2002-0484
to
2002-0492

COUGHLINPORTERLUNDEEN
A CONSULTING STRUCTURAL AND CIVIL ENGINEERING CORPORATION

Structural Calculations

for the

Former Woodway High School
Seismic Upgrade

Integrus Architecture

5/1/02

Project S010083-01

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EXPIRES 7/25/03

Prepared by: Bryan Zagers

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COUGHLINPORTERLUNDEEN
A CONSULTING STRUCTURAL AND CIVIL ENGINEERING CORPORATION

Old Woodway High School

Structural Calculations

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General

COUGHLIN PORTER LUNDEEN
A Consulting Structural and Civil Engineering Corporation

March 13, 2002

Integrus Architecture
720 Third Avenue
Suite 2300
Seattle, WA 98104

RE: *Old Woodway High School*
Compliance with UBC Chapter 34, §3403

To Whom It May Concern:

The seismic evaluation and renovation of Old Woodway High School is being performed on a voluntary basis by the school district. It is important to note that this retrofit is not required per UBC §3401. The 1997 Uniform Building Code allows "alterations of existing structural elements, or additions of new structural elements, which are not required by Section 3401 and are initiated for the purpose of increasing the lateral-force-resisting strength or stiffness of an existing structure" provided they meet the criteria described in the exception to §3403.2. The alterations comply with Items 1-5 listed in the exception of §3403.2. All alterations to the structural elements in the construction documents are provided to increase the lateral-force-resisting strength or stiffness.

The basis of our evaluation was the Federal Emergency Management Agency document 178 (FEMA-178) NEHRP Handbook for the Seismic Evaluation of Existing Buildings. FEMA-178 is a standard evaluation methodology developed by the Building Seismic Safety Council (BSSC) for the Federal Emergency Management Agency (FEMA). FEMA-178 is the current consensus standard for life-safety evaluation of existing structures and is accepted by most building departments for substantial alteration projects. The purpose of the methodology is to provide guidance in the review of a building's response to earthquakes based on a life-safety philosophy. This document, published in June 1992, provides the most current consensus information. The federal government, as reference, specifies the use of FEMA-178 when seismic evaluations of federally owned buildings are required.

FEMA-178 recommends the use of seismic forces that are lower than the Uniform Building Code (UBC) and the code provisions proposed by the National Earthquake Hazards Reduction Program (NEHRP) for new buildings. The building can be analyzed for resistance to lower force levels in life-safety evaluations because non-life threatening damage to the building is accepted. In other words, substantial damage may be sustained by the building while still providing life-safety protection for the occupants. If the building must survive major earthquakes without sustaining severe damage in addition to providing life-safety protection, then it is analyzed using a higher force level. In general, only buildings of greater importance such as hospitals, fire stations and other essential facilities are upgraded to a higher force level.

A building does not meet the life-safety objectives of FEMA-178 if any of the following events occur during an earthquake:

- The entire building collapses.
- Any portion of the building collapses.
- The components of the building fail and fall.
- The exits and entry routes are blocked, preventing the evacuation and rescue of the occupants.

To summarize, life-safety is the primary concern; building damage and reoccupancy is the secondary concern.

The analysis methodology of FEMA-178 employs a quick check methodology (Tier 1 analysis) and a more intensive Tier 2 analysis methodology. The Tier 1 quick check employs a set of checklists for each building type. The checklist contains a set of (generally qualitative) evaluation statements which help identify areas of concern with regard to the structures' ability to adequately transmit earthquake forces to the foundation and supporting grade.

The Tier 2 analysis methodology involves numerical calculations to determine the stiffness and strength of various framing elements and connections within the structure, based on material and geometric properties. The values derived from the analysis are compared to code prescribed allowables in order to determine the "weak links" in the structural system.

FEMA-178 prescribes an effective peak acceleration coefficient of 0.2 for the Puget Sound region. In accordance with recent research in the seismicity of the region, a value of 0.3 was used for the analysis.

In general, the building will be upgraded to the FEMA-178 guidelines. FEMA-178 is a guideline and was used recognizing the economic implications of accomplishing certain upgrades. However, in no case were upgrades included or not included that would be a detriment to the existing lateral and gravity force capacity of the structures.

Sincerely,
COUGHLIN PORTER LUNDEEN, INC.

Bryan Zagers

Bryan R. Zagers

- SEISMIC WEIGHT TAKE-OFF:

  - TYPICAL ROOF

ROOFING	5 psf
2" T&G	4.3 psf
CEILING	1.7 psf
$5\frac{1}{4} \times 16\frac{1}{4}$ GL @ 8'-0" o.c.	3.0 psf
$\frac{5.25'' \times 16.25''}{144} \times 34\text{pcf} \;/ 8'$	
MISC/A/E/M	4 psf
	18 psf *

* DOES NOT INCLUDE IN-PLANE & OUT-OF-PLANE WALL WEIGHTS

- TYPICAL SEISMIC FORCE

ASSUME SHORT PERIOD, CONSERVATIVE

$$C_s = \frac{2.12(0.3)}{3.5} \quad ; \quad R = 3.5$$ (REINFORCED MASONRY SHEAR WALLS)

$$= 0.182$$

$$V_b = C_s \times W$$

$$= 0.182\,W$$

- TYPICAL OUT-OF-PLANE WALL FORCE

$$F_p = 0.67(A_v C_c W_c)$$ PER § 2.4.6

$$= 0.67(0.3)(0.9)(50\text{psf}) = 0.181 \times W_c$$

$$= 9.0 \text{ psf}$$

- DESIGN CONNECTIONS FOR TAPERED GLU-LAM

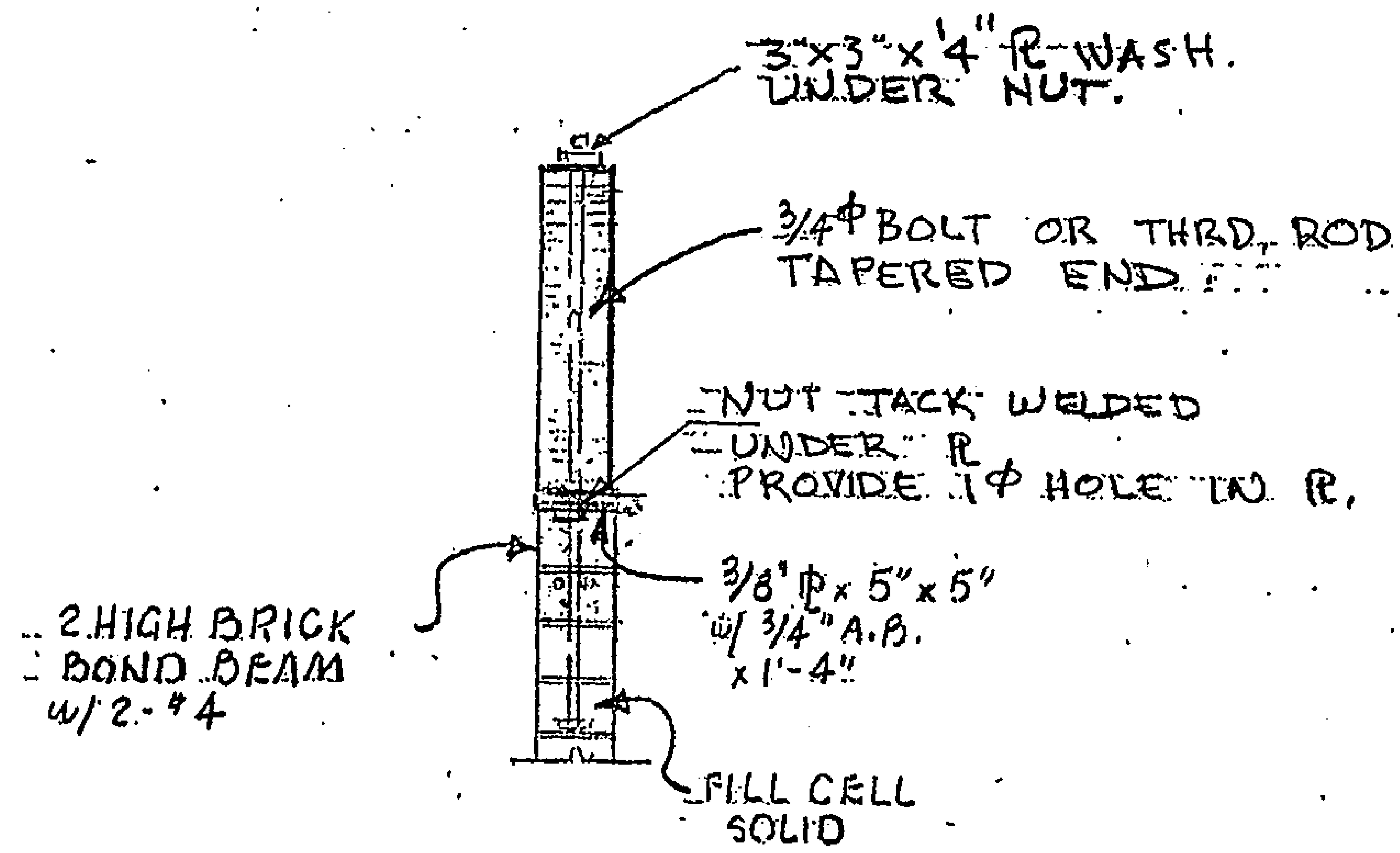
  - FORCES

    $F_p = 0.181 (50 \text{ psf} \times 6' \times 1') =$

    $= 54 \text{ plf}$ OUT-OF-PLANE

    $F = 300 \text{ plf}$ IN-PLANE - MATCH DIAPHRAGM STRENGTH

  - EX. CONNECTION CAPACITY



~~BOLTS @ 10'-0" O.C.~~

TAPERED BM. TO BRICK WALL

SCALE 1/2" = 1'-0"

  - IN-PLANE

    $Z_{\parallel} = 2760^{\#}$ PER NDS 8.2d FOR 14 1/4 EMBED (IN-PLANE)

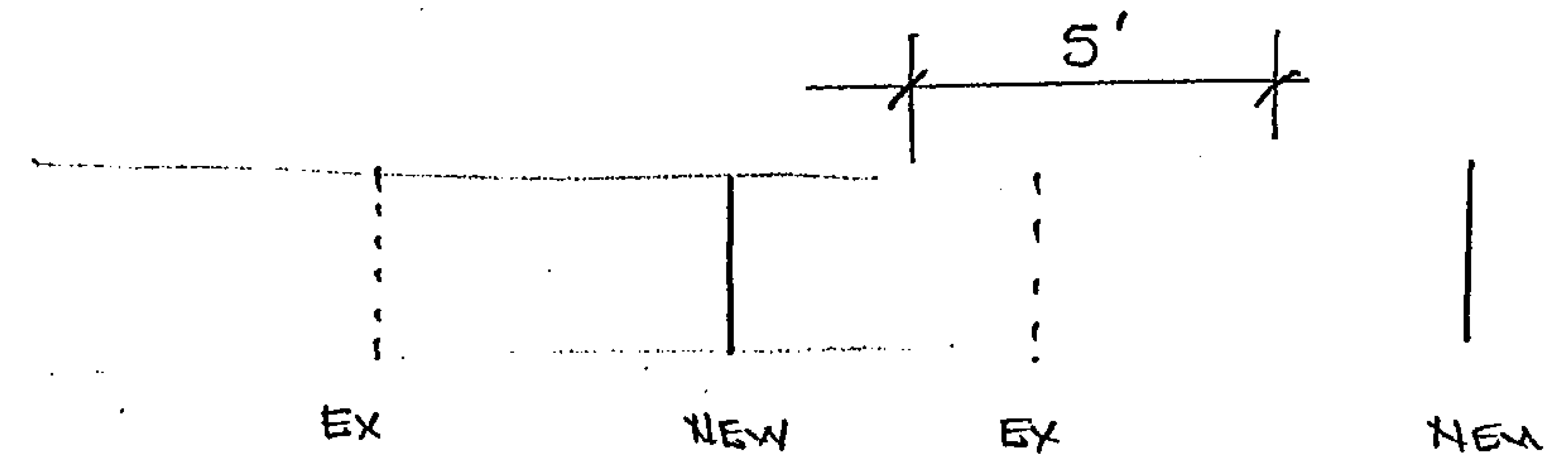
    $Z_{\perp} = 1460^{\#}$

USE ULTIMATE VALUES PER 1994 NEHRP

$\phi Z_{n_{\parallel}} = 0.85(2.16)(2.76^k) = 5.1^k$

$\phi Z_{n\perp} = 0.85(2.16)(1.46^k) = 2.7^k$

50



- FOR IN-PLANE

$\phi v_n = \frac{5100^{\#}}{10'} = 510\ plf > v = 300\ plf$ OK w/o ANY NEW CONNECTIONS.

- FOR OUT-OF-PLANE

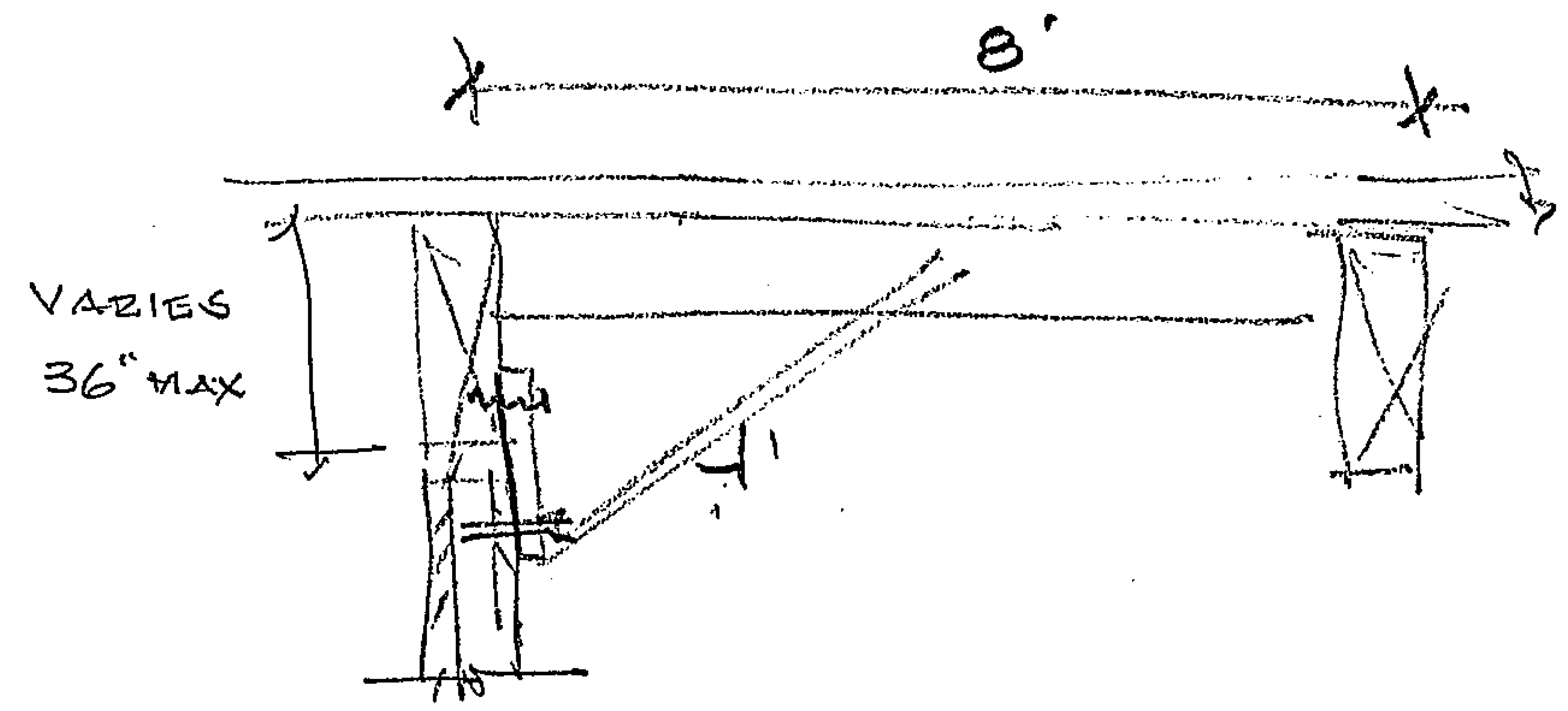
$F_p = 54\ plf \times 5' = 270^{\#} \times 6 = 1620$

$\phi Z_{n\perp} = 2.7^k$ GOOD BY FACTOR OF 10. SAY GOOD FOR SPACING REQUIREMENTS < 6'. DESIGN NEW OUT-OF-PLANE BRACES IGNORING EXISTING.

- DESIGN NEW CONNECTION

$F_p = 54\ plf \times 10' = 540^{\#}$

$F = 300\ plf \times 5' = 1500^{\#}$



- DESIGN EMBED PL

- USE (2) 5/8"ϕ RODS IN EPOXY PER ICBO ER-5279

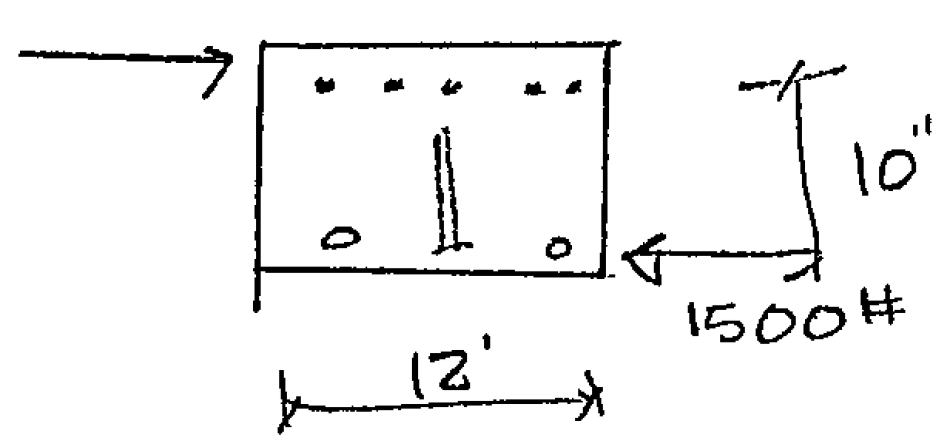
$\phi P_n = 2 \times 1.7(1.25 \times 175^{\#})$

$= 744^{\#} > 540^{\#}$ OK TENSION

assumed 5' oc.

