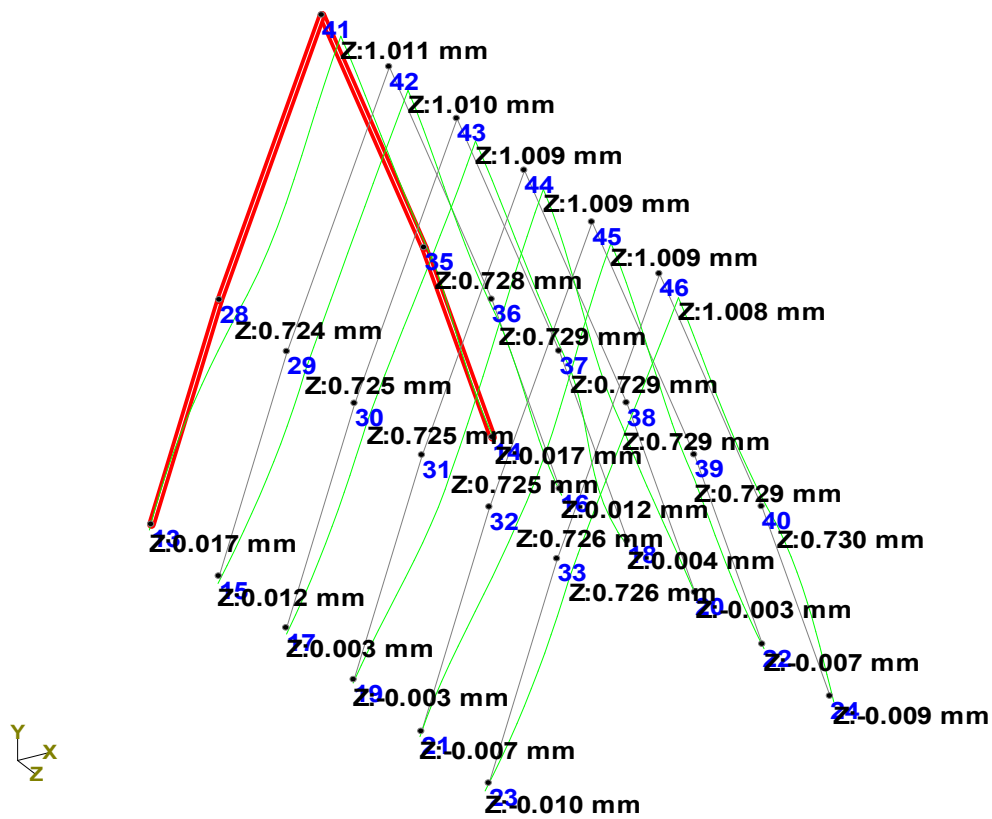
	Node	L/C	Horizontal X mm	Vertical Y mm	Horizontal Z mm	Resultant mm	Rotational rX deg	rY deg	rZ deg
Max X	31	2002 1D	2.548	-0.942	-0.002	2.716	0.007	0.041	-0.019
Min X	38	2003 1D	-2.658	-0.977	0.004	2.832	0.006	-0.037	0.021
Max Y	33	2011 0.6	-1.501	0.091	-0.007	1.503	-0.026	-0.004	-0.001
Min Y	39	2003 1D	-2.379	-1.018	0.004	2.588	0.002	0.059	0.026
Max Z	41	2020 0.6	-0.003	-0.221	1.011	1.035	0.002	0.000	0.000
Min Z	46	2021 0.6	-0.001	-0.216	-0.996	1.020	0.000	0.000	0.000
Max rX	35	2016 1D	-0.162	-0.482	0.542	0.744	0.047	0.009	0.021
Min rX	13	2017 1D	-0.195	-0.376	0.019	0.424	-0.051	0.033	-0.012
Max rY	39	2003 1D	-2.379	-1.018	0.004	2.588	0.002	0.059	0.026
Min rY	32	2002 1D	2.340	-1.005	-0.002	2.547	0.003	-0.057	-0.025
Max rZ	18	2007 1D	-0.060	-0.365	-0.003	0.370	-0.029	-0.005	0.085
Min rZ	17	2006 1D	0.074	-0.379	-0.004	0.386	-0.030	0.004	-0.090
Max Rst	38	2003 1D	-2.658	-0.977	0.004	2.832	0.006	-0.037	0.021

Member Length (L) : 5.6 m
 Max. Allow. Displacement : $L/300 = 18.667 \text{ mm}$
 Max. Displacement Z (ΔX) : 2.658 mm $\leq L/300$ **OK!**
 Max. Displacement Z (ΔZ) : 1.011 mm $\leq L/200$ **OK!**

Displacement along X



Displacement along Z



III.2.4 CONNECTION DETAIL

WELDED CONNECTION SUMMARY FOR PROPOSED A-FRAME HOUSE

Welded Connection Properties

Member Size	Member				Weld	
	dw mm	bf mm	tw mm	tf mm	S mm	a mm
HSS6X4X.125	152.4	101.6	3.048	3.048	3	2.12
HSS6X4X.125	152.4	101.6	3.048	3.048	3	2.12

1. Beam to Column

Connection	Main Member Size	Beam Member Size	Weld Group Properties						Shear Capacity		V	Axial Capacity		P	Moment Capacity (z)	Mz	Moment Capacity (y)	My	Combined Ratio (shall be <1.0)	Remarks				
			lwz mm	lwy mm	lz mm ⁴	ly mm ⁴	dy mm	dz mm	Vws	Vms		Pwa	Tm		Mwz		Mwy							
Case 1	HSS6X4X.125	HSS6X4X.125	394	597	6959114	3724932	76	51	384.20	148.05	16.14	OK	409.82	370.13	2.67	OK	15.48	7.39	OK	23.22	3.00	OK	0.256	OK