$\phi V_n = 2 \times 1.7(1.25 \times 540^{\#}) = 2.3^k > V_u = 1500^{\#}$ OK

- USE PL 1/4"



$\phi V_n = 0.9(0.6 \times 36^{ksi})(0.25" \times 12")$

$= 58^k > 1.5^k$ OK

$M_u = \dfrac{1.5^k \times 10"}{2} = 7.5^{in\text{-}k}$

- CHECK BENDING OOP
$\uparrow 0.271^k$

$M_u = \dfrac{0.271^k \times 9"}{4} = 1.75^{in}$

$\phi M_n = 0.9(36^{ksi})(0.25")\dfrac{(12")^2}{6}$ ← USE S_x SO ELASTIC

$\phi M_n = 0.9(36)(13")\left(\dfrac{.25}{4}\right)^2 = 6.6^{in\text{-}k}$

$= 194^{in\text{-}k} >> M_u = 7.5^{in\text{-}k}$ OK

- USE (8) 1/4"ϕ SIMPSON SDS SCREWS

$Z = 315^{\#}$ /SCREW PER ICBO ER-5268

$\phi Z_n = 0.85(2.16)(315^{\#}) \times 8 = 4.63^k > 1.5^k$ OK

- DESIGN OUT-OF-PLANE BRACE

$L_c = 42'' \times 1.41$

$= 60''$

$P_u = 540^{\#} \times \dfrac{60''}{42''} = 771^{\#}$

USE ∠ 3×3×3/16 $\phi_c P_n = 20^k$ FOR $L_c = 5'$ PER LRFD TABLE FOR ∠

$> 0.77^k$ OK 3-115

- DESIGN CONN. TO EMBED

· USE PL 1/4" × 5" × 5" $\phi P_n = 0.85(0.25)(5'')(36^{ksi})$

$= 38^k >> 0.77^k$ OK

· USE 3/16 WELD TO EMBED $\phi P_u = 2 \times 5'' \times 4.18^{k}/_{''} = 42^k$ OK

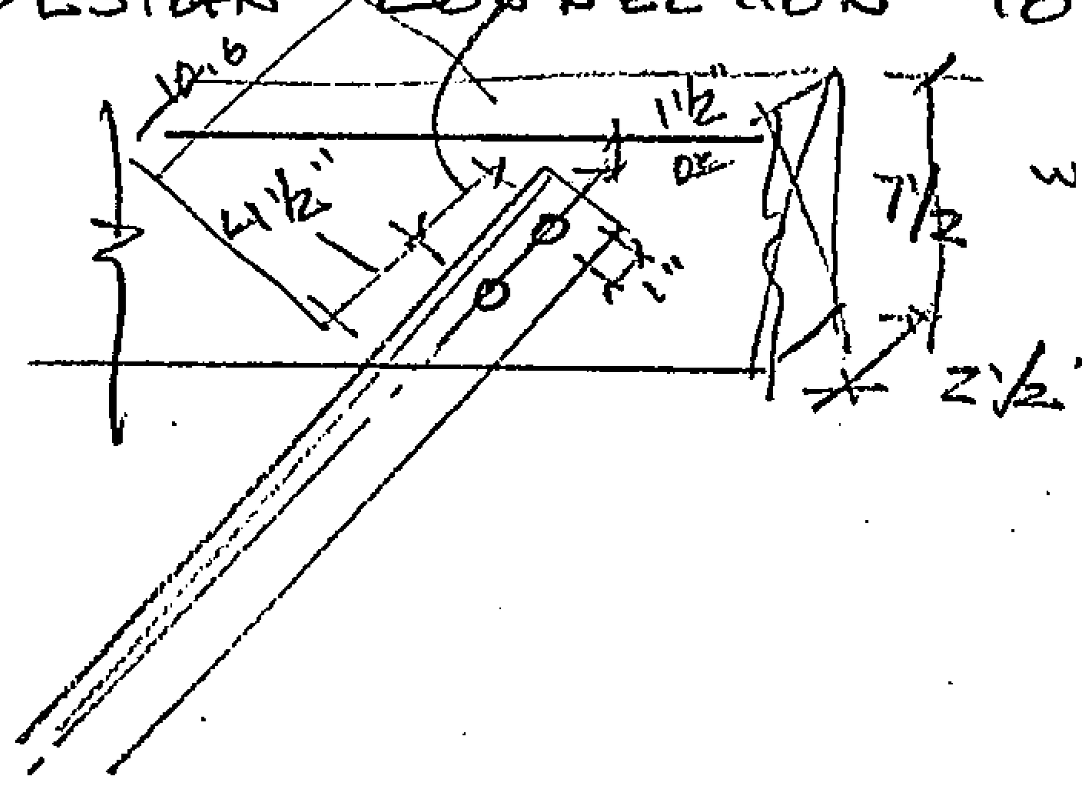
· USE (1) 3/4"ϕ SLIP-CRITICAL BOLT

$\phi P_n = 7.5^k > 0.77^k$ OK

type of wood?

- DESIGN CONNECTION TO BLOCKING

2 1/2 MIN



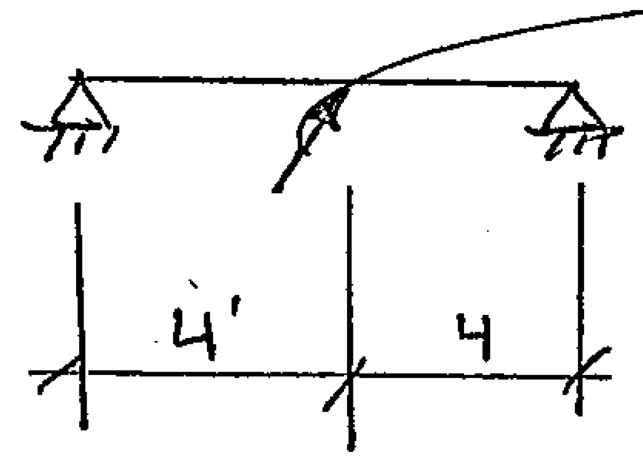
w/ (2) 5/8"ϕ THRU-BOLTS

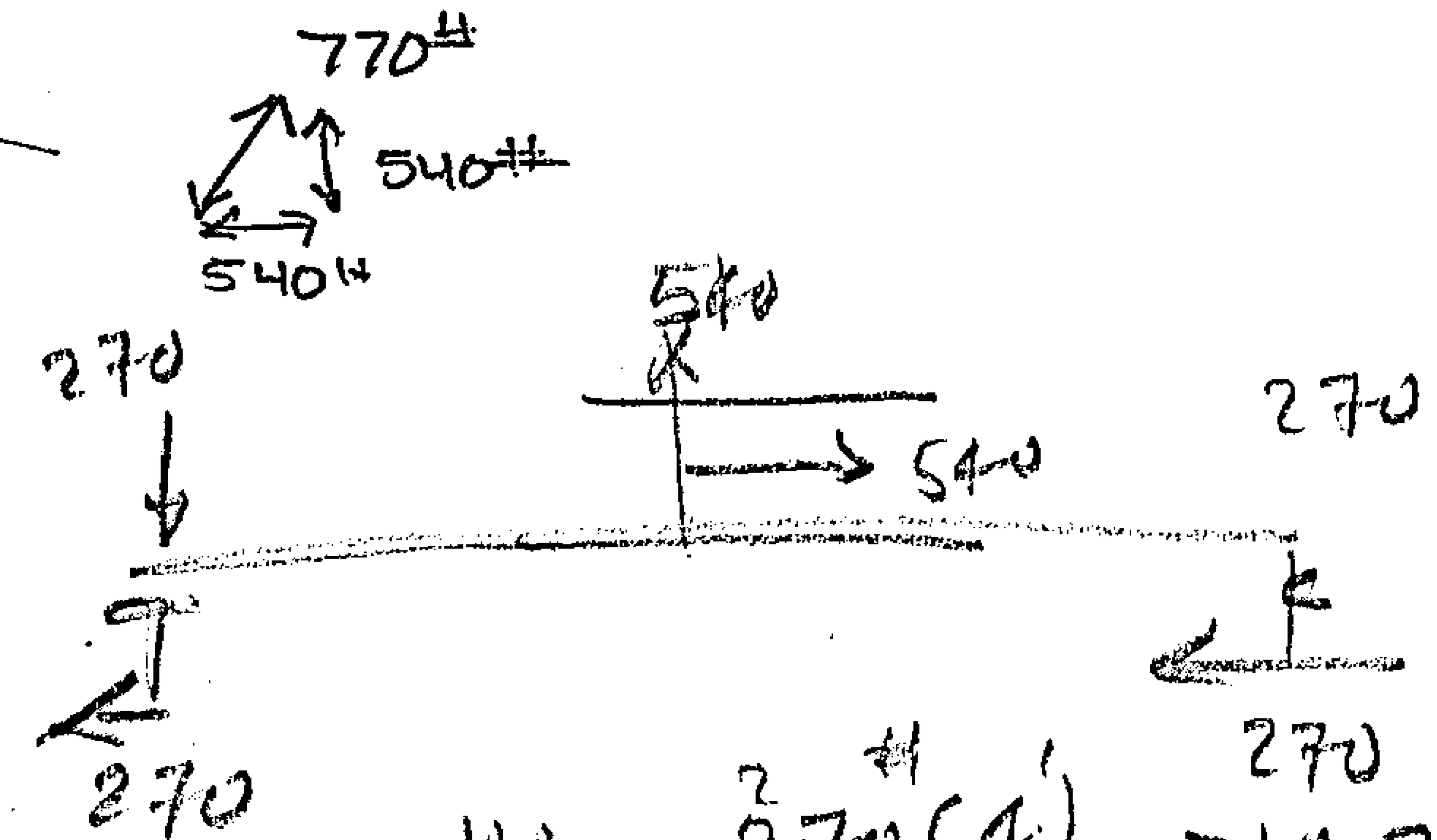
ASSUME ⊥ TO GRAIN FOR 2 1/2" MAIN MEMB

$\phi P_n = 2 \times 0.85(2.16)(440^{\#})$

$= 1.6^k > 0.77^k$

• DESIGN BLOCKING





$M_u = \frac{540^{\#} \times 8'}{4}$

$= 1.08^{ft\text{-}k}$

$M = 270^{\#}(4') = 1080^{\#'}$ ok

USE 3x8 $M_{ALLOW} = 2.48^{ft\text{-}k} > 1.08^{ft\text{-}k}$ OK
(ALLOW) (ULT)

$V_{ALLOW} = 1.21^{k}$ OK

• DESIGN HORIZ. CLIPS

USE (3) A35 CLIPS EA. SIDE

Each side? detail 12/S4.1

$\phi P_n = (3)(0.85 \times 2.16) \times 450^{\#}$ $\times 2$

$= \cancel{2355}'' > 540^{\#}$ OK

USE (1) A35 VERT EA. END for uplift

$\phi P_n = 0.85 \times 2.16 \times 450^{\#} = 826^{\#} > \frac{540''}{2} = 270^{\#}$

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- DESIGN TYPICAL CONNECTION FOR WALLS $\perp$ TO PURLINS:

  - FORCES (SEE CALC FOR TAPERED GL CONNECTION):

    $F_{IN-PLANE} = 300\,plf$

    $F_{OUT-OF-PLANE} = 54\,plf$

  - DESIGN CONNECTION FOR OUT-OF-PLANE

    USE EX. GL BEAMS @ 8' o.c.

    $F = 54\,plf \times 8' = 432^{\#}$ < CONNECTION FOR TAPERED GL

    USE SAME CONN. FOR ELEMENTS IN COMMON

  - USE (2) CONNECTION TYPES

    ∴ ∠ BRACE SIM TO TAPERED GL CONN — 8/A5.1 ? 9/S4.1

    ∴ STRAP TIE 13/ used on 2/S4.1 ?

    USE LTTT20B $\quad P_{ALLOW} = 12220^{\#}$ w (2) 1/2"φ BOLTS

    $\phi P_n = 0.85(2.16)(12220^{\#}) = 22.4^{k}$ OK

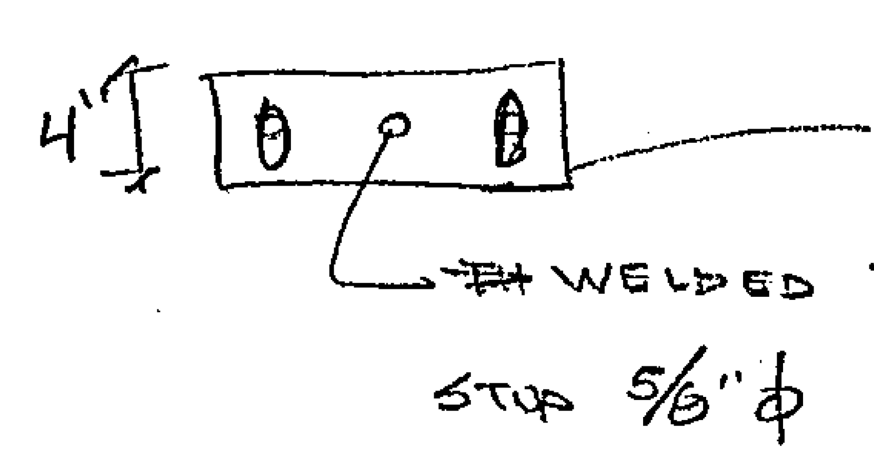
    USE (1) 5/8"φ ROD

    $\phi_t P_n = \cancel{\phi}\ 175^{\#} \times 1.7 \times 1.25$

    $= 372^{\#}$ NO GOOD

    ∴ USE (2) 5/8"φ RODS

    $\phi_t P_n = 2 \times 372^{\#} = 744^{\#}$ OK



$M_u = \dfrac{0.432^{k} \times 9''}{4} = 0.972^{in\text{-}k}$

$\phi M_n = 0.9(36^{ksi})(4'')\dfrac{(0.25'')^2}{4}$

$= 2.03^{in\text{-}k}$ OK

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- CONNECTION OF BLOCKING TO GL

  $P_u = 432^{\#}$

  USE (2) A35F EA. SIDE

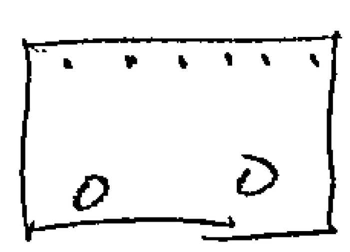
  $\phi P_n = 4 \times 450^{\#} \times 0.85 \times 2.16$

  $= 3.31^{k} > P_u = 432^{\#}$ OK

- DESIGN CONNECTION FOR IN-PLANE

  $P_u = 24' \times 300\,plf$ ∴ ASSUME 24' OF ~~TRUSS~~ STRUT FOR 8' SECTION OF WALL

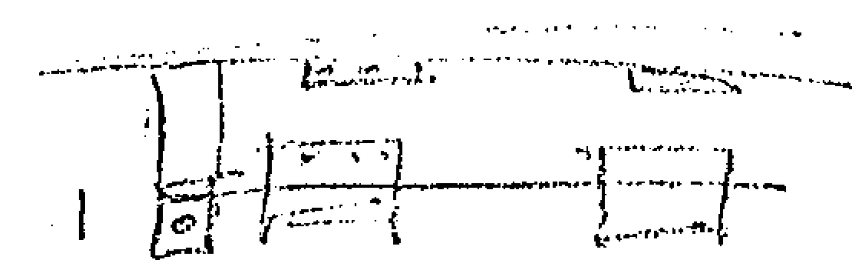
  $= 7.2^{k}$



w/ (2) 5/8"ϕ THREADED RODS - EPOXY

$\phi V_n = 2 \times (1.7 \times 1.25 \times 540^{\#}) = 2.3^{k}$

∴ USE (2) PL's EACH BAY (4'-0" OC)



- ~~DESI~~ USE (8) 1/4"ϕ SDS SCREWS TO GL

  $\phi Z_n = 4.63^{k} > 2.3^{k}$ OK

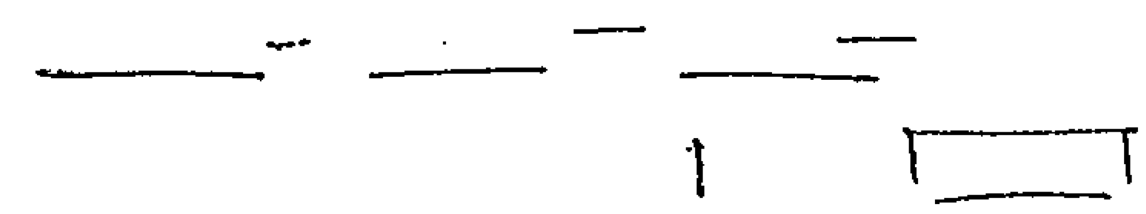
  SO 2.3^{k}/PLATE $\cong 8' \times 300\,plf = 2.4^{k}$

DIAPHRAGM SHEAR CAPACITY, SO @ ~~WALLS~~ LOCATIONS WHERE WALL OCCURS EA. BAY ONLY (1) PL IS REQUIRED. USE (2) PL @ OTHER LOCATIONS WHERE LOAD IS DRAGGED IN.