2. Beam to Beam

Connection	Main Member Size	Beam Member Size	Weld Group Properties						Shear Capacity		V		Axial Capacity		P	Moment Capacity (z)	Mz	Moment Capacity (y)	My	Combined Ratio (shall be <1.0)	Remarks			
			lwz mm	lwy mm	Iz mm ⁴	Iy mm ⁴	dy mm	dz mm	Vws	Vms			Pwa	Tm										
Case 2	HSS6X4X.125	HSS6X4X.125	203	293	6959114	3724932	76	51	232.85	128.18	8.99	OK	232.85	356.06	12.40	OK	9.55	3.33	OK	14.32	0.68	OK	0.129	OK

- Notes:
1. Refer to sample calculation for definition of variables and design procedure.
 2. Allowable stresses:
 - fsa = 345.00 Shear stress of steel
 - fyk = 450.00 Yield stress of steel
 - ftua = 450.00 Tensile stress of steel
 - fba = 450.00 Bearing stress on steel
 - ftba = 620.00 Yield stress of bolt
 - fsba = 330.00 Shear stress on bolt
 - fw = 237.00 Shear stress on weld
 - Ø = 0.90 tensile yielding
 - Ø = 0.75 tensile/shear rupture
 - Ø = 1.00 shear yielding
 - Ø = 0.75 block shear strength
 - Ø = 0.75 bearing strength

WELDED CONNECTION DESIGN PROPOSED A-FRAME HOUSE - CASE 1

Beam to Column

(1) DESIGN LOADING DATA

1-A. Design Forces (STAAD Beam End Forces)

Beam No.	Load Combination	End Forces (kN)	
14	106 1.2DL+0.5LL+1.3W(-Z)	P = 2.667	max axial force
13	102 1.2DL+1.6LL	Vy = 15.81	max shear along principal axis
13	104 1.2DL+0.5LL+1.3W(-X)	Vz = 3.129	max shear along minor axis
13	103 1.2DL+0.5LL+1.3W(+X)	Mx = 0.002	max torsional moment
13	104 1.2DL+0.5LL+1.3W(-X)	My = 3.001	max moment about minor axis
13	101 1.4DL	Mz = 7.39	max moment about principal axis

1-B. Design Code

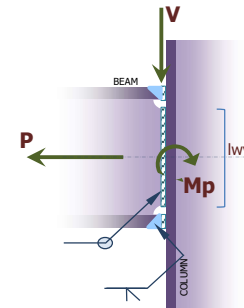
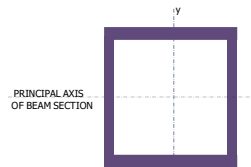
: NSCP 2015

(2) MATERIAL AND CONNECTION DATA

2-A. Steel Section

Supporting Column :		HSS6X4X.125	
D _w =	152.4	mm	depth
B _f =	101.6	mm	flange width
T _f =	3.048	mm	flange thickness
T _w =	3.048	mm	web thickness

Connecting Beam :		HSS6X4X.125
d =	152.4 mm	depth
bf =	101.6 mm	flange width
tf =	3.048 mm	flange thickness
tw =	3.048 mm	web thickness
F or f_y =	240 N/mm ²	yield strength



2-B. Weld

s =	3 mm	size of weld
a =	2.10 mm	throat of weld
l _{wz} =	394.208 mm	length of weld along z-z axis
l _{wy} =	597.408 mm	length of weld along y-y axis
F _{EXX} =	410 N/mm ²	yield strength

2-C. Reduction Factors

Y _a =	1.00	-	structural analysis factor
Y _b =	1.10	-	structural member factor
Y _i =	1.00	-	structural factor
Y _m =	1.05	-	material partial factor for steel sections

Member :		
Ø =	0.90	- tensile yielding
Ø =	1.00	- shear yielding
Ø =	1.00	- compressive strength
Ø =	0.90	- flexure strength

Weld :		
ϕ =	0.75	- shear yielding
ϕ =	0.80	- tensile yielding

2-E. Weld Group Properties

I _z =	6959114 mm ⁴	moment of inertia, I _z
I _y =	3724932 mm ⁴	moment of inertia, I _y
J =	6393632 mm ⁴	polar moment of inertia
d _y =	76 mm	y - distance from neutral axis
d _z =	50.80 mm	z - distance from neutral axis

WELDED CONNECTION DESIGN PROPOSED A-FRAME HOUSE - CASE 1

Beam to Column

(3) STRENGTH

Steel Section

f_s	=	$0.6f_y$		144.00	N/mm ²
f_y	=			240.00	N/mm ²
f_t	=			400.00	N/mm ²

Shear strength of steel

Yield strength of steel

Tensile strength of steel

Weld

τ	=	0.6 FEXX		246.00	N/mm ²
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Shear strength of weld

(4) WELD METAL CAPACITY CHECK

4-A. Weld Shear Capacity (Vws)

$$Vws = \phi (\Sigma a) \times \tau$$

$$Vws = 0.75 \times 2.1 \times (394.208 + 597.408) \times 246 / (1000)$$

$$Vws = 384.20 \text{ kN}$$

4-B. Weld Axial Capacity (Pwa)

$$Pwa = \phi (\Sigma a) \times \tau$$

$$Pwa = 0.8 \times 2.1 \times (394.208 + 597.408) \times 246 / (1000)$$

$$Pwa = 409.82 \text{ kN}$$

4-C. Weld Bending Moment Capacity at Major Axis (Mwz)

$$Mwz = \phi (J / d_y) \times \tau$$

$$Mwz = 0.75 \times (6393632.27 / 76.2) \times 246 / (1000 \times 1000)$$

$$Mwz = 15.5 \text{ kN-m}$$

4-D. Weld Bending Moment Capacity at Minor Axis (Mwm)

$$Mwy = \phi (J / d_z) \times \tau$$

$$Mwy = 0.75 \times (6393632.27 / 50.8) \times 246 / (1000 \times 1000)$$

$$Mwy = 23.2 \text{ kN-m}$$

(5) BASE METAL CAPACITY

5-A. Steel Member Shear Capacity (Vms)

$$Vms = \phi f_s \times A_w$$

$$Vms = 1 \times 144 \times [(95.504 \times 2) + 146.304] \times 3.05 / (1000)$$

$$Vms = 148.05 \text{ kN}$$

5-B. Steel Member Tensile Capacity (Tm)