- DESIGN TIES FOR GL-FILLERS
  - AS STRUT TO CONNECT (2) BAYS

$2 \times 8' \times 300\text{plf}$

$= 4.8^k$



USE HSA32 $\phi R_n = 1.7 \times 1.9^k = 3.23^k$ SAT OK

USE 4D2A $\phi R_n = 1.7 \times 3.7^k = 6.3^k$ SAT OK

- DESIGN TYPICAL OUT-OF-PLANE WALL BRACING (INTERIOR WALLS) (FOR WALLS NOT STRONGBACKED)
  - FORCES

    $F_p = 0.181 \times 50\text{psf}$

    $= 9.1\text{ psf}$

    ~~FOR 10' WALLS,~~

- DESIGN CONNECTION FOR 10' WALLS PARALLEL TO BEAMS
  - WHERE WALL EXTENDS TO SHEATHING

    SPACE @ 6' OC

    $F_p = 6' \times 5' \times 9.1\text{ psf} = 273^{\#}$

    USE LLT 20B

    $\phi P_n = 1.7 \times 1750^{\#} = 3^{k}$

    IN GENERAL, ANCHORAGE IS CONTROLLED BY EPOXY BOLT CAPACITY, ASSUME HOLLOW CMU.

    $\phi_t P_n = 1.7(1.25 \times 175^{\#}) = 372^{\#}$ OK

  - WHERE WALL DOES NOT EXTEND TO SHEATHING
    - USE CONN. SIM TO TAPERED GL BM (SEE OTHER CALC)

      $\phi P_n = 744^{\#}$ (LIMITED BY (2) EPOXY BOLTS)

  - WHERE WALL IS ⊥ TO GL BEAMS
    - SPACE @ 8' OC & USE LTT SIM TO CONNECTION FOR WALLS ⊥ TO BEMS EXT.

      $\phi_t P_n = 744^{\#}$ (LIMITED BY EPOXY)

      $P_u = 8' \times 5' \times 9.1\text{ psf} = 364^{\#}$ OK

Project: OLD WOODWAY | Designed By: BRZ Date
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- DESIGN CONNECTIONS FOR 20' WALLS (CONT'D)
  - WHERE WALL EXTENDS TO SHEATHING
    SPACE @ 4'-0" oc
    $F = 4' \times 10' \times 9.1 \text{ psf} = 364\#$
    - SEE CALCS FOR 10' WALL
      $\phi_t P_n = 372\# > P_u = 364\#$ OK
  - WHERE WALL STOPS BELOW SHEATHING
    - SEE CALCS FOR 10' WALLS, SPACE @ 4' oc
      $\phi_t P_n = 744\#$ OK
  - WHERE WALL IS $\perp$
    $\phi_t P_n = 744\# > 364\#$ OK

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Unit A

UNIT A

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns, steel beams are supported by steel columns.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity ($A_v = 0.4$). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1) FIELD VERIFY CONNECTION OF WALL TO GLU-LAM FILLER IN E/W DIRECTION & TAPERED GL IN N/S DIRECTION

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F N/A WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F N/A SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

UNIT A

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

SHEAR WALLS

15psi

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Sec. 5.3.1)

FIELD VERIFY

T F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of the walls. (Sec. 5.3.2)

FIELD VERIFY

T F REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Sec. 5.3.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry walls are no more than *8 feet long. (Sec. 7.1.6)

T F OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than *25 percent of the wall length, and the available length appears sufficient. (Sec. 7.1.4)

SATISFIES DETAILED CHECK → T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1) STRAIGHT SHEATHING.

T F CROSS TIES: There are continuous cross ties between diaphragm chords. (Sec. 7.1.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than *40 feet and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.2.3)

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than *40 feet in areas of high seismicity (A_v greater than or equal to 0.2) and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.3.1)

N/A

UNIT A

CONNECTIONS

T F N/A WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

? T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

? T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

? T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

- UNIT A SHEARING STRESS QUICK CHECK PER § 5.3.1

  - SEISMIC WEIGHT

    6" CMU WALL 50 psf × 5' × 114' — 28.5^k

    ROOF 18 psf × 68' × 62' — 69.2^k

    98^k

  - BASE SHEAR

    $V_b = 0.182 \times 98^k$

    $= 17.8^k$

  - CHECK WALLS IN N/S DIRECTION

    $L_w = 64'$

    $v_{avg} = \dfrac{17.8^k}{64' \times 12"/' \times 6"}$

    $= 3.9$ psi < 15 psi OK

  - CHECK WALLS IN E/W DIRECTION

    $L_w = 50'$

    $v_{avg} = \dfrac{17.8^k}{50' \times 12"/' \times 6"}$

    $= 4.9$ psi < 15 psi OK

· UNIT A DIAPHRAGM CHECK PER § 7.2.1

$$DCR = \frac{2.5(0.3)\left(69.2^k + \frac{28.5^k}{2}\right)}{2 \times 300plf \times 48'}$$

$$= 2.17$$

w/ 64' SPAN, MAX DCR = 4.5 PER FIGURE C7.4.4.1 OK

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Unit B

UNIT B

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns, steel beams are supported by steel columns.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Sec. 5.3.1)

T F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of the walls. (Sec. 5.3.2)

T F REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Sec. 5.3.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry walls are no more than *8 feet long. (Sec. 7.1.6)

T F OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than *25 percent of the wall length, and the available length appears sufficient. (Sec. 7.1.4)

SATISFIES DETAILED CHECK

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

T F CROSS TIES: There are continuous cross ties between diaphragm chords. (Sec. 7.1.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than *40 feet and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.2.3)

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than *40 feet in areas of high seismicity (A_v greater than or equal to 0.2) and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.3.1)

N/A

Unit B

CONNECTIONS

T N/A F WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

- UNIT B SHEAR STRESS CHECK
  - SEISMIC WEIGHT:

6" CMU WALL	$50psf \times 6' \times 260'$	78^k
ROOF	$18psf \times 92' \times 164'$	272^k
		350^k

  - BASE SHEAR:

$V_b = 0.182 \times 350^k$

$= 63.7^k$

- CHECK WALLS IN N/S DIRECTION

$L_w = 112'$

$v_{avg} = \dfrac{63.7^k}{112' \times 12''/' \times 6''}$

$= 7.9\ psi < 15 psi$ OK

- WALLS IN E/W DIRECTION GOOD BY INSPECTION
- CHECK DIAPHRAGM IN E/W DIRECTION)

$DCR = \dfrac{2.5(0.3)(272^k + 30^k)}{2 \times 300 plf \times 74'}$ w

$= 5.1$

SPAN = 160', MAX DCR = 3.1

$2 \times 300plf \times 74' + 200plf \times 100'$

$\dfrac{226.5^k}{64.4} = 3.52$

$= 3.5$ w/ CROSSWALLS

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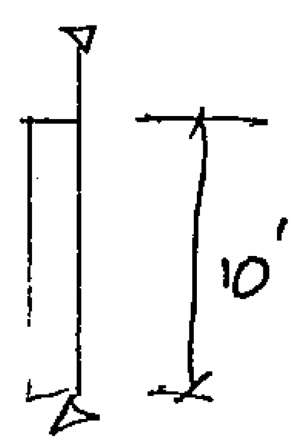
- DESIGN STRONGBACK FOR WALL @ N & S END OF UNIT B:

  - FORCES:

    $F_p = 0.181 \times 50_{psf}$

    $= 9.1_{psf}$

  - SO FOR TS @ 8' OC



$F_p = 8' \times 9.1_{psf} = 72.8_{plf}$

SAY 12', $M_u = 1/8\,(73_{plf})\,(12')^2$

$= 1.31^{ft\text{-}k}$

$$Z_x \geq \frac{1.31^{ft\text{-}k} \times 12''/'}{0.9 \times 46^{ksi}}$$

$\geq 0.38\ in^3$

  - DEFLECTION

    $\Delta_{MAX} = \frac{1}{10} \times 6'' = 0.6''$ PER FEMA-178 § 8.4.2

    $\frac{12' \times 12''}{600} = 0.240$ USE $L/360$

    $$I_{360} = \frac{wL^3}{43} = \frac{0.073 \times 12'^3}{43}$$

    $= 2.93\ in^4$

    USE TS 4×4×1/4 $Z_x = 4.97\ in^3$
    $I_x = 8.22\ in^4$

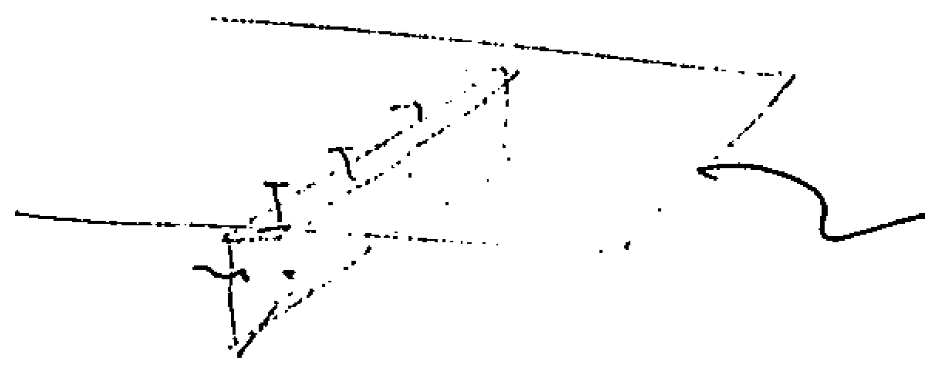
  - BASE CONN.

    $R = 6' \times 73_{plf} = 438^{\#}$ USE (2) 5/8"φ EPOXIED GBI

  - TOP

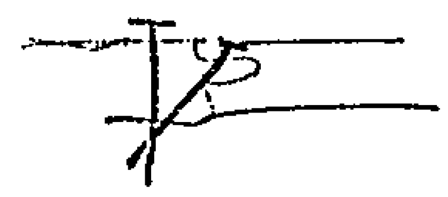
- DESIGN TOP CONNECTION

$R_u = 73 plf \times 6' = 439^{\#}$



EX 2x6 T&G NAILED TO TAPERED GL w/ (2) 20d EACH BOARD

- ~~USE~~ USE (4) #8 SCREWS TO GL BEAM @ 6" OC



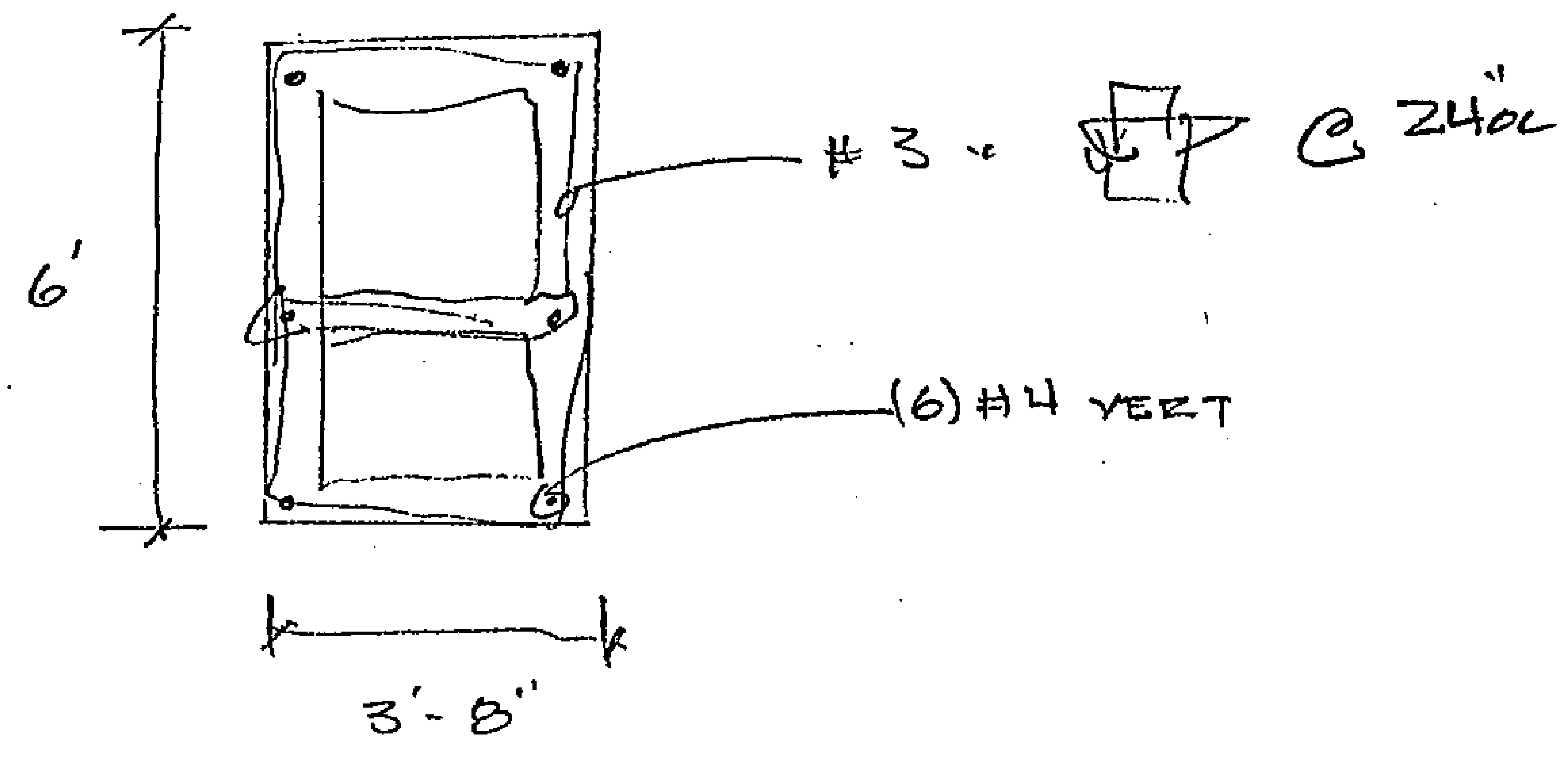
$\phi R_n = 4 \times 117^{\#} \times (0.85 \times 2.16)$
$= 859^{\#}$

CHECK EX 20d TO TRANSFER LOAD TO DIAPH.

$(4 \times 115^{\#} + 4 \times 92^{\#}) \times 0.85 \times 2.16 = 830^{\#}$ OK

USE (4) # 8 SCREWS

• Check ex. Chimney 26' tall



• Forces

$F_p = 0.67 (A_v C_c W_c)$ ≡ 6 2-10 § 24.6

; $A_v = 0.3$

$C_c = 2.4$

$W_c = 50\,psf \times (6' + 6') + 3' + 3' + 3'$

$= 1050\ plf$

$= 0.67 (0.3)(2.4)(1050\ plf)$

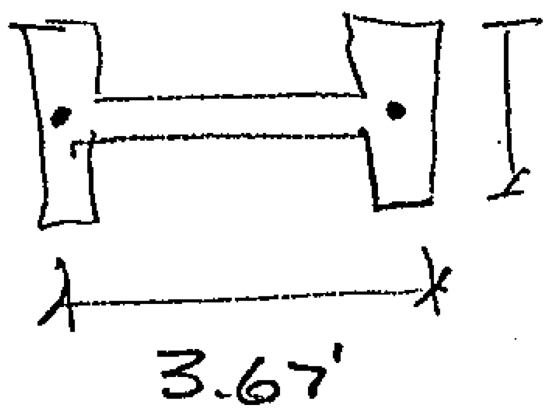
$= 507\ plf$

• Check short direction P_D

$V_u = 507\ plf \times 26' = 13.2^k$

$M_u = 507\ plf \times \frac{26'^2}{2} = 171\ ft\text{-}k$

so $M_{u_{PIER}} = 57\ ft\text{-}k$



$T_s = 0.2 \times 40^{ksi}$

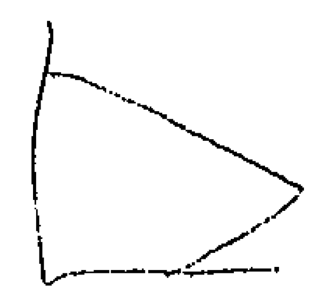
$= 8^k$

$a = \dfrac{8^k}{0.85 \times 1.5^{ksi} \times 24"} = 0.261"$

$\phi M_n = 0.8(8^k)\left(\dfrac{44" - 0.261"}{2}\right)$

$= 281^{in\text{-}k}$

$= 23.4^{ft\text{-}k}$ (IGNORING COMP STL.



$P_D = \cancel{8}\ \dfrac{1050\,plf \times 26'}{3}$

$= 9.1^k$ $\qquad \phi M_n = 38.5^{ft\text{-}k}$ w/ AXIAL LOAD INCLUDED

ADD $\angle$ 6x6x5/16 EA. CORNER, $A_s = 3.65\,in^2$

LOOKING @ WHOLE CHIMNEY → SO $T_s = 0.2 \times 40^{ksi} \times 3 + 2 \times 3.65\,in^2 \times 36^{ksi}$

$= 287^k$

$a = \dfrac{287^k}{0.85 \times 1.5^{ksi} \times 72"} = 3.12\,in^2$

$\phi M_n = 0.8(287)\left(44" - \dfrac{3.12}{2}\right)$

$= 9744^{in\text{-}k}$

$= 812^{ft\text{-}k} \gg M_u = 57^{ft\text{-}k}$

- DETERMINE CUTOFF POINT $\qquad M_u = \dfrac{507}{3}\,plf \times \dfrac{h^2}{2} = 23.4^{ft\text{-}k}$

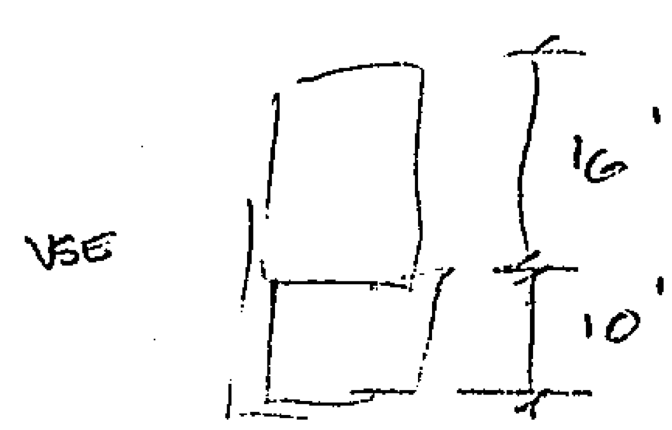
$= h = 16'$

· @ FOUNDATION ·

(4) EPOXY BOLTS IN CORE

$5.1^k \times 4 \times 1.2 = 21^k$ OK

USE 12' $\angle$'s



- DESIGN STRONGBACKING FOR PARTIAL HT. CMU WALLS IN UNIT B

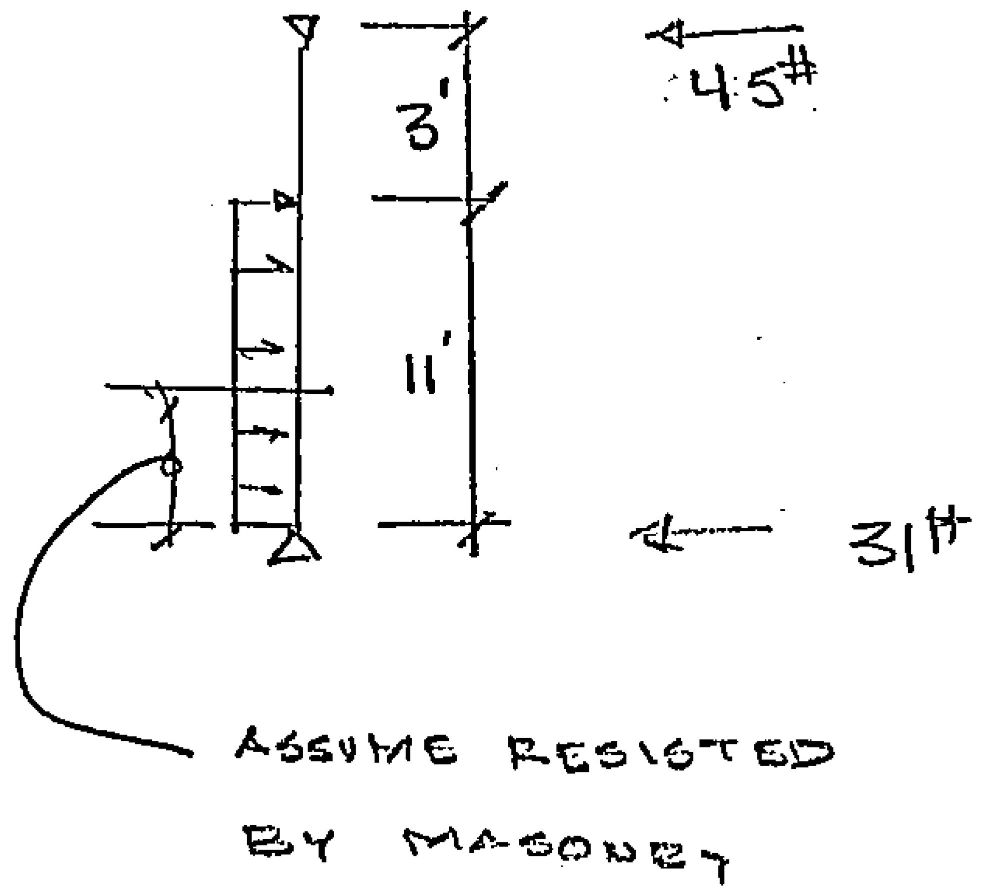
- FORCES:

$F_p = 0.181\ w$

- W/STUDS @ 16" o.c.

$F_p = 0.181 \times 1' \times 1.33' \times 60\ psf$

$= 14.4\ plf$



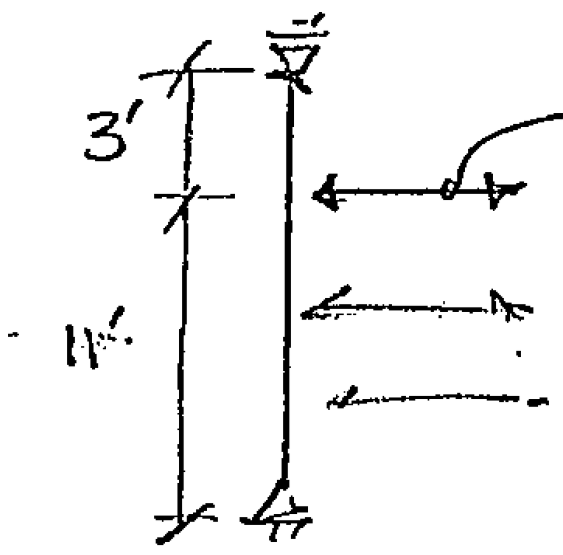
$M_u = 0.20$ ft-k

∴ USE 350S162-18 @ 16" o.c.

$\phi M_n = 0.9\,(33^{ksi})\,(0.302\,in^3 \times 1.2)$

$= 0.896$ ft-k OK

- W/ TS @ 6'-0" o.c.



$F_p = 0.181 \times 5.5' \times 6' \times 55\ psf$ (CONSERVATIVE LOCATION OF FORCE, ACTUALLY DISTRIBUTED)

$= 328^{\#}$

$M_u = \dfrac{0.328^k \times 3' \times 11'}{14'}$

$= 0.773$ ft-k

- DESIGN BOLTS

$F_p = 0.181 \times 6' \times 55\ psf$
$= 60\ plf$

USE 5/8" φ EPOXY
@ 2' o.c. $F_p = 120^{\#}$ (ULT)

$P_{ALLOW} = 175^{\#}$ (ALLOW) OK

$Z_x \geq \dfrac{0.773^{ft\text{-}k} \times 12"/'}{0.9 \times 46\ ksi}$

$\geq 0.224\ in^3$

USE TS 4×2×1/8

WEAK AXIS

$\phi M_n = 0.9 \times 46 \times 1.09 = 45$ in-k
$= 3.76$ ft-k

- DESIGN CONNECTION @ TOP

$F_p = 320^{\#}$

USE (8) #10 SCREWS (1" EMBED)

$\phi R_n = 0.85(2.16)(8 \times 90^{\#})$

$= 1320^{\#} > 320^{\#}$

- CONN @ BASE

USE (2) 1/2" ϕ EPOXY BOLTS INTO SLAB

$2 \times 1930^{\#} = 3.86^{k}$ (ALLOWABLE)

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Unit C

UNIT C

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

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> Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

2. T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

1. T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

C-1

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Sec. 5.3.1)

? T F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of the walls. (Sec. 5.3.2)

? T F REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Sec. 5.3.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry walls are no more than *8 feet long. (Sec. 7.1.6)

T F OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than *25 percent of the wall length, and the available length appears sufficient. (Sec. 7.1.4)

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1) DCR = 82/29 max = 2.8

T F CROSS TIES: There are continuous cross ties between diaphragm chords. (Sec. 7.1.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than *40 feet and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.2.3)

T ~~F~~ UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than *40 feet in areas of high seismicity (A_v greater than or equal to 0.2) and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.3.1)

C-2

CONNECTIONS

T N/A F WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

2 T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

2 T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

2 T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

C-3

- SHEAR STRESS CHECK FOR HI ROOF OF UNIT C:

  - SEISMIC WEIGHT:

6" CMU	50 psf × 13' × 507'	330^K
ROOF	18 psf × (80'×64' + 112'×100')	294^K
		624^K

176
176
75'
30

  - BASE SHEAR

$V_b = 0.182 \times 624^K$

$= 114^K$

  - CHECK SHEAR STRESS IN E/W DIRECTION:

$L_w = 155'$

$v_{avg} = \dfrac{114^K}{155' \times 12''/' \times 6''}$

$= 10.2$ psi < 15 psi ∴ OK $v_u = 735$ plf

  - CHECK SHEAR STRESS IN N/S DIRECTION:

$L_w = 352'$

112,
112
64

$v_{avg} = \dfrac{114^K}{352' \times 12''/' \times 6''}$

$= 4.5$ psi < 15 psi OK $v_u = 324$ plf

- CHECK DIAPHRAGM IN E/W DIRECTION

$$DCR = \frac{2.5 \times 0.3 \times (294^k + 229^k)}{2 \times 300plf \times 80'}$$

$= 8.2$ NO GOOD

DIAPHRAGM SPAN = 176', MAX DCR = 2.9 PER FIGURE C7.4.4.1

REQUIRED DIAPH. SHEAR CAPACITY = 850 plf ULTIMATE

OK, w/ PLYWOOD SHEATHING

- CHECK DIAPHRAGM IN N/S DIRECTION

$$DCR = \frac{(2.5)(0.3)(294^k + 101^k)}{2 \times 300plf \times 176'}$$

$= 2.8$

DIAPHRAGM SPAN = 100', MAX DCR = 3.8 PER FIGURE C7.4.4.1

OK

- DESIGN RETROFIT @ LOW ROOF @ UNIT L (GRID 12)

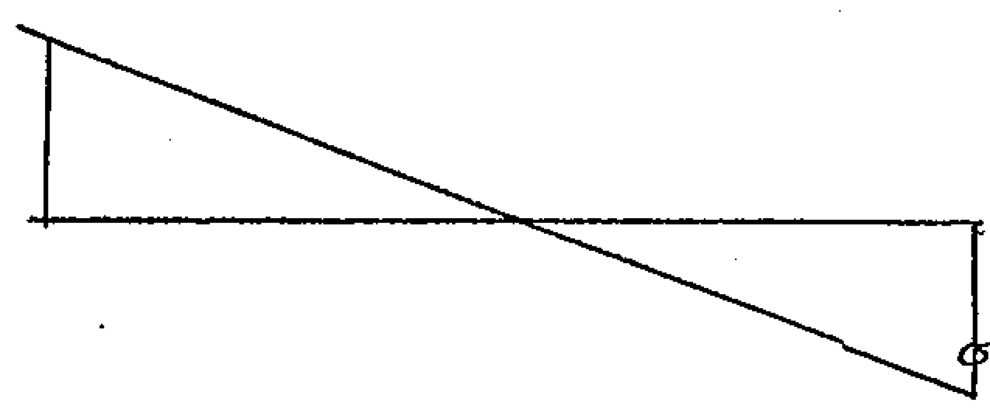
  - FORCES

    $V = \frac{114^k}{2} = 57^k$ FROM HIGH ROOF , $M_{OT} = 57^k \times 16' = 912^{ft\text{-}k}$

    - TRANSFER LENGTH = 100'

      $v_u = \frac{57^k}{100'} = 570$ plf

  - DESIGN STEEL PL ANCHORS

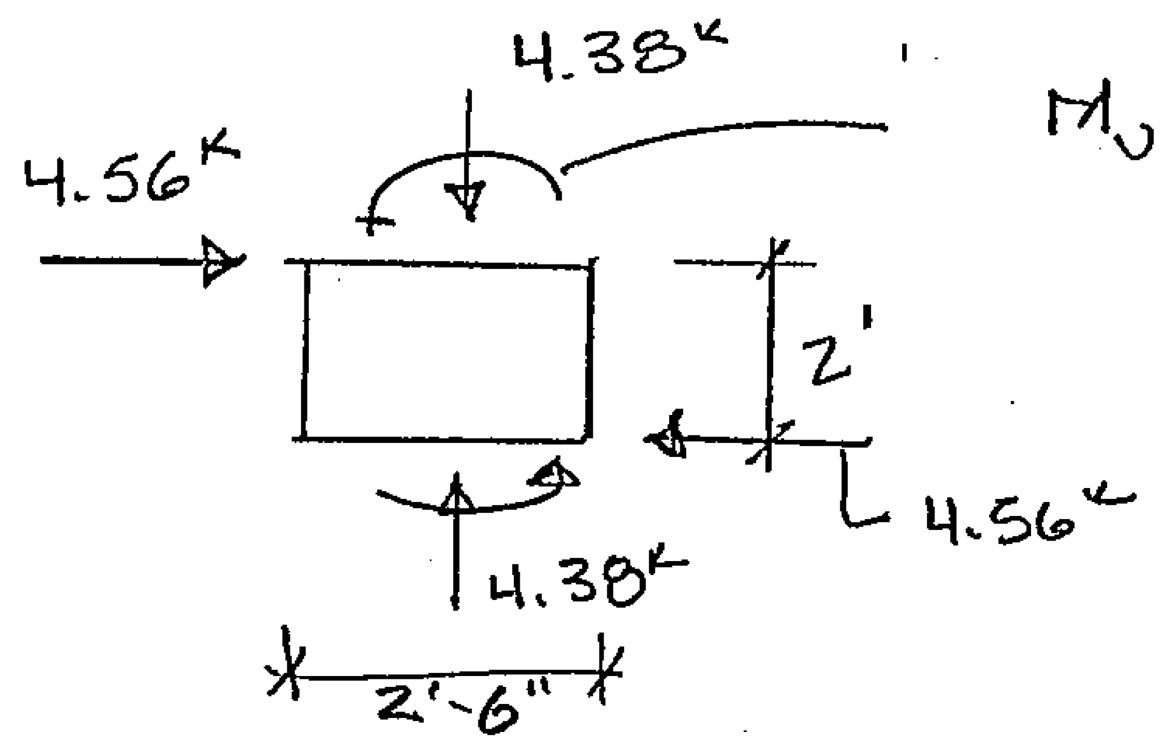


$P = \frac{912^{ft\text{-}k}}{1/6(100')^2} = 547$ plf

USE PL @ 8'-0" oc , $V_u = 570\text{plf} \times 8' = 4.56^k$

$P_u = 547\text{plf} \times 8' = 4.38^k$

$M_u = \frac{4.56^k \times 2'}{2} = 4.56^{ft\text{-}k}$



w/ (4) BOLTS $P_{u\,max} = \frac{4.38^k}{4} + \frac{4.56^{ft\text{-}k}}{2.25'} = 3.12^k$ /BOLT

$V_{u\,max} = \frac{4.56^k}{4} = 1.14^k$ /BOLT

SO $R_u = \sqrt{(3.12^k)^2 + (1.14^k)^2} = 3.32^k$ /BOLT

PER ICBO ER-5279 (SIMPSON SET) $P_{allow} = 1.91^k$/BOLT

SO $\phi P_n = 1.7 \times 1.91^k$/BOLT $= 3.25^k$/BOLT OK

USE THRU-BOLTS, NOT EMBEDDED

- DESIGN STEEL ℞

, TRY ℞ 1/4" x 30"

$\phi V_n = 0.9\,(0.6 \times 36^{ksi}) \times (0.25'' \times 30'')$

$= 146^k > V_u = 4.56^k$ OK

- CHECK COMBINED AXIAL & BENDING

$M_y = \frac{1}{6}(0.25)(30'')^2 \times 36^{ksi} = 1350^{in-k}$

$P_y = 36^{ksi} \times 0.25 \times 30'' = 270^k$

ASSUMING NO BUCKLING $\frac{54.72^{in-k}}{1350^{in-k}} + \frac{4.38^k}{270^k} = 0.06 \ll 1.0$

GOOD BY INSPECTION

- CHECK BUCKLING CAPACITY

$L_c = 24''$

$\lambda_c = \frac{1.0\,(24'')}{0.289 \times 0.25\,\pi} \sqrt{36/29000}$

$= 3.73$

$F_{cr} = \left[\frac{0.877}{3.73^2}\right] \times 36^{ksi} = 2.27^{ksi}$

$\phi_c P_n = 0.85 \times 2.27^{ksi} \times 0.25'' \times \cancel{22.77}\ 30'' = 14.5^k > 4.4^k$ OK

$\phi_b M_n = 0.9\,(2.27^{ksi})(\frac{1}{6} \times 0.25'' \times 30'')^2 = 76.6^{in-k} > M_u = 54.72$

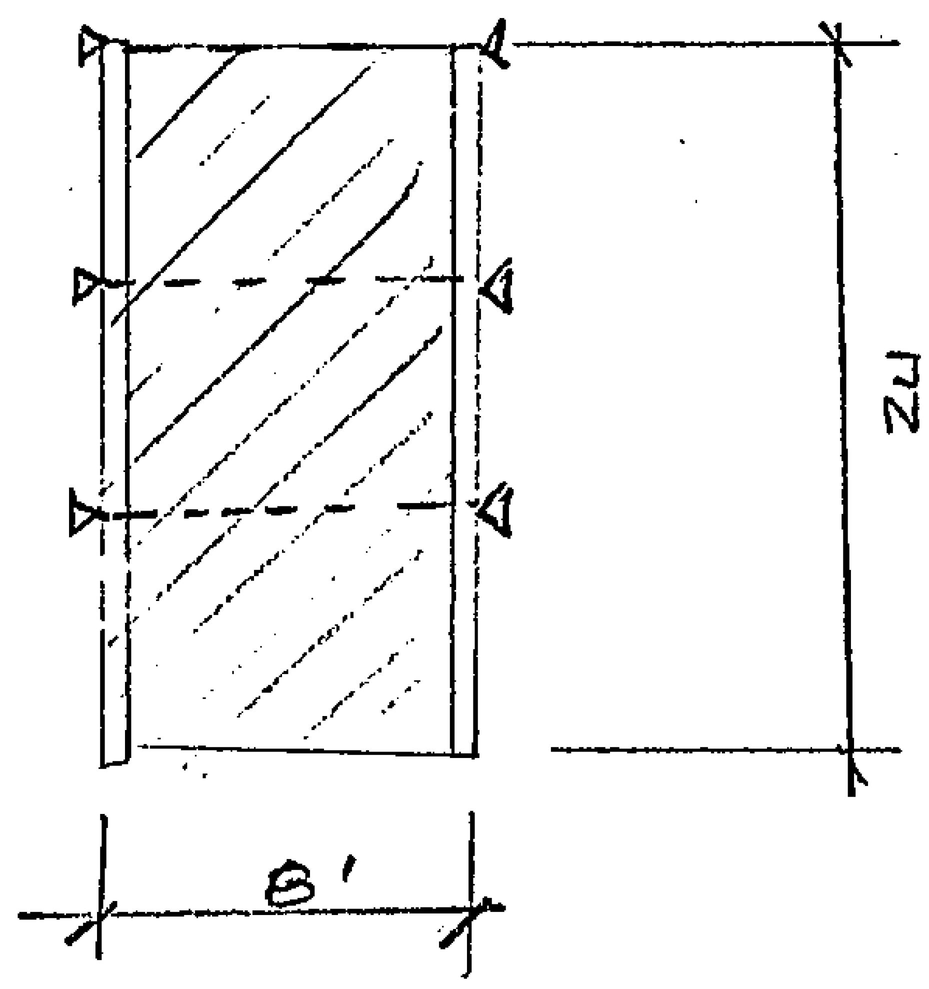
$= 6.39^{ft-k}$

CONSERVATIVE CALL

- DESIGN WALL BRACING FOR UNIT C/D @ CONC. PILASTERS
  - FORCES

$F_p = 0.181\,W_c$

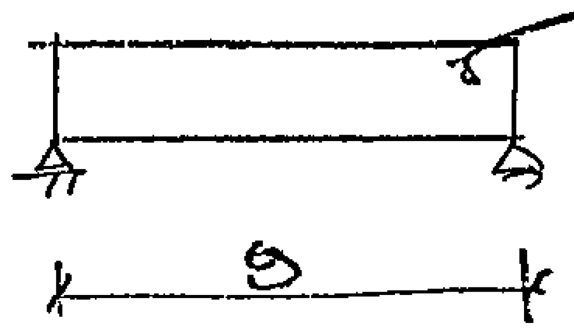
SO FOR 6" CMU $F_p = 0.181 \times 50\,psf$
$= 9.1\,psf$