$$Tm = \phi f_t \times A_t$$

$$Tm = 0.9 \times 400 \times [(95.504 \times 2) + 146.304] \times 3.05 / (1000)$$

$$Tm = 370.13 \text{ kN}$$

(6) WELD CONNECTION CAPACITY CHECK

6-A. Shear Capacity Check (Vc)

$$Vc = \min (Vws, Vms)$$

$$Vc = \min (384.2, 148.05)$$

$$V = [(Vz + Vz_{Mx})_2 + (Vy + Vy_{Mx})_2]^{0.5}$$

$$Vc = 148.05 \text{ kN}$$

$$V = 16.14 \text{ kN}$$

Requirement: $V / Vc < 1$

OK! Ratio : 0.109

6-B. Axial Capacity Check (Pc)

$$Pc = \min (Pwa, Tm)$$

$$Pc = \min (409.82, 370.13)$$

$$Pc = 370.13 \text{ kN}$$

$$P = 2.67 \text{ kN}$$

Requirement: $P / Pc < 1$

OK! Ratio : 0.007

6-C. Bending Moment Capacity

$$Mzc = Mwz$$

$$Mzc = 15.5 \text{ kN-m}$$

$$Mz = 7.39 \text{ kN-m}$$

Requirement: $Mz / Mzc < 1$

OK! Ratio : 0.477

6-D. Bending Moment Capacity

$$Myc = Mwy$$

$$Myc = 23.2 \text{ kN-m}$$

$$My = 3.00 \text{ kN-m}$$

Requirement: $My / Myc < 1$

OK! Ratio : 0.129

6-E. Safety Verification of Connection Subject to Combined Stresses

$$SV = \left\{ \left(\frac{M_y}{M_{yc}} \right)^2 + \left(\frac{M_z}{M_{zc}} \right)^2 + \left(\frac{V}{Vc} \right)^2 \right\} \leq 1.0$$

$$SV = [(7.39/15.48)^2 + (3/23.22)^2 + (16.14/148.05)^2]$$

$$SV = 0.256$$

Requirement: $SV < 1$

OK! Ratio : 0.2565

WELDED CONNECTION DESIGN PROPOSED A-FRAME HOUSE - CASE 2

Beam to Beam

(1) DESIGN LOADING DATA

1-A. Design Forces (STAAD Beam End Forces)

Beam No.	Load Combination	End Forces (kN)	
30	103 1.2DL+0.5LL+1.3W(+X)	P = 12.4	max axial force
29	102 1.2DL+1.6LL	Vy = 8.841	max shear along principal axis
31	104 1.2DL+0.5LL+1.3W(-X)	Vz = 0.192	max shear along minor axis
31	104 1.2DL+0.5LL+1.3W(-X)	Mx = 0.022	max torsional moment
31	104 1.2DL+0.5LL+1.3W(-X)	My = 0.683	max moment about minor axis
30	103 1.2DL+0.5LL+1.3W(+X)	Mz = 3.33	max moment about principal axis

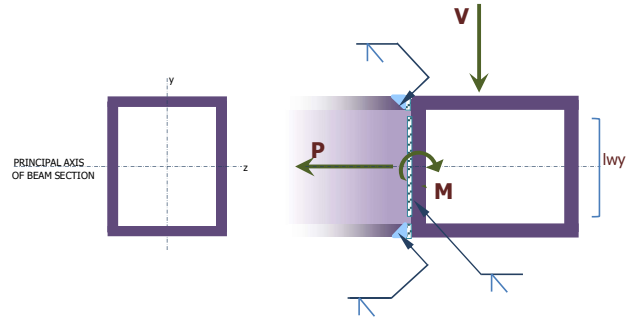
1-B. Design Code

: NSCP 2015

(2) MATERIAL AND CONNECTION DATA

2-A. Steel Section

Supporting Beam :		HSS6X4X.125
D _w =	152.4 mm	depth
B _f =	101.6 mm	flange width
T _f =	3.048 mm	flange thickness
T _w =	3.048 mm	web thickness
Connecting Beam :		HSS6X4X.125
d =	152.4 mm	depth
bf =	101.6 mm	flange width
tf =	3.048 mm	flange thickness
tw =	3.048 mm	web thickness
F or f _y =	240 N/mm ²	yield strength



2-B. Weld

s =	3 mm	size of weld
a =	2.10 mm	throat of weld
lwz =	203.2 mm	length of weld along z-z axis
lwy =	292.608 mm	length of weld along y-y axis
F _{EXX} =	410 N/mm ²	yield strength

2-C. Standard Values of Factors

Member :		
φ =	0.90	tensile yielding
φ =	1.00	shear yielding
φ =	1.00	compressive strength
φ =	0.90	flexure strength
Weld :		
φ =	0.75	shear yielding
φ =	0.80	tensile yielding

2-D. Weld Group Properties

I _z =	6959114 mm ⁴	moment of inertia, I _z
I _y =	3724932 mm ⁴	moment of inertia, I _y
J =	3942287 mm ³	polar moment of inertia
d _y =	76 mm	y - distance from neutral axis
d _z =	50.80 mm	z - distance from neutral axis

(3) ALLOWABLE STRESS

Steel Section

f _s =	0.6f _y	144.00 N/mm ²	Shear strength of steel
f _y =		240.00 N/mm ²	Yield strength of steel
f _t =		400.00 N/mm ²	Tensile strength of steel

Weld

τ =	0.6 F _{EXX}	246.00 N/mm ²	Shear strength of weld
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WELDED CONNECTION DESIGN PROPOSED A-FRAME HOUSE - CASE 2

Beam to Beam

(4) WELD METAL CAPACITY CHECK

4-A. Weld Shear Capacity (Vws)

$$\begin{aligned} Vws &= \phi (\Sigma a) \times \tau \\ Vws &= 0.75 \times 2.1 \times (203.2 + 292.608) \times 246 / (1000) \end{aligned} \quad Vws = 232.85 \text{ kN}$$