- CHECK h/t w/ h = 8'

$\frac{8' \times 12''/'}{6} = 16 = 16$ LIMIT PER TABLE C7.4.7.1 FEMA-178

- DESIGN ∠



$9.1\,psf \times 8' = 73\,plf$

$M = \frac{0.073^{k/'} \times 8'^2}{8} = 0.584^{ft\text{-}k}$
$= 7.01^{in\text{-}k}$

$Z_x \geq \frac{7.01^{in\text{-}k}}{0.9 \times 36^{ksi}} = 0.216\,in^3$

∴ USE ∠ 2½ × 2½ × 3/16 $\phi M_n = 0.9 \times 36 \times 0.545$
$= 17.7^{in\text{-}k} > M_u = 7^{in\text{-}k}$ OK

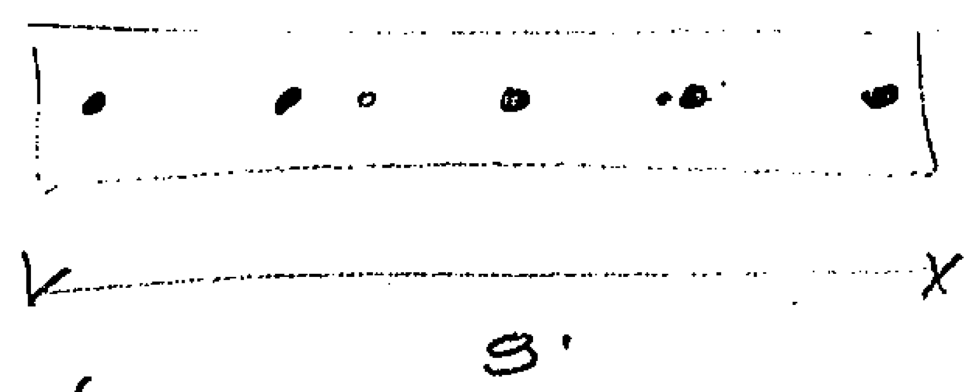
$I_{600} = \frac{(073)(8')^3}{25.78} = 1.45\,in^4$

~~USE~~

- DESIGN EPOXY BOLTS FOR 73 plf

$\phi_t P_n = 1.7 \times 175^{\#}$

$= 298^{\#}$ PER ER-5279



USE ~~2~~ (5) OK

- DESIGN CONN @ PILASTER

$R_u = 73\text{plf} \times 8' = .584^k$

USE (1) 5/8"ϕ EPOXY BOLT

CONTROLLED BY STEEL STRENGTH $\phi R_n = 1.7 \times 5.9^k = 10^k$ OK

- DESIGN CONN @ TOP FOR IN-PLANE

  - SUFFECIENT WALL @ THIS BLDG, MATCH ~~DDRD~~

USE CONN. SIM TO TYPICAL GLU-LAM BLOCKING

BUT USE (2) 3/4"ϕ SC BOLT

• CHECK CONNECTION OF GL TO COLUMN FOR WALL OUT-OF-PLANE FORCES:

not done?

$F_p = 0.67\ (A_v\ C_c\ W_c)$

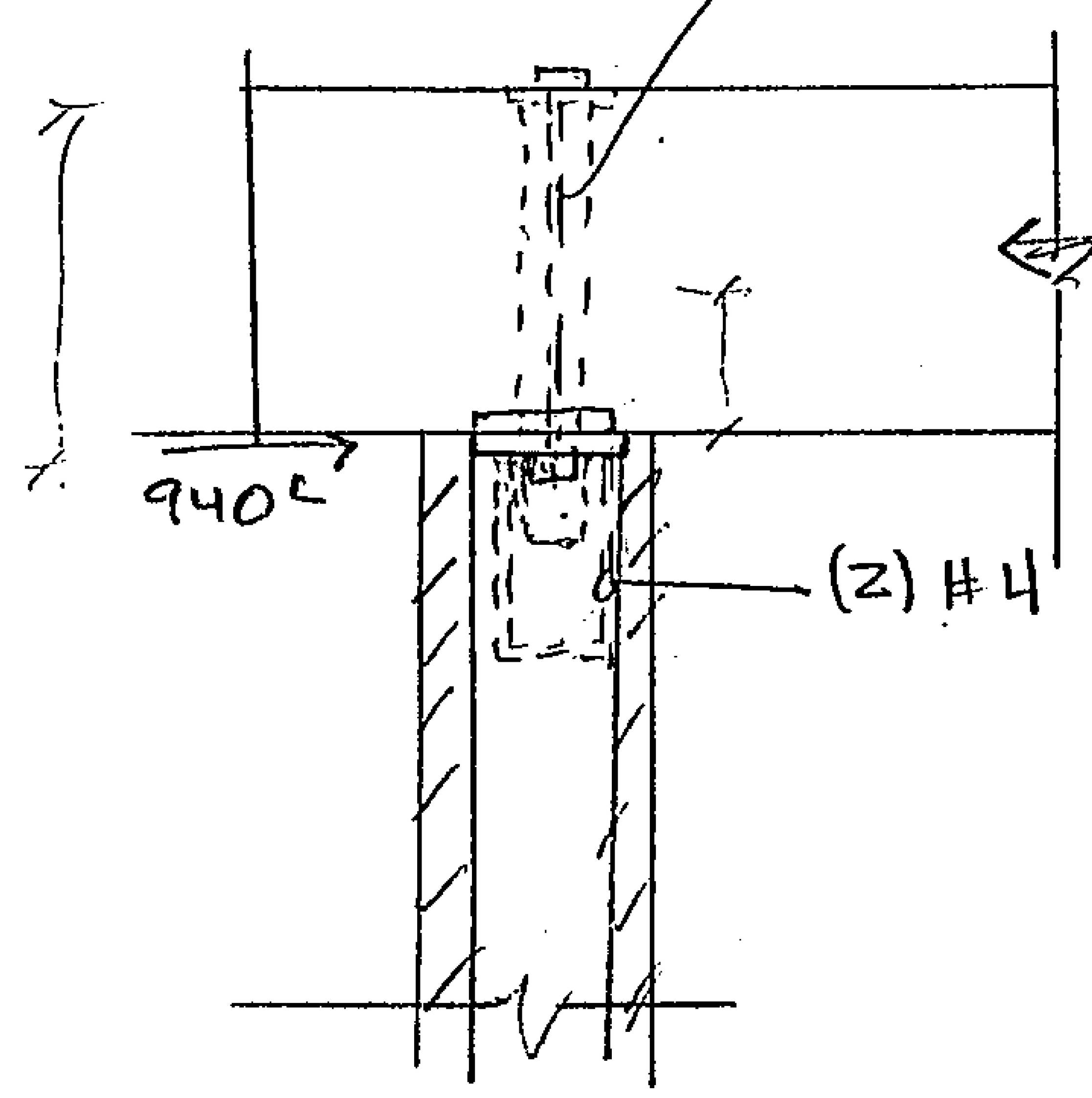
$= 0.67\ (0.3 \times 0.9 \times W_c)$

$= 0.181 \times W_c$

SO FOR 13' × 8' TRIB $W_c = 50\text{psf} \times 13' \times 8' = 5.2^k$

$F_p = 0.181 \times 5.2^k$

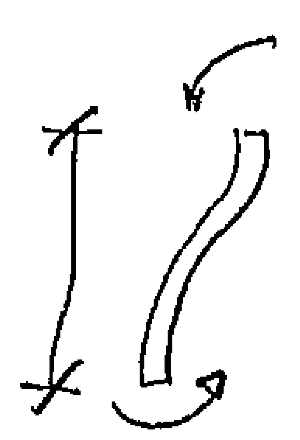
$= 940^\#$ (ULTIMATE)



$\phi V_{n_s} = 7.95^k$ OK

$\phi V_n = (4 \times 0.2 \times 40^{ksi}) \times 0.6 \times 0.85$

$= 16.3^k$ OK

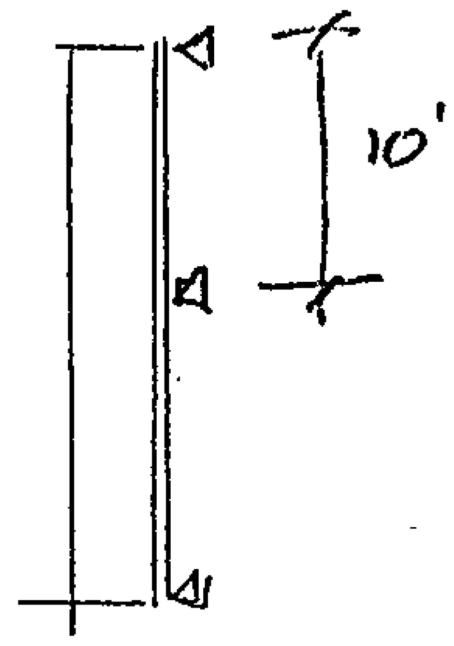


- CHECK BRACING @ UNIT D FOR 20' WALLS (24' @ UNIT C)

$F_p = 0.101 \cdot W_c$

$= 9.1$ psf FOR 6" CMU

- CHECK 20' WALL



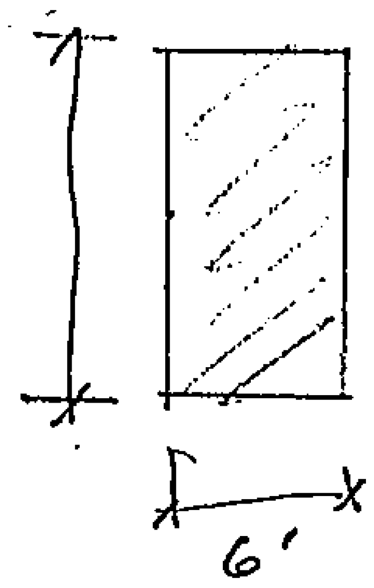
FOR 1' TRIB WIDTH

$M_u = \frac{1}{8}(9.1\,\text{psf})(10')^2$

$= 114$ #-ft

$= \cancel{0.114}$ 1.37 in-k

ALLOWABLE



$F_T = \cancel{19}$ 9 psi ~~MAX~~ MIN PER UBC § 2107.3.5 FOR UNREINFORCED MASONRY w/ MASONRY CEMENT – LOWER BOUND

SO $f_b = \frac{M}{S}$ ASSUMING NO COMPRESSION – CONSERVATIVE

$= \dfrac{\cancel{0.114}\ 1.37}{\frac{1}{6}(12")(6")^2}$

$= \cancel{0.00158}$ ksi $= \cancel{158}$ psi 19 psi

ULTIMATE

COUGHLINPORTERLUNDEEN
A CONSULTING STRUCTURAL AND CIVIL ENGINEERING CORPORATION

Unit D

UNIT D

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns, steel beams are supported by steel columns.

> Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.
>
> Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

2. T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

D-1

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Sec. 5.3.1)

? T F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of the walls. (Sec. 5.3.2)

? T F REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Sec. 5.3.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry walls are no more than *8 feet long. (Sec. 7.1.6)

T F OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than *25 percent of the wall length, and the available length appears sufficient. (Sec. 7.1.4)

SATISFIES DETAILED CHECK → OK

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

T F CROSS TIES: There are continuous cross ties between diaphragm chords. (Sec. 7.1.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than *40 feet and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.2.3)

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than *40 feet in areas of high seismicity (A_v greater than or equal to 0.2) and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.3.1)

UNIT D

CONNECTIONS

T NA F WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

D3

- UNIT D SHEAR STRESS CHECK

  - SEISMIC WEIGHT

    6" CMU $(376' + 346') \times 11' \times 50\,psf$ — 397^k

    N-S EW

    $207^k + 190$

68
23
25
56
52
120

    ROOF $[(72' \times 192') + (68' \times 152')] \times 18\,psf$ — 435^k

    832^k

  - BASE SHEAR

    $V_b = 0.182 \times 832^k$

    $= 151^k$

  - CHECK WALLS IN E/W DIRECTION

    $L_w = 346'$

    $v_{avg} = \dfrac{151^k}{346' \times 12''/' \times 6''}$

    $= 6.1\,psi \quad (436\,plf)$

    * NEED TO COMBINE WALL ON GRID 66 w/ v_{avg} FROM UNIT E

  - CHECK WALLS IN N/S DIRECTION

    $L_w = 376'$

    $v_{avg} = \dfrac{151^k}{376' \times 12''/' \times 6''}$

    $= 5.6\,psi \quad (402\,plf)$

- CHECK DIAPHRAGM

  - E/W DIRECTION

PRO-RATE BY TRIB AREA OF DIAPH. SPAN UNDER CONSIDERATION

$$DCR = \frac{2.5(0.3)(435^k + 207^k)}{2 \times 300 plf \times 136'} \times \underbrace{\frac{136' \times 104'}{24160}}_{0.585}$$

$= 3.5$

DIAPHRAGM SPAN = 104', MAX DCR = 4 OK ~~5.9/4 = 1.5~~

  - N/S DIAPHRAGM

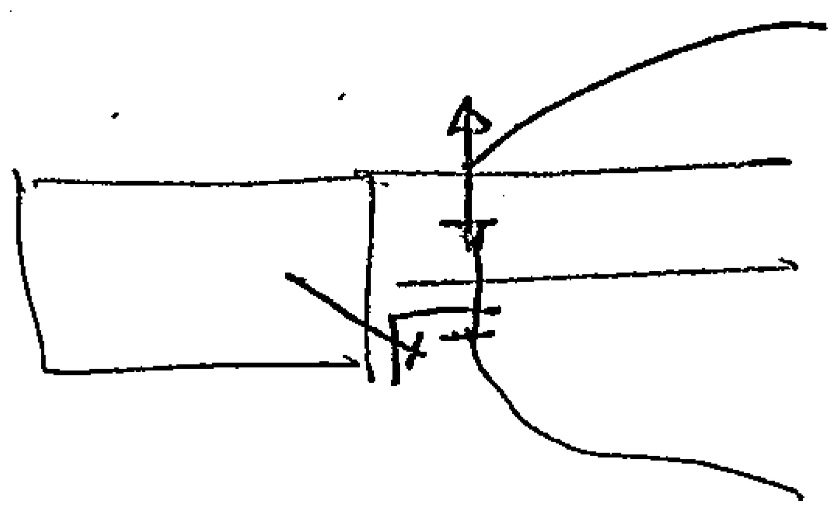
$$DCR = \frac{2.5(0.3)(435^k + 190^k)}{2 \times 300 plf \times 152'} \quad \underbrace{\frac{152' \times 68'}{24160}}_{0.427}$$