4-B. Weld Axial Capacity (Pwa)

$$\begin{aligned} Pwa &= (\Sigma a) \times \tau / (y_b) \\ Pwa &= 2.1 \times (203.2 + 292.608) \times 246 / (1.1 \times 1000) \end{aligned} \quad Pwa = 232.85 \text{ kN}$$

4-C. Weld Bending Moment Capacity at Major Axis (Mwz)

$$\begin{aligned} Mwz &= \phi (J / d_y) \times \tau \\ Mwz &= 0.75 \times (3942287.32 / 76.2) \times 246 / (1000 \times 1000) \end{aligned} \quad Mwz = 9.5 \text{ kN-m}$$

4-D. Weld Bending Moment Capacity at Minor Axis (Mwm)

$$\begin{aligned} Mwy &= \phi (J / d_z) \times \tau \\ Mwy &= 0.75 \times (3942287.32 / 50.8) \times 246 / (1000 \times 1000) \end{aligned} \quad Mwy = 14.3 \text{ kN-m}$$

(5) BASE METAL CAPACITY

5-A. Steel Member Shear Capacity (Vms)

$$\begin{aligned} Vms &= f_s \times A_w / \gamma_m (\gamma_a \gamma_b) \\ Vms &= 144 \times [(95.504 \times 2) + 146.304] \times 3.05 / (1.05 \times 1 \times 1.1 \times 1000) \end{aligned} \quad Vms = 128.18 \text{ kN}$$

5-B. Steel Member Tensile Capacity (Tm)

$$\begin{aligned} Tm &= f_t \times A_t / \gamma_m (\gamma_a \gamma_b) \\ Tm &= 400 \times [(95.504 \times 2) + 146.304] \times 3.05 / (1.05 \times 1 \times 1.1 \times 1000) \end{aligned} \quad Tm = 356.06 \text{ kN}$$

(6) WELD CONNECTION CAPACITY CHECK

6-A. Shear Capacity Check (Vc)

$$\begin{aligned} Vc &= \min (Vws, Vms) \\ Vc &= \min (232.85, 128.18) \\ V &= [(V_z + V_{z_{Mx}})^2 + (V_y + V_{y_{Mx}})^2]^{0.5} \end{aligned} \quad \begin{aligned} Vc &= 128.18 \text{ kN} \\ V &= 8.99 \text{ kN} \end{aligned}$$

Requirement: $V / Vc < 1$

OK! Ratio : 0.07

6-B. Axial Capacity Check (Pc)

$$\begin{aligned} Pc &= \min (Pwa, Tm) \\ Pc &= \min (232.85, 356.06) \end{aligned} \quad \begin{aligned} Pc &= 232.85 \text{ kN} \\ P &= 12.40 \text{ kN} \end{aligned}$$

Requirement: $P / Pc < 1$

OK! Ratio : 0.053

6-C. Bending Moment Capacity

$$\begin{aligned} Mzc &= Mwz \\ Mzc &= 9.5 \text{ kN-m} \\ Mz &= 3.33 \text{ kN-m} \end{aligned}$$

Requirement: $Mz / Mzc < 1$

OK! Ratio : 0.349

6-D. Bending Moment Capacity

$$\begin{aligned} Myc &= Mwy \\ Myc &= 14.3 \text{ kN-m} \\ My &= 0.68 \text{ kN-m} \end{aligned}$$

Requirement: $My / Myc < 1$

OK! Ratio : 0.048

6-E. Safety Verification of Connection Subject to Combined Stresses

$$\begin{aligned} SV &= \left\{ \left(\frac{M_y}{M_{yc}} \right)^2 + \left(\frac{M_z}{M_{zc}} \right)^2 + \left(\frac{V}{Vc} \right)^2 \right\} \leq 1.0 \\ SV &= [(3.33/9.55)^2 + (0.68/14.32)^2 + (8.99/128.18)^2] \end{aligned} \quad \begin{aligned} SV &= 0.129 - \end{aligned}$$

Requirement: $SV < 1$

OK! Ratio : 0.1289

PROJECT	PROPOSED 1 UNIT A-FRAME HOUSE	
OWNER/CLIENT	MADEL A. MALAZA	
ADDRESS	GARCIA, STA. MONICA, SURIGAO DEL NORTE	

SLABS				
Material Specification:			NSCP	2015
Concrete Strength, f_c' =	21	MPa	Steel Reinforcement	db, mm f_y , MPa
Concrete Weight =	Normal		Smaller than or equal to	12 276
Unit Weight =	23.6	kN/m ³	Larger than or equal to	16 414

Slab Specification:		
Slab Designation:	S-1	
Occupancy Type	Residential	
Concrete strength, f_c'	21	MPa
Rebars, f_y	276	MPa
Short span, L_a	2.95	m
Long span, L_b	3.2	m
Dead load, $s_d l$	1.2	kPa
Live load, $s_l l$	1.9	kPa
use slab thickness, t	120	mm
Main bar diameter, ϕb	10	mm
Temp bar diameter, ϕb	10	mm
Concrete cover	20	mm

Case:	TWO-WAY		
	-		
min t =	68.333	mm	OK!
reqd d =	120	mm	OK!
ϕV_c	61.674	kN	OK!
	μ_u	reqd s	use s
	kNm	mm	mm
M1, dis =	5.416	251.749	250
M2, mid =	2.753	500.026	350
M3, cont =	0.918	1509.791	350
Temp =	0	0.000	0
Temp =	1.818689536	678.453	350
Temp =	0.606229845	2046.189	350

Mark	Slab t (mm)	Short direction steel, mm			Long direction steel, mm			Type
		Main bars			Temp bars			
		"a"	"b"	"c"	"a"	"b"	"c"	
S-1	120	10 mm Ø @ 250	10 mm Ø @ 350	10 mm Ø @ 350	10 mm Ø @ 0	10 mm Ø @ 350	10 mm Ø @ 350	TWO-WAY