$= 2.2$

DIAPHRAGM SPAN = 68', MAX DCR 4.4 OK

• DESIGN CONN @ UNIT D EXTERIOR TO ATTACH MASONRY WALL TO CONC. PILASTER

EX. WALL HAS HORIZ. REINF TO SPAN HORIZONTALLY



$F_p = 0.181 \times 55_{psf} \times 4' = 39.8_{plf}$

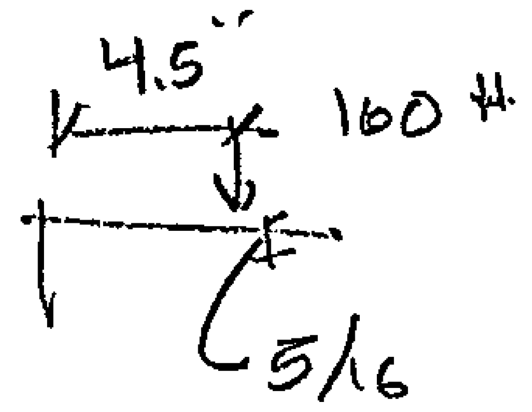
w/ BOLTS @ 4'-0" oc

$P_u = 4' \times 40_{plf} = 160_{plf}$

$\phi P_n = 1.7 \times 175^{\#} = 298^{\#}$ OK

BOLT IN CONC GOOD W/ INSPECTION

• CHECK LEG OF ∠

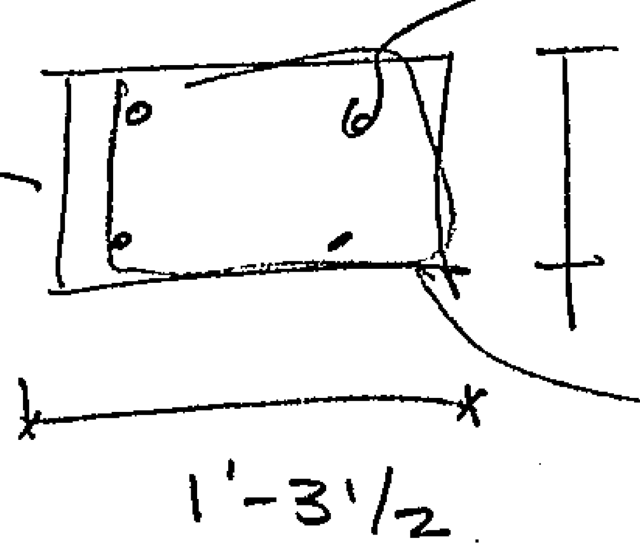


$M_u = 4.5" \times 0.16^k = 0.72^{in\text{-}k}$

$\phi M_n = 0.9 \times 36^{ksi} \times 12" \times \frac{0.3125^2}{4}$

$= 9.5^{in\text{-}k}$ OK

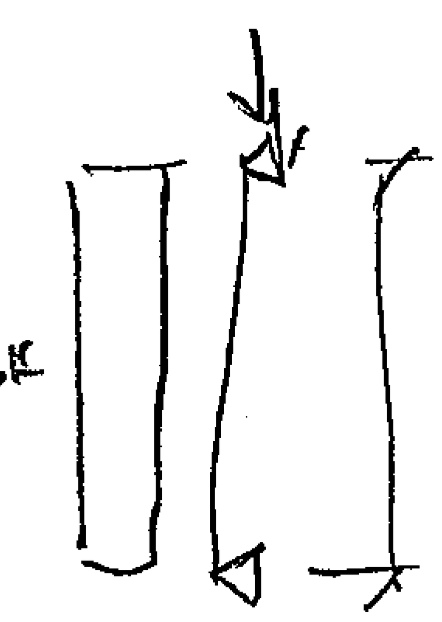
• CHECK EX. PILASTER



$D = 8' \times 16' \times 20_{psf}$

$= 2.6^k$

$L = 3.2^k$



$M_u = \frac{1}{8}(.08)(24)^2$

$= 5.76^{k\text{-}ft}$

$P_u = 1.4(2.6) + 1.7(3.2$

$= 9.1^k$

↑ USE FULL FACTORED COMBO – CONSERVATIVE

```
OOOOOOO    OOOOO     OOOOO     OOOOO     OOOOO    OO
OO    OO  OO    OO  OO    OO  OO    OO  OO    OO  OO
OO    OO  OO    OO  OO    OO  OO    OO  OO    OO  OO
OO    OO  OO        OO    OO  OO        OO    OO  OO
OO    OO  OO        OOOOOOOO  OO        OO    OO  OO
OOOOOOO   OO    OO  OO    OO  OO    OO  OO    OO  OO
OO        OO    OO  OO    OO  OO    OO  OO    OO  OO
OO         OOOOO    OO    OO   OOOOO     OOOOO    OOOOO   (TM)
```

==
Computer program for the Strength Design of Reinforced Concrete Sections
==

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General Information:
====================

File Name: untitled.col
Project: Old Woodway
Column: Ex. PC Col — Engineer: BRZ
Code: ACI 318-95 — Units: English

Run Option: Investigation — Slenderness: Not considered
Run Axis: X-axis — Column Type: Structural

Material Properties:
====================

f'c = 3 ksi — fy = 40 ksi
Ec = 3122.02 ksi — Es = 29000 ksi
fc = 2.55 ksi — Rupture strain = Infinity
Ultimate strain = 0.003 in/in
Beta1 = 0.85

Section:
========

Rectangular: Width = 15.5 in — Depth = 9.5 in

Gross section area, Ag = 147.25 in^2
Ix = 1107.44 in^4 — Iy = 2948.07 in^4
Xo = 0 in — Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7

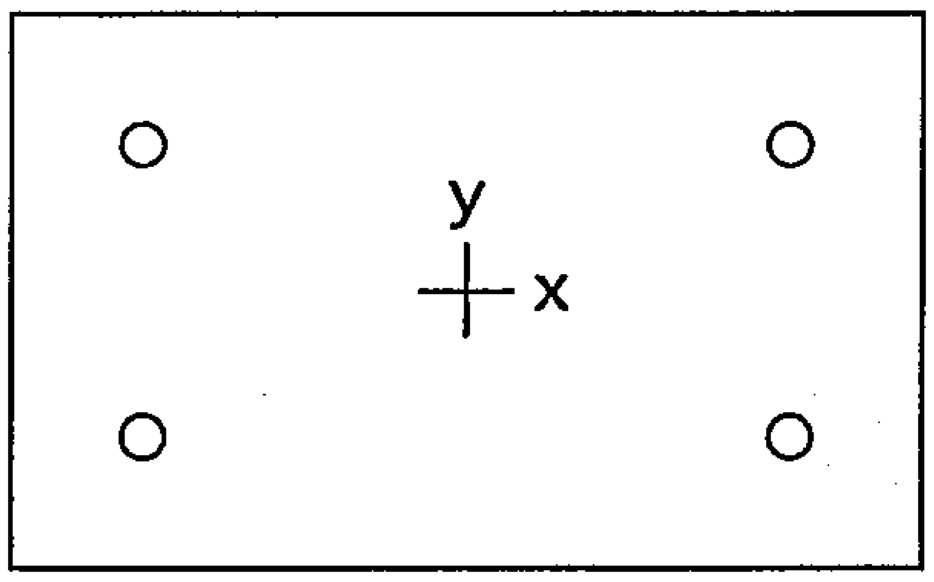
Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 1.76 in^2 at 1.20%
4 #6 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
===

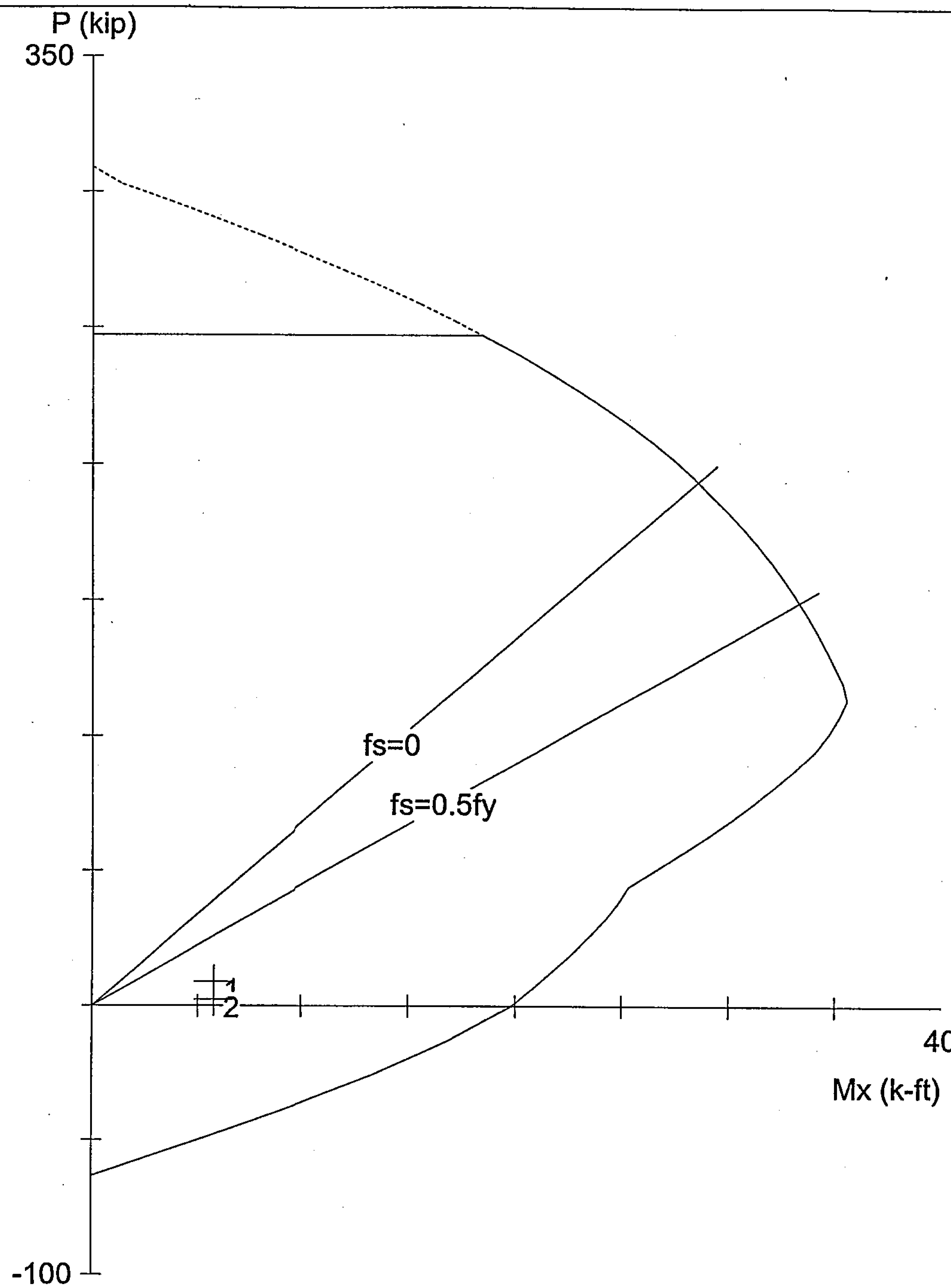
No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	9.1	5.8	21.2	3.679
2	2.3	5.8	20.2	3.502

*** Program completed as requested! ***

D-8



15.5 x 9.5 in



Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/13/02

Time: 07:37:25

PCACOL V3.00 (PCA 1999) - Licensed to: Coughlin Porter Lundeen, Inc., Seattle, WA

File: untitled.col

Project: Old Woodway

Column: Ex. PC Col		Engineer: BRZ	
f'c = 3 ksi	fy = 40 ksi	Ag = 147.25 in^2	4 #6 bars
Ec = 3122 ksi	Es = 29000 ksi	As = 1.76 in^2	Rho = 1.20%
fc = 2.55 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 1107.44 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 2948.07 in^4
Beta1 = 0.85		Clear spacing = 4.25 in	Clear cover = 1.88 in

D-9

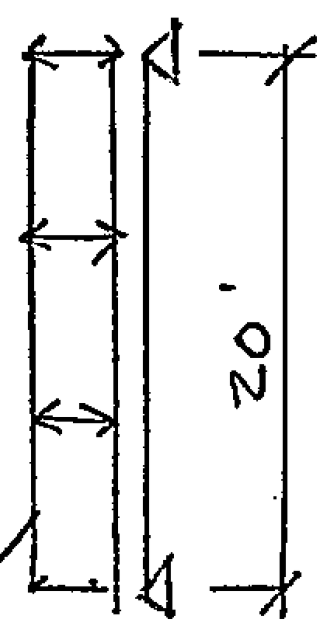
- DESIGN STRONGBACKING FOR WALL @ S. END OF UNIT D
  GRID 66 / AD-AH, AS-AW

- FORCES

$F_p = 0.181 \times 50$ psf

$= 9.1$ psf

- FOR STRONGBACKS @ 8' oc



$F_p = 8' \times 9.1\text{psf} = 73$ plf

- FOR STRENGTH

$M_u = \frac{1}{8}(73\text{plf})(20')^2$

$= 3.65$ ft-k

$Z_x \geq \frac{3.65^{\text{ft-k}} \cdot 12"/'}{0.9 \times 46^{\text{ksi}}}$

≥ 1.06 in^3

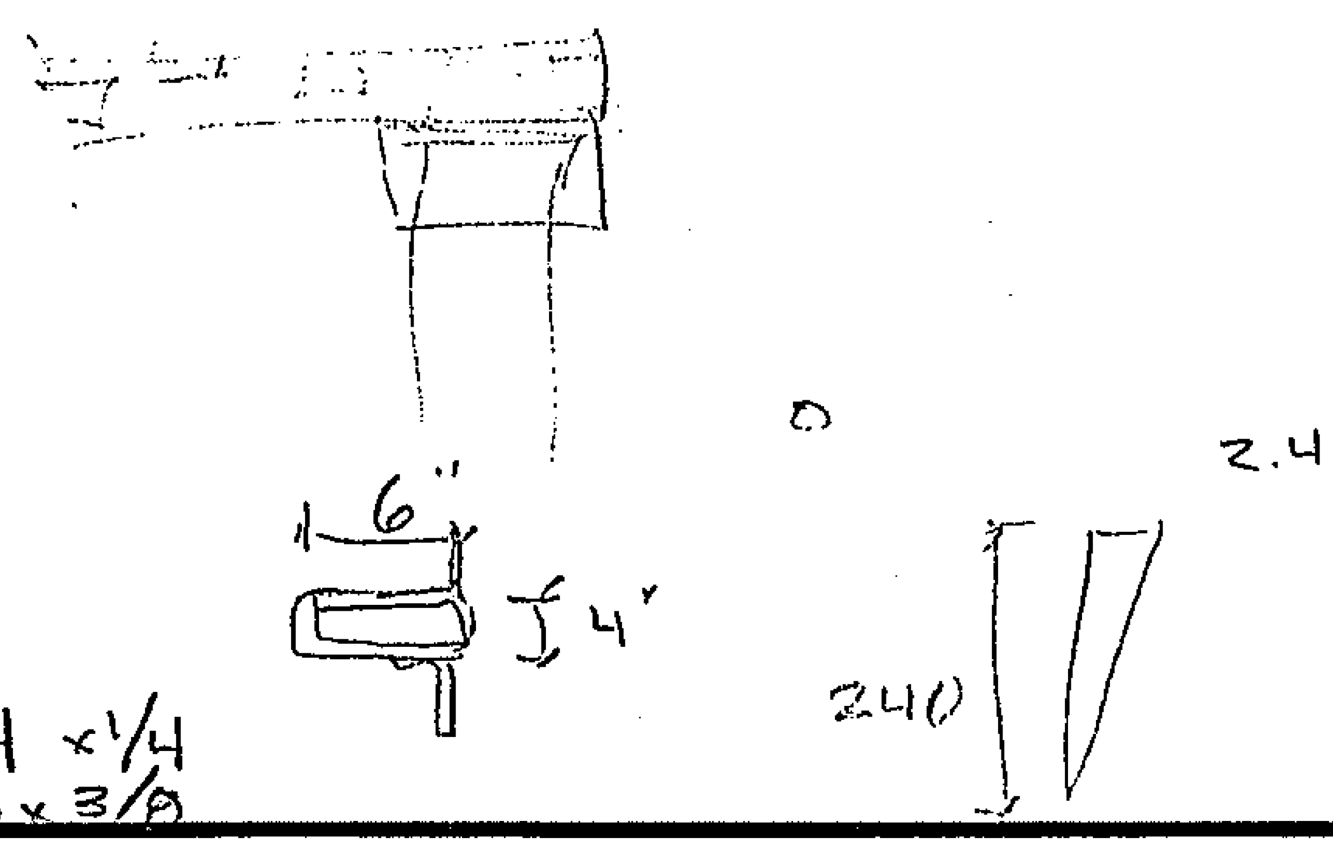
USE TS 4x2x3/16 $\phi M_n = 0.9(46^{\text{ksi}})(1.52\text{ in}^3) = 62.9^{\text{in-k}} = 5.24^{\text{ft-k}}$

- STIFFNESS

$I_{600} = \frac{wL^3}{25.78}$

$= \frac{(0.073^{k/'})(20')^3}{25.78}$

$= 22.65$



USE TS 6x4 x1/4
OR ∠ 6x4 x 3/8

Project:
Designed By: Date
Project No: $\Delta = \frac{L}{240}$ 0.6 Client: L/400
Checked By: Sheet D-10 of

Unit E

UNIT E

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns, steel beams are supported by steel columns.

> Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.
>
> Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

E-1

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Sec. 5.3.1)

T F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of the walls. (Sec. 5.3.2)

T F REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Sec. 5.3.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry walls are no more than *8 feet long. (Sec. 7.1.6)

T F OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than *25 percent of the wall length, and the available length appears sufficient. (Sec. 7.1.4)

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

T F CROSS TIES: There are continuous cross ties between diaphragm chords. (Sec. 7.1.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than *40 feet and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.2.3)

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than *40 feet in areas of high seismicity (A_v greater than or equal to 0.2) and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.3.1)

CONNECTIONS

T N/A F WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

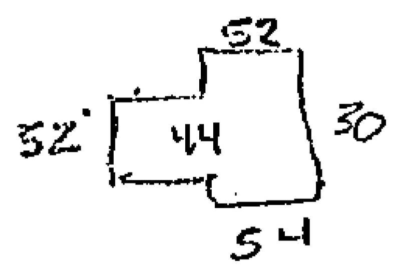
T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

UNIT E SHEAR STRESS CHECK:

- SEISMIC WEIGHT

6" CMU   232' x 7.5' x 50 psf 87^k



54'
44'
30
69
52'

ROOF   [76' x 64' + 72' x 138'] x 18 psf 266^k

353^k

- BASE SHEAR

$V_b = 0.182 \times 353^k$

$= 64^k$

- CHECK WALLS IN E/W DIRECTION

$L_w = 126'$

$v_{avg} = \dfrac{64^k}{126' \times 12"/' \times 6"} = 7.1 \text{ psi}$

- CHECK WALLS IN N/S DIRECTION

$L_w = 106'$

$v_{avg} = 64^k / 106 \times 12 \times 6 = 8.4 \text{ psi}$

- CHECK DIAPHRAGM IN N/S DIRECTION:

$DCR = \dfrac{2.5(0.3)(266^k + 28^k)}{2 \times 300 \text{ plf} \times 72'} = 5.1$

DIAPHRAGM SPAN = 136'   MAX DCR = 3.5

Project: OLD WOODWAY - UNIT E   Designed By: BRZ   Date
Project No: Client: Checked By: Sheet E-4 of

COUGHLINPORTERLUNDEEN
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Unit F

UNIT F

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns, steel beams are supported by steel columns.

> Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.
>
> Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

F-1

UNIT F

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Sec. 5.3.1)

T F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of the walls. (Sec. 5.3.2)

T F REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Sec. 5.3.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry walls are no more than *8 feet long. (Sec. 7.1.6)

T F OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than *25 percent of the wall length, and the available length appears sufficient. (Sec. 7.1.4)

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

T F CROSS TIES: There are continuous cross ties between diaphragm chords. (Sec. 7.1.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than *40 feet and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.2.3)

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than *40 feet in areas of high seismicity (A_v greater than or equal to 0.2) and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.3.1)

F-2

UNIT F

CONNECTIONS

T N/A F WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

F-3

- UNIT F SHEAR STRESS CHECK

  - SEISMIC WEIGHT

    6" CMU WALL $184' \times 5' \times 50_{psf}$ 46^K

    120' 0 64 0 ROOF = $[124' \times 200' - 26' \times 48] \times 18_{psf}$ $\underline{424^K}$

    470^K

  - BASE SHEAR

    $V_b = 0.182 \times 470^K$

    $= 85.5^K$

  - SHEAR STRESS IN E/W DIRECTION

    $v_{avg} = \dfrac{85.5^K}{184' \times 12" \times 6}$

    $= 6.5$ psi

  - SHEAR STRESS IN N/S DIRECTO

    $v_{avg} = \dfrac{85.5^K}{32' \times 12"/' \times 6"}$

    $= 37$ psi NO GOOD

• DESIGN LATERAL SYSTEM FOR BLDG-F - (NORTH-SOUTH DIRECTION)

• BASE SHEAR

$V = 86^k$ FOR CMU WALL

$V = 86^k \times 4.5/6.5 = 60^k$ FOR PLYWOOD SW

• DETERMINE LENGTH OF PSW REQ'D:

$L_w = \frac{60000\#}{300 plf} = 200'$ w/ 8d @ 6" oc

$= \frac{60000\#}{450 plf} = 133'$ w/ 8d @ 4" oc

$= \frac{60000\#}{585 plf} = 103'$ w/ 8d @ 3" oc

$\frac{60000}{150} = 400\#$

∴ USE $5 \times 30' = 150'$ OF WALL w/ 8d @ 4" oc

— check capacity relative to hem fir lumber. Hem fir has SG of 0.43 ⇒ use 0.82 × str. I lumber values

15/32" 1/2" ply w 8d @ 4" oc. ⇒ $430 \times 0.82 = 352.6$ #/ft.

— check capacity of anchor bolt too

table 8.2A NDS pg 61 for HF 16" oc.

$790\# \times 1.33 = \frac{1050.7}{2} = 525\#/ft. / 400 = 1.3$

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Unit G

UNIT G

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

These buildings have perimeter bearing walls of reinforced brick or concrete-block masonry. These walls are the vertical elements in the lateral-force-resisting system. The floors and roofs are framed either with wood joists and beams with plywood or straight or diagonal sheathing or with steel beams with metal deck or without a concrete fill. Wood floor framing is supported by interior wood posts or steel columns, steel beams are supported by steel columns.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

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BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

G-1

UNIT G

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

SHEAR WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the reinforced masonry shear walls. (Sec. 5.3.1)

T F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of the walls. (Sec. 5.3.2)

T F REINFORCING AT OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. (Sec. 5.3.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

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G-2

Unit G

CONNECTIONS

T F WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

G-3

- UNIT G SHEARING STRESS CHECK

- SEISMIC WEIGHT

6" CMU WALL $237' \times 5' \times 50$ psf — 59^k

ROOF — 424^k

Total — 483^k

70'
8
66
49
3
36

- BASE SHEAR

$V_b = 0.182 \times 483^k$

$= 87.9^k$

- SHEAR STRESS IN E/W DIRECTION

$L_w = 52'$

$v_{avg} = \dfrac{88^k}{52' \times 12''/' \times 6''}$

$= 23.5$ psi   NO GOOD   ADD PSW

- SHEAR STRESS IN N/S DIRECTION

$L_w = 185'$

$v_{avg} = \dfrac{88^k}{185' \times 12''/' \times 6''}$

$= 6.6$ psi   OK

$\dfrac{\cancel{32000}}{\cancel{120}}$ 80 / 3 $= 290$

- DETERMINE PSW REQ'D

$15 \text{ psi} \times 52' \times 12''/' \times 6'' = 56^k$ CARRIED BY CMU, $\Delta = 88^k - 56^k = 32^k$

USE 120' OF WALL w/ 450plf CAPACITY (8d @ 4" o.c.)

$\phi V_n = 54^k > 32^k$ OK

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Unit H

Unit 14

EVALUATION STATEMENTS FOR BUILDING TYPE 13: REINFORCED MASONRY BEARING WALLS WITH WOOD/METAL DECK DIAPHRAGMS

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T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

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T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

U-1

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

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DIAPHRAGMS

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T F OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls constitute less than *25 percent of the wall length, and the available length appears sufficient. (Sec. 7.1.4)

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T F CROSS TIES: There are continuous cross ties between diaphragm chords. (Sec. 7.1.2)

T F UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than *40 feet and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.2.3)

T F UNTOPPED DIAPHRAGMS: Untopped metal deck diaphragms consist of horizontal spans of less than *40 feet in areas of high seismicity (A_v greater than or equal to 0.2) and have span/depth ratios less than or equal to *3 to 1. (Sec. 7.3.1)

CONNECTIONS

T N/A F WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Sec. 8.2.1)

T F MASONRY WALL ANCHORS: Masonry walls are connected to the wood or metal deck diaphragms. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

T F WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (Sec. 8.4.4)

H-3

- UNIT H SHEAR STRESS CHECK
  - SEISMIC WEIGHT

N-S CMU 32' + 4' + 22' + 40' + 14' + 17' + 6' + 22' + 28' + 28' + 44' + 34' + 29' = 320' OF WALL

+62' OF WALL NOT SHEAR WALL

SO 382' × 5' × 50 psf — 96^k

E-W CMU 4' + 15' + 8' + 16' + 4' + 40' + 40' + 4' + 8' + 32' + 8' = 179' — 45^k

ROOF (210' × 150') × 18 psf — 567^k

708^k

- BASE SHEAR

$V_b = 0.182 \times 708^k$

$= 129^k$

- SHEAR STRESS IN N/S DIRECTION

$v_{avg} = \dfrac{129^k}{320' \times 12''/' \times 6''} = 5.6\ \text{psi} < 15\ \text{psi}$ OK

- SHEAR STRESS IN E/W DIRECTION

$v_{avg} = \dfrac{129^k}{179' \times 12'' \times 6''} = 10\ \text{psi} < 15\ \text{psi}$ OK

CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date FEB 28, 02 Designed DT
Date Checked

ROOF FRAMING

A) DEAD LOAD

- deck, insoul, joist grd & joists .... 20 psf
- ceiling & mech. .... 5 psf

D_L = 25 psf

B) LIVE LOAD

a) basic snow

$P_f = C_e \times I \times P_g$

$C_e = 1.0$ (slope less then 25%)

$I = 1.0$

$P_g = 25$ psf

$P_f = 1.0 \times 1.0 \times 25 = 25$ psf

b) min roof Live load

$A = 102 \times 148 = 15096$ sf $\Rightarrow$ $L_R = 12$ psf

SNOW LOAD GOVERNS ..... $L_L = 25$ psf

CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date FEB 28, 02 Designed DT

Date Checked

SNOW DRIFTS

A) AROUND PARAPETS (7'-0 HIGH, Wb = 148 ft

$h_r = 7.0\ ft$

$h_b = 25/17.25 = 1.44\ ft$

$h_d = \frac{1}{2}(0.43\sqrt[3]{W_b}\sqrt[4]{P_g+10} - 1.5)$

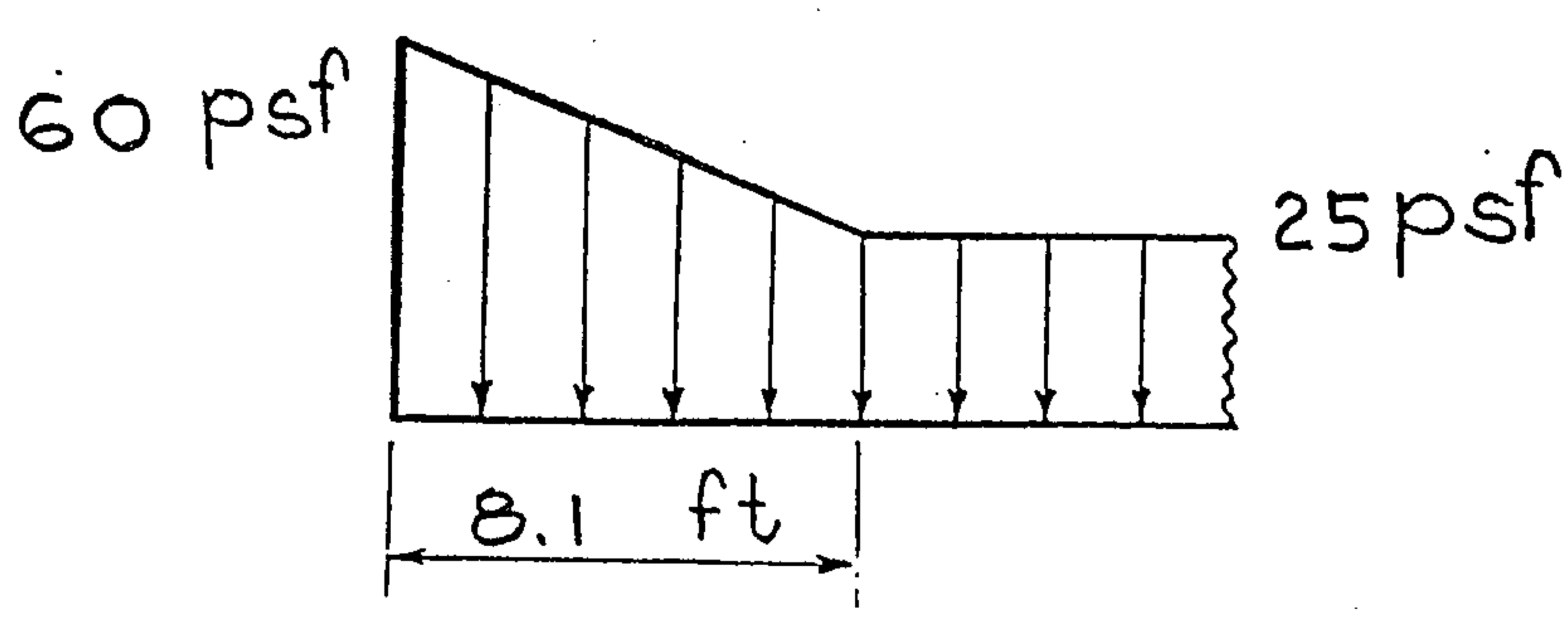
$\underline{W_b = 148\ ft}$

$h_d = \frac{1}{2}(0.43\sqrt[3]{148}\sqrt[4]{25+10} - 1.5) = 2.02\ ft$

$P_m = 25 + 2.02 \times 17.25 = 59.8\ psf$

$$W_d = \min \begin{cases} 4 \times 2.02 = 8.08\ ft \\ 4(7 - 1.44) = 22.24\ ft \end{cases}$$

$W_d = 8.1\ ft$



CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date FEB 28, 02 Designed DT

Date Checked

SNOW DRIFTS

A) AROUND PARAPETS (6'-0 HIGH, Wb = 148 ft

$h_r = 6.0\,ft$

$h_b = 25/17.25 = 1.44\,ft$

$h_d = \frac{1}{2}(0.43\sqrt[3]{W_b}\,\sqrt[4]{P_g+10} - 1.5)$

$P_m = D(h_d + h_b) \le D h_r$

$D = 0.13 P_g + 14 \le 35\,psf$

$= 0.13(25) + 14$

$= 17.25$

$P_m = 17.25(2.02 + 1.44) = 59.67$

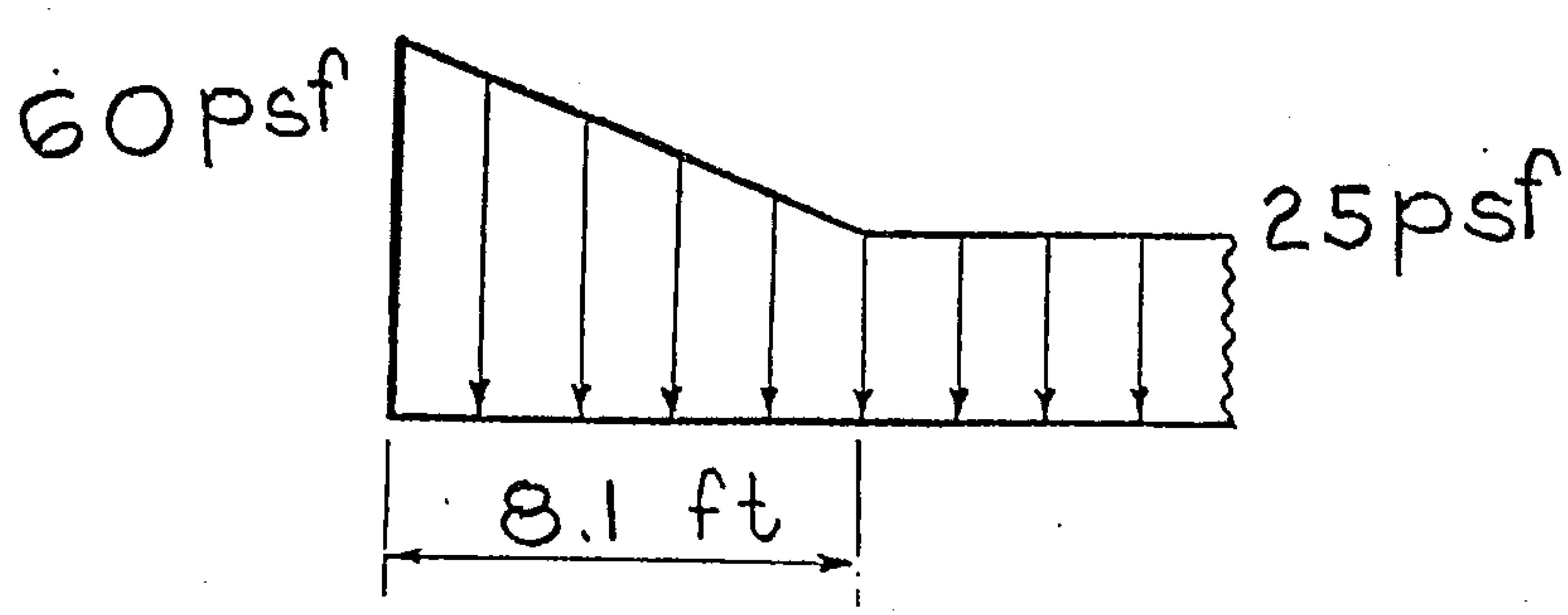
$\underline{W_b = 148\,ft}$

$h_d = \frac{1}{2}(0.43\sqrt[3]{148}\,\sqrt[4]{25+10} - 1.5) = 2.02\,ft$

$P_m = 25 + 2.02 \times 17.25 = 59.8\,psf$

$$W_d = \min \begin{cases} 4 \times 2.02 = 8.08\,ft \\ 4(6 - 1.44) = 18.24\,ft \end{cases}$$

$W_d = 8.1\,ft$



SNOW DRIFTS

A) AROUND PARAPETS (5'-0 HIGH, $W_b = 148$ ft

$h_r = 5.0$ ft

$h_b = 25/17.25 = 1.44$ ft

$h_d = \frac{1}{2}(0.43\sqrt[3]{W_b}\sqrt[4]{P_g+10} - 1.5)$

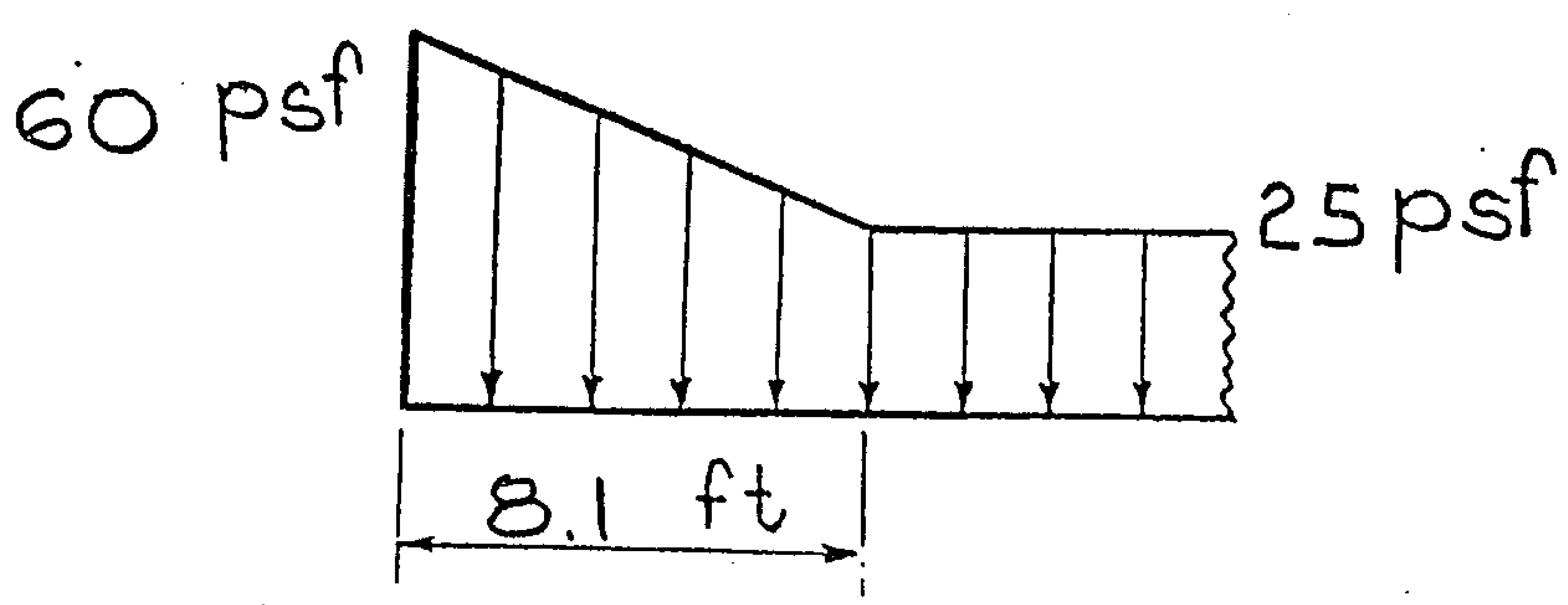
<u>$W_b = 148$ ft</u>

$h_d = \frac{1}{2}(0.43\sqrt[3]{148}\sqrt[4]{25+10} - 1.5) = 2.02$ ft

$P_m = 25 + 2.02 \times 17.25 = 59.8$ psf

$$W_d = \min \begin{cases} 4 \times 2.02 = 8.08 \text{ ft} \\ 4(5 - 1.44) = 14.24 \text{ ft} \end{cases}$$