Slab Specification:		
Slab Designation:	2S-1	
Occupancy Type	Residential	
Concrete strength, f_c'	21	MPa
Rebars, f_y	276	MPa
Short span, L_a	1.9	m
Long span, L_b	2.1	m
Dead load, $s_d l$	1.2	kPa
Live load, $s_l l$	1.9	kPa
use slab thickness, t	120	mm
Main bar diameter, ϕb	10	mm
Temp bar diameter, ϕb	10	mm
Concrete cover	20	mm

Case:	TWO-WAY		
	-		
min t =	44.444	mm	OK!
reqd d =	120	mm	OK!
ϕV_c	61.674	kN	OK!
	μ_u	reqd s	use s
	kNm	mm	mm
M1, dis =	1.564	883.805	350
M2, mid =	0.757	1830.480	350
M3, cont =	0.252	5501.060	350
Temp =	1.052317888	1176.501	350
Temp =	0.593663616	2089.615	350
Temp =	0.197887872	6279.597	350

SLAB SCHEDULE								
Concrete, f_c' =		21 MPa			Steel f_y =		276 MPa for 10d and smaller	
					Steel f_y =		414 MPa for 12d and larger	
Mark	Slab t (mm)	Short direction steel, mm			Long direction steel, mm			Type
		Main bars			Temp bars			
		"a"	"b"	"c"	"a"	"b"	"c"	
S-1	120	10 mm Ø @ 250	10 mm Ø @ 350	10 mm Ø @ 350	10 mm Ø @ 0	10 mm Ø @ 350	10 mm Ø @ 350	TWO-WAY

PROJECT	PROPOSED 1 UNIT A-FRAME HOUSE		
OWNER/CLIENT	MADEL A. MALAZA		
ADDRESS	GARCIA, STA. MONICA, SURIGAO DEL NORTE		

BEAMS					
Material Specification:			NSCP	2015	
Concrete Strength, f_c' =	21	MPa	Coarse aggregate =	20	mm
Concrete Weight =	Normal		Steel Reinforcement	db, mm	f_y , MPa
Unit Weight =	24	kN/m ³	Smaller than or equal to	12	276
			Larger than or equal to	16	414

Beam Specification:		CASE:	SINGLY REINFORCED BEAM		
Beam Designation:	B-1		Both ends continuous		
Concrete, f_c' =	21	MPa	Slab left, d_l =	7.250	kN/m
f_y , main =	276	MPa	Slab left, l_l =	4.750	kN/m
f_y , shear =	276	MPa	Slab right, d_l =	0.000	kN/m
Span, L =	2.5	m	Slab right, l_l =	0.000	kN/m
Width, b =	175	mm	Beam l_l =	0	kPa
Depth, h =	300	mm			
Concrete cover =	40	mm			
Main bar, dbf =	16	mm			
Stirrup, dbv =	10	mm			
Stirrup legs =	2				
				Moment	Left
					Mid
					Right
				Mu (kNm)	17.467
				use n top	2
				use n bot	2
				x (m)	reqd s
					adopt s
				0.242	N/A
				0.250	N/A
				0.500	N/A
				0.750	N/A
				1.000	N/A
				1.250	N/A

MARK	SECTION	h (mm)	LOC	MAIN BARS			STIRRUPS
	b (mm)			LEFT	MID	RIGHT	
B-1	175	300	TOP	2-16 mm Ø	2-16 mm Ø	3-16 mm Ø	10Ø (2 legs) 1 @50mm, 3 @100mm, 3 @100mm, 1 @100mm, 1 @200mm O.C to CL
			BOTTOM	2-16 mm Ø	3-16 mm Ø	3-16 mm Ø	

Beam Specification:		CASE:	SINGLY REINFORCED BEAM		
Beam Designation:	B-2(1)		Both ends continuous		
Concrete, f_c' =	21	MPa	Slab left, d_l =	1.990	kN/m
f_y , main =	276	MPa	Slab left, l_l =	1.680	kN/m
f_y , shear =	276	MPa	Slab right, d_l =	3.630	kN/m
Span, L =	3.5	m	Slab right, l_l =	2.380	kN/m
Width, b =	150	mm	Beam l_l =	0	kPa
Depth, h =	300	mm			
Concrete cover =	40	mm			
Main bar, dbf =	16	mm			
Stirrup, dbv =	10	mm			
Stirrup legs =	2				
				Moment	Left
					Mid
					Right
				Mu (kNm)	21.789
				use n top	2
				use n bot	2
				x (m)	reqd s
					adopt s
				0.242	N/A
				0.350	N/A
				0.700	N/A
				1.050	N/A
				1.400	N/A
				1.750	N/A

MARK	SECTION	h (mm)	LOC	MAIN BARS			STIRRUPS
	b (mm)			LEFT	MID	RIGHT	
B-2(1)	150	300	TOP	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø	10Ø (2 legs) 1 @50mm, 4 @100mm, 1 @100mm, 1 @200mm, 1 @200mm O.C to CL
			BOTTOM	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø	

Beam Specification:		CASE:	SINGLY REINFORCED BEAM		
Beam Designation:	B-2(2)		Both ends continuous		
Concrete, f_c' =	21	MPa	Slab left, d_l =	2.320	kN/m
f_y , main =	276	MPa	Slab left, l_l =	1.520	kN/m
f_y , shear =	276	MPa	Slab right, d_l =	2.760	kN/m
Span, L =	1.9	m	Slab right, l_l =	1.810	kN/m
Width, b =	150	mm	Beam l_l =	0	kPa
Depth, h =	300	mm			
Concrete cover =	40	mm			
Main bar, dbf =	16	mm			
Stirrup, dbv =	10	mm			
Stirrup legs =	2				
				Moment	Left
					Mid
					Right
				Mu (kNm)	1.966
				use n top	2
				use n bot	2
				x (m)	reqd s
					adopt s
				0.242	N/A
				0.190	N/A
				0.380	N/A
				0.570	N/A
				0.760	N/A
				0.950	N/A

MARK	SECTION	h (mm)	LOC	MAIN BARS			STIRRUPS
	b (mm)			LEFT	MID	RIGHT	
B-2(2)	150	300	TOP	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø	10Ø (2 legs) 1 @50mm, 1 @200mm, 1 @200mm, 1 @200mm, 1 @200mm O.C to CL
			BOTTOM	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø	

SLAB SCHEDULE						
Concrete, f_c' =			Steel f_y =			
			276 MPa for 12d and smaller			
			Steel f_y =			
			414 MPa for 16d and larger			
MARK	SECTION	h (mm)	LOC	MAIN BARS		
	b (mm)			LEFT	MID	RIGHT
B-1	175	300	TOP	2-16 mm Ø	2-16 mm Ø	3-16 mm Ø
			BOTTOM	2-16 mm Ø	3-16 mm Ø	3-16 mm Ø
B-2(1)	150	300	TOP	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø
			BOTTOM	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø
B-2(2)	150	300	TOP	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø
			BOTTOM	2-16 mm Ø	2-16 mm Ø	2-16 mm Ø

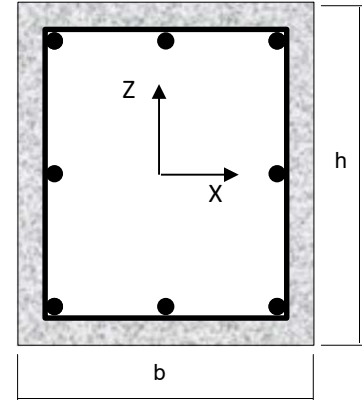
DESIGN OF RECTANGULAR CONCRETE COLUMN

Project : PROPOSED 1 UNIT A-FRAME HOUSE
 Owner/Client : MADEL A. MALAZA
 Address : GARCIA, STA. MONICA, SURIGAO DEL NORTE

Member: **C-1**

A. Required Loads

Load Case	Member No.	Axial	Shear		Bending	
		P_u	V_{ux}	V_{uz}	M_{ux}	M_{uz}
		(kN)	(kN)	(kN)	(kN-m)	(kN-m)
Max Comp.	3	90.167	-6.906	-10.696	0.984	2.145
Max Fx	1	84.877	43.422	18.469	-1.633	-4.021
Max Fz	1	84.877	-43.422	18.469	-1.633	-4.021
Max Mx	7	83.817	-43.422	18.469	9.448	22.033
Max Mz	7	83.817	-43.422	18.469	9.448	22.033
Max Tens.	-	0	0	0	0	0



B. Design Parameters

Material Properties :

Concrete Weight :	$w_c =$	24.00	kN/m ³	Design Criteria
Compressive Strength :	$f'_c =$	20.7	MPa	Design Criteria
Main Steel Bar Strength:	$f_y =$	276	MPa	Design Criteria
Sec. Steel Bar Strength:	$f_{yt} =$	276	MPa	Design Criteria

Column Dimension :

Width :	$b =$	250	mm	≥ 200 mm	
Depth :	$h =$	250	mm	≥ 200 mm	
Height :	$L =$	1800	mm		
Concrete Cover :	$C_v =$	50	mm		Design Criteria
Gross Concrete Area :	$A =$	62500	mm ²	$b * h$	

Steel Rebar :

Main Vertical Bar Diameter :	$d_b =$	16	mm	$\geq \varnothing 12$ mm	
Tie Bar Diameter :	$t_b =$	10	mm	$\geq \varnothing 10$ mm	Sect. 7.10.5.1

Bar Arrangement

Top Side Bars :	$tpb =$	3	nos.		
Bottom Side Bars :	$btb =$	3	nos.		
Left Side Bars :	$lsb =$	3	nos.		
Right Side Bars :	$rsb =$	3	nos.		
No. of Vert. Bars :	$b_n =$	8	nos.	≥ 4 nos.	Sect. 10.9.2
Steel Area :	$A_{st} =$	1608	mm ²		
Clear Spacing X-direction :	$S_{cx} =$	41	mm	$\geq \max(40\text{mm}, 1.5d_b, (4/3)d_{agg})$	Sect. 7.6.3
Clear Spacing Y-direction :	$S_{cy} =$	41	mm	$\geq \max(40\text{mm}, 1.5d_b, (4/3)d_{agg})$	Sect. 7.6.3
Steel Ratio :	$\rho =$	2.6%		$1\% \leq \rho \leq 8\%$	Sect. 10.9.1

C. Check for Slenderness

Bracing Condition :		Nonsway	
Unsupported length along X :	$L_{ux} =$	1800.0	mm
Unsupported length along Z :	$L_{uz} =$	1800.0	mm
Effective Length Factor X-dir :	$k_x =$	1.0	
Effective Length Factor Z-dir :	$k_z =$	1.0	

Project : PROPOSED 1 UNIT A-FRAME HOUSE
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Radius of Gyration at X : rx : 72.17 h / $\sqrt{12}$
 Radius of Gyration at Z : rz : 72.17 b / $\sqrt{12}$
 Slenderness Ratio at X : SLRx : 24.94 SLRx \leq 40 ---> OK Sect. 10.10.1
 Slenderness Ratio at Z : SLRz : 24.94 SLRz \leq 40 ---> OK Sect. 10.10.1

D. Check for Biaxial Capacity

Biaxial Design Equations :

Sect. 10.3.6

$$\phi P_o = \phi [0.85(f'_c)(A_g - A_{st}) + f_y(A_{st})]$$

$$\phi P_{n,max} = \phi * 0.80 * [0.85(f'_c)(A_g - A_{st}) + f_y(A_{st})]$$

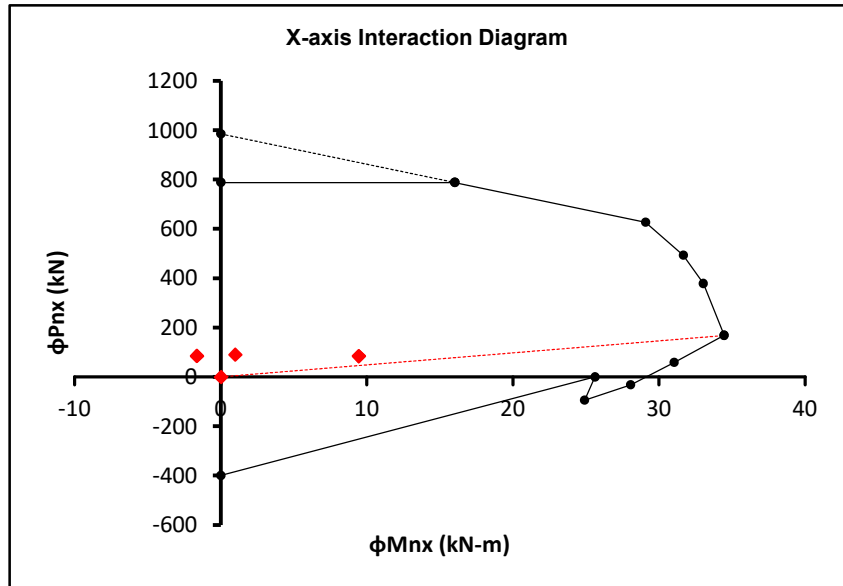
$$\phi P_n = \phi(0.85f'_c ab) + \sum_{i=1}^n \phi F_{si}$$

$$\phi M_n = \phi(0.85f'_c ab) \left(\frac{h}{2} - \frac{a}{2} \right) + \sum_{i=1}^n \phi F_{si} \left(\frac{h}{2} - d_i \right)$$

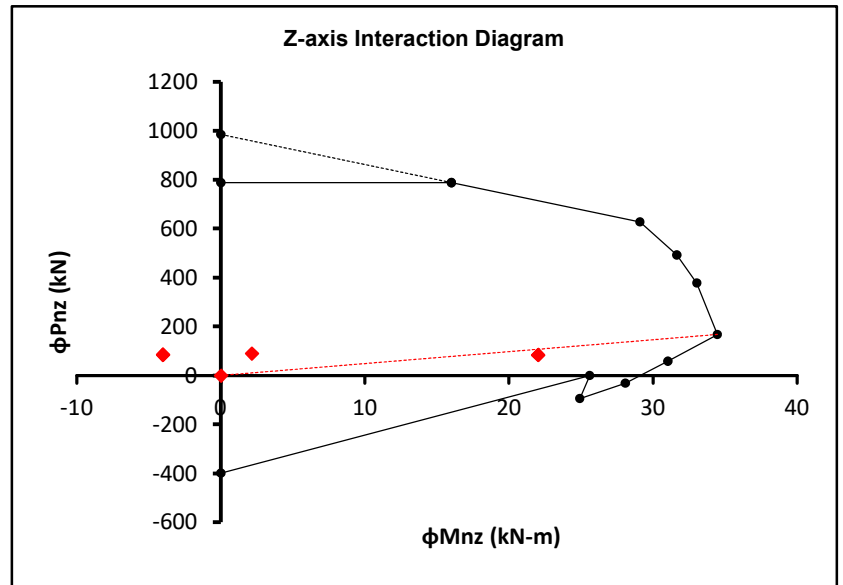
$$\phi P_{nt} = \sum_{i=1}^n -\phi f_y A_{si}$$

ϕ : As per Sect. 9.3.2

Uniaxial Capacity about X-axis			
POINT	ϕP_{nx} (kN)	ϕM_{nx} (kN-m)	e_z (mm)
ϕP_o	984.96	0.00	0.00
$0.8\phi P_o$	787.97	0.00	0.00
$0.8\phi P_o$	787.97	16.02	20.33
$0.00*\epsilon_s$	627.29	29.08	46.36
$-0.25*\epsilon_s$	492.29	31.66	64.31
$-0.50*\epsilon_s$	377.69	33.03	87.45
$-1.00*\epsilon_s$	167.73	34.46	205.47
$-1.50*\epsilon_s$	58.29	31.03	532.25
$-2.00*\epsilon_s$	-31.60	28.07	-888.17
$-2.50*\epsilon_s$	-94.65	24.91	-263.19
ϕM_u	0.00	25.61	∞
$\phi P_{nt,max}$	-399.55	0.00	0.00



Uniaxial Capacity about Z-axis			
POINT	ϕP_{nz} (kN)	ϕM_{nz} (kN-m)	e_x (mm)
ϕP_o	984.96	0.00	0.00
$0.8\phi P_o$	787.97	0.00	0.00
$0.8\phi P_o$	787.97	16.02	20.33
$0.00*\epsilon_s$	627.29	29.08	46.36
$-0.25*\epsilon_s$	492.29	31.66	64.31
$-0.50*\epsilon_s$	377.69	33.03	87.45
$-1.00*\epsilon_s$	167.73	34.46	205.47
$-1.50*\epsilon_s$	58.29	31.03	532.25
$-2.00*\epsilon_s$	-31.60	28.07	-888.17
$-2.50*\epsilon_s$	-94.65	24.91	-263.19
ϕM_u	0.00	25.61	∞
$\phi P_{nt,max}$	-399.55	0.00	0.00



Project : PROPOSED 1 UNIT A-FRAME HOUSE
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Biaxial Capacity at Design Eccentricity										
If $P_u \geq 0.1 * P_o$										
Load Case	e_{uz} (mm)	ϕP_{nx} (kN)	e_{ux} (mm)	ϕP_{nz} (kN)	ϕP_o (kN)	P_u (kN)	$\frac{P_u}{\geq 0.1 * P_o}$	ϕP_n (kN)	Ratio ($P_u / \phi P_n$)	Remarks
Max Axial	10.91	787.97	23.79	787.97	984.96	92.87	N/A	N/A	N/A	N/A
Max Mx	112.72	787.97	262.87	0.00	984.96	86.52	N/A	N/A	N/A	N/A
Max Mz	112.72	787.97	262.87	0.00	984.96	86.52	N/A	N/A	N/A	N/A
If $P_u \leq 0.1 * P_o$										
Load Case	e_{uz} (mm)	ϕM_{nx} (kN)	e_{ux} (mm)	ϕM_{nz} (kN)	M_{ux} (kN-m)	M_{uz} (kN-m)	$(M_{ux} / \phi M_{nx}) + (M_{uz} / \phi M_{nz}) \leq 1$		Remarks	
Max Axial	10.91	32.03	23.79	32.03	0.98	2.15	0.10		Pass	
Max Mx	112.72	31.83	262.87	31.83	9.45	22.03	0.99		Pass	
Max Mz	112.72	31.83	262.87	31.83	9.45	22.03	0.99		Pass	

Equations:

$$P_o = (0.85f'_c)(A_g - A_{st}) + f_y(A_{st})$$

$$1/\phi P_n = 1/\phi P_{nx} + 1/\phi P_{nz} - 1/\phi P_o$$

E. Check for Shear

Load Case	N_u (kN)	V_{ux} (kN)	V_{uz} (kN)	V_u (kN)
Max Comp.	90.17	-6.91	-10.70	12.73
Max Mz	83.82	-43.42	18.47	47.19
Max My	83.82	-43.42	18.47	47.19
Max Tens.	0.00	0.00	0.00	0.00

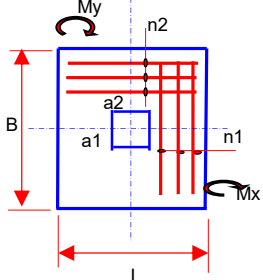
$$V_u = \sqrt{V_{ux}^2 + V_{uz}^2}$$

Strength Reduction Factor :	$\phi =$	0.75		Sect. 9.3.2
Modification Factor :	$\lambda =$	1.0	Normal weight concrete	Sect. 8.6.1
Axial Force :	$N_u =$	83.82	kN	
Required Shear Force :	$V_u =$	47.19	kN	
Concrete Shear Force :	$\phi V_c =$	28.92	kN	$\phi 0.17(1 + N_u/14A_g)\lambda\sqrt{f'_c}b_wd$ Sect. 11.2.1.2
Provided Shear Reinf. :	$A_{v,prov} =$	157	mm ²	Ø10mm 2-leg ties
Minimum Shear Reinf. :	$S_{req'd} =$	495	mm ²	$V_u > 0.5\phi V_c$ Sect. 11.4.6.1
Minimum Tie Spacing :	$S_{min} =$	37	mm	$(4/3)d_{agg}$
Maximum Tie Spacing :	$S_{max} =$	250	mm	$\min(16d_b, 48d_{tb}, \min(b, h))$ Sect. 7.10.5.2
	Use $s =$	150	mm	$s_{min} \leq s \leq s_{max} \rightarrow OK$

F. Check for Axial Tension Capacity

Strength Reduction Factor :	$\phi =$	0.9		
Pure Axial Tension Capacity :	$\phi P_{nt} =$	-399.55	kN	$-\phi * f_y * A_{st}$
Required Axial Tension :	$P_{ut} =$	0.00	kN	$\phi P_{nt} \geq P_u \rightarrow OK$

PROJECT: **PROPOSED 1 UNIT A-FRAME HOUSE**
 OWNER/CLIENT: **MADEL A. MALAZA**
 ADDRESS: **GARCIA, STA. MONICA, SURIGAO DEL NORTE**

<p>Mark: F1</p>  <p>Ultimate Soil Pressure :</p> <p>$q_{u,max} = 151.03$ kPa $q_{u,min} = 29.31$ kPa use $q_u = 90.17$ kPa $dx = 1.00$ m $dy = 1.00$ m</p> <p>Enter Bar Diameter:</p> <p>bar $\phi = 16$ mm $A_b = 201.06$ mm²</p>	<p>Design Specifications:</p> <p>$f_c' = 20.7$ Mpa $f_y = 276$ MPa $q_a = 192$ kPa</p> <p>Column Dimension:</p> <p>$a_1 = 0.25$ m $a_2 = 0.25$ m</p> <p>Depth from bottom to NGL:</p> <p>$H = 0.75$ m</p> <p>Trial Footing Thickness:</p> <p>try $t = 0.25$ m $d = 0.175$ m</p> <p>Check for Beam Shear:</p> <p>$V_u = 18.03$ kN $\phi V_c = 112.80$ kN safe for beam shear!</p> <p>Check for Punching Shear:</p> <p>$V_u = 79.90$ kN $\phi V_c = 152.27$ kN safe for punching shear!</p>	<p>Load Data:</p> <p>$P_u = 90.17$ kN $M_{u,y} = 8.43$ kN.m $M_{u,x} = 1.72$ kN.m $P = 60.11$ kN $W_{footing} = 6.01$ kN $e_{ux} = 0.093$ m $e_{uy} = 0.019$ m $q_{a,adj} = 192.00$ kPa</p> <p>Design of steel in x-dir:</p> <p>$M_u = 25.17$ kN.m $R_u = 0.91$ MPa $m = 15.686275$ $req'd p = 0.00340$ $min p = 0.00507$ $use p = 0.00453$ $A_s = 793.20$ mm² $req'd n2 = 3.95$ pcs use $n2 = 6$ pcs $s (mm) = 140.80$ OK!</p>	<p>Preliminary $A(m^2) = 0.34$ assume $L (m) = 1$ required $B (m) = 0.34$</p> <p>Trial Footing Dimension :</p> <p>try $L (m) = 1$ try $B (m) = 1$</p> <p>Check Footing Dimension:</p> <p>$q_{max} = 106.69$ kPa Footing Dimension is OK!</p> <p>Design of steel in y-dir:</p> <p>$M_u = 25.17$ kN.m $R_u = 0.91$ MPa $m = 15.686275$ $req'd p = 0.00340$ $min p = 0.00507$ $use p = 0.00453$ $A_s = 793.20$ mm² $req'd n2 = 3.95$ pcs use $n2 = 6$ pcs $s (mm) = 140.80$ OK!</p>	<p>kPa</p> <p>kN.m</p> <p>MPa</p> <p>mm²</p> <p>pcs</p> <p>pcs</p>
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