$W_d = 8.1$ ft



CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date FEB 28, 02 Designed DT

Date Checked

SNOW DRIFTS

A) AROUND PARAPETS (3'-0 HIGH, Wb = 148 ft

$h_r = 3.0\,ft$

$h_b = 25/17.25 = 1.44\ ft$

$h_d = \frac{1}{2}(0.43\sqrt[3]{W_b}\ \sqrt[4]{P_g+10} - 1.5)$

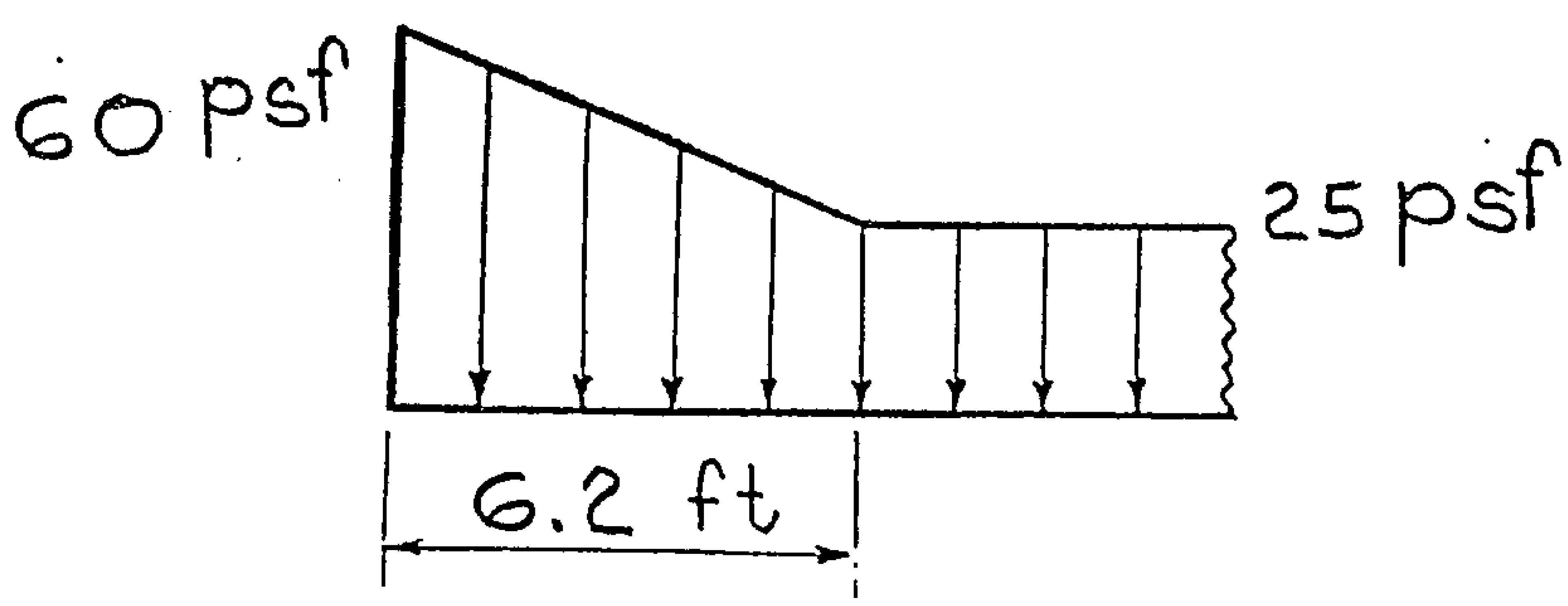
$\underline{W_b = 148\,ft}$

$h_d = \frac{1}{2}(0.43\sqrt[3]{148}\ \sqrt[4]{25+10} - 1.5) = 2.02\ ft$

$P_m = 25 + 2.02 \times 17.25 = 59.8\ psf$

$$W_d = \min\begin{cases} 4 \times 2.02 = 8.08\,ft \\ 4(3 - 1.44) = 6.24\,ft \end{cases}$$

$W_d = 6.2\ ft$



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WESTGATE VILLAGE Date FEB 28, 02 Designed DT

Date Checked

SNOW DRIFTS

A) AROUND PARAPETS (7'-0 HIGH, Wb = 102 ft

$h_r = 7.0 \text{ ft}$

$h_b = 25/17.25 = 1.44 \text{ ft}$

$h_d = \frac{1}{2}(0.43\sqrt[3]{W_b}\sqrt[4]{P_g+10} - 1.5)$

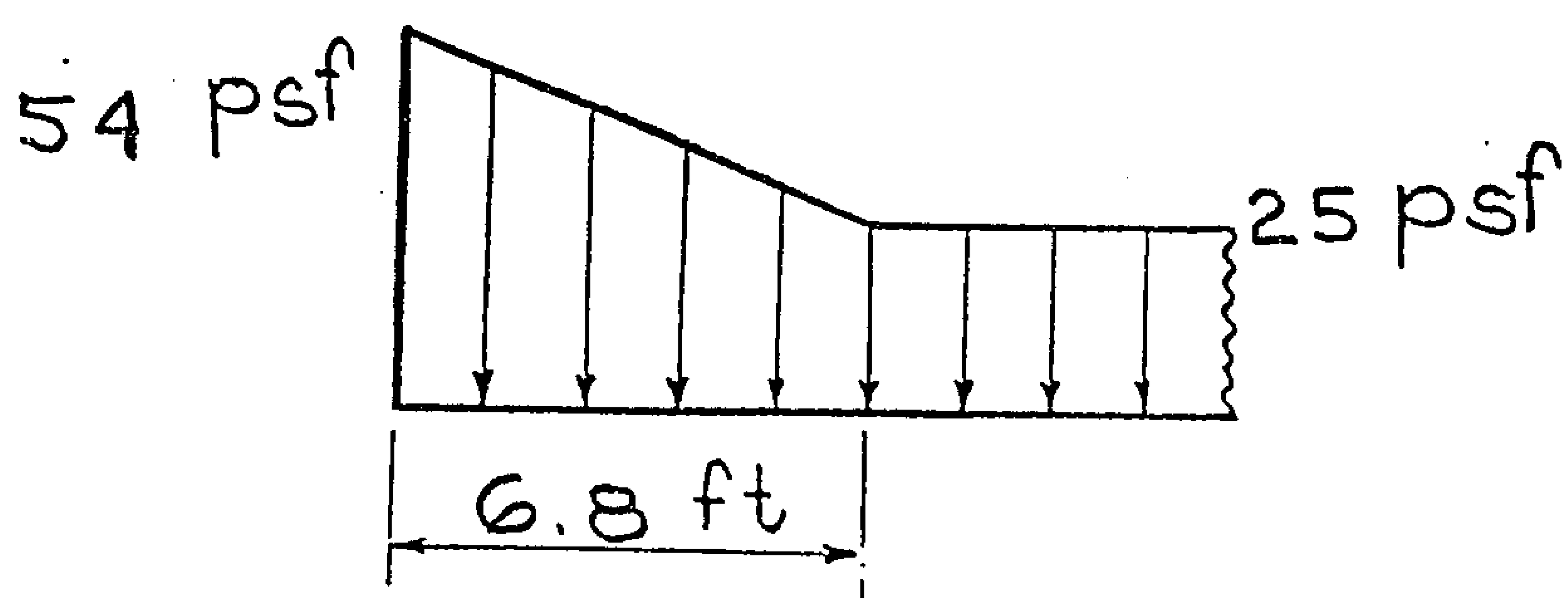
$W_b = 102 \text{ ft}$

$h_d = \frac{1}{2}(0.43\sqrt[3]{102}\sqrt[4]{25+10} - 1.5) = 1.69 \text{ ft}$

$P_m = 25 + 1.69 \times 17.25 = 54.15 \text{ psf}$

$$W_d = \min \begin{cases} 4 \times 1.69 = 6.76 \text{ ft} \\ 4(7 - 1.44) = 22.24 \text{ ft} \end{cases}$$

$W_d = 6.8 \text{ ft}$



SNOW DRIFTS

A) AROUND PARAPETS (6'-0 HIGH, Wb = 102ft

$h_r = 6.0\,ft$

$h_b = 25/17.25 = 1.44\ ft$

$h_d = \frac{1}{2}(0.43\sqrt[3]{W_b}\ \sqrt[4]{P_g+10} - 1.5)$

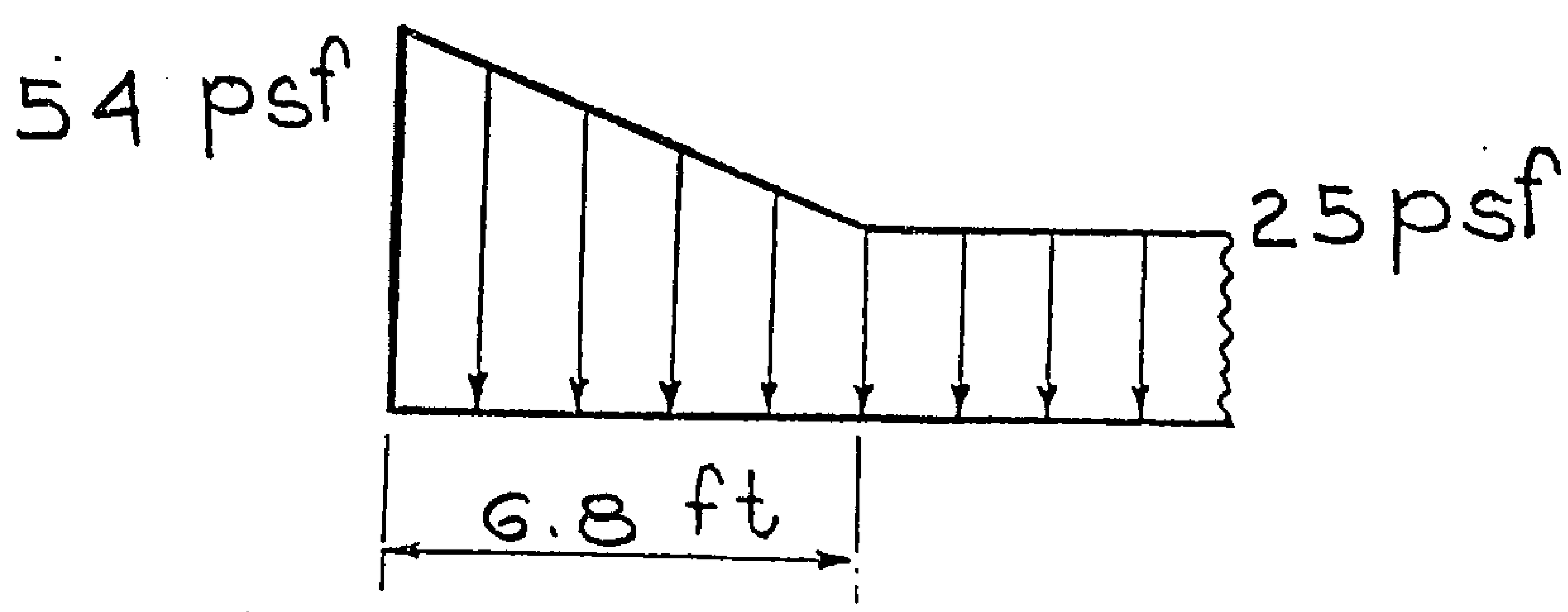
$W_b = 102\ ft$

$h_d = \frac{1}{2}(0.43\sqrt[3]{102}\ \sqrt[4]{25+10} - 1.5) = 1.69\ ft$

$P_m = 25 + 1.69 \times 17.25 = 54.15\ psf$

$$W_d = \min\begin{cases} 4 \times 1.69 = 6.76\,ft \\ 4(6 - 1.44) = 18.24\,ft \end{cases}$$

$W_d = 6.8\ ft$



WESTGATE VILLAGE Date FEB 28, 02 Designed DT

Date Checked

SNOW DRIFTS

A) AROUND PARAPETS (5'-0 HIGH, $W_b = 102$ ft

$h_r = 5.0$ ft

$h_b = 25/17.25 = 1.44$ ft

$$h_d = \frac{1}{2}\left(0.43\sqrt[3]{W_b}\,\sqrt[4]{P_g + 10} - 1.5\right)$$

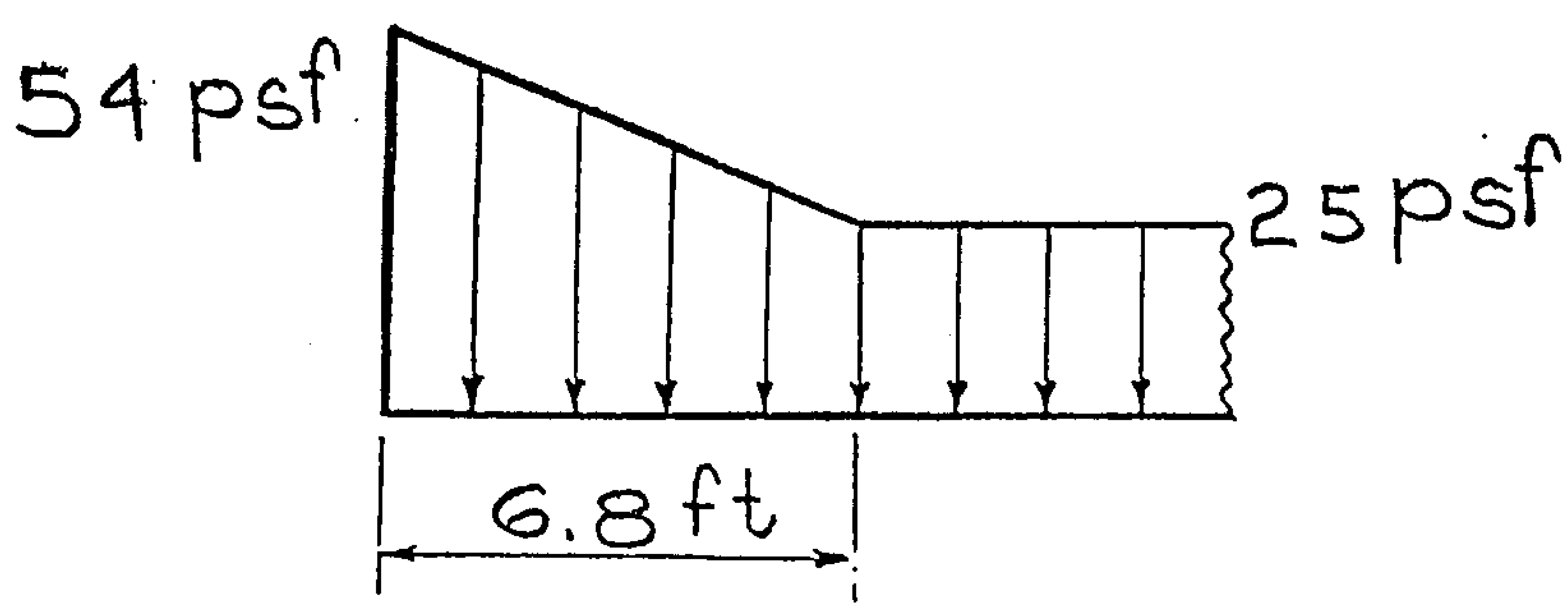
$\underline{W_b = 102 \text{ ft}}$

$$h_d = \frac{1}{2}\left(0.43\sqrt[3]{102}\,\sqrt[4]{25+10} - 1.5\right) = 1.69 \text{ ft}$$

$P_m = 25 + 1.69 \times 17.25 = 54.15$ psf

$$W_d = \min\begin{cases} 4 \times 1.69 = 6.76 \text{ ft} \\ 4(5 - 1.44) = 14.24 \text{ ft} \end{cases}$$

$W_d = 6.8$ ft



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WESTGATE VILLAGE Date FEB 28, 02 Designed DT

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SNOW DRIFTS

A) AROUND PARAPETS (4'-0 HIGH, Wb = 102 ft

$hr = 4.0 ft$

$hb = 25/17.25 = 1.44\ ft$

$hd = \frac{1}{2}(0.43\sqrt[3]{Wb}\sqrt[4]{P_g+10} - 1.5)$

Wb = 102 ft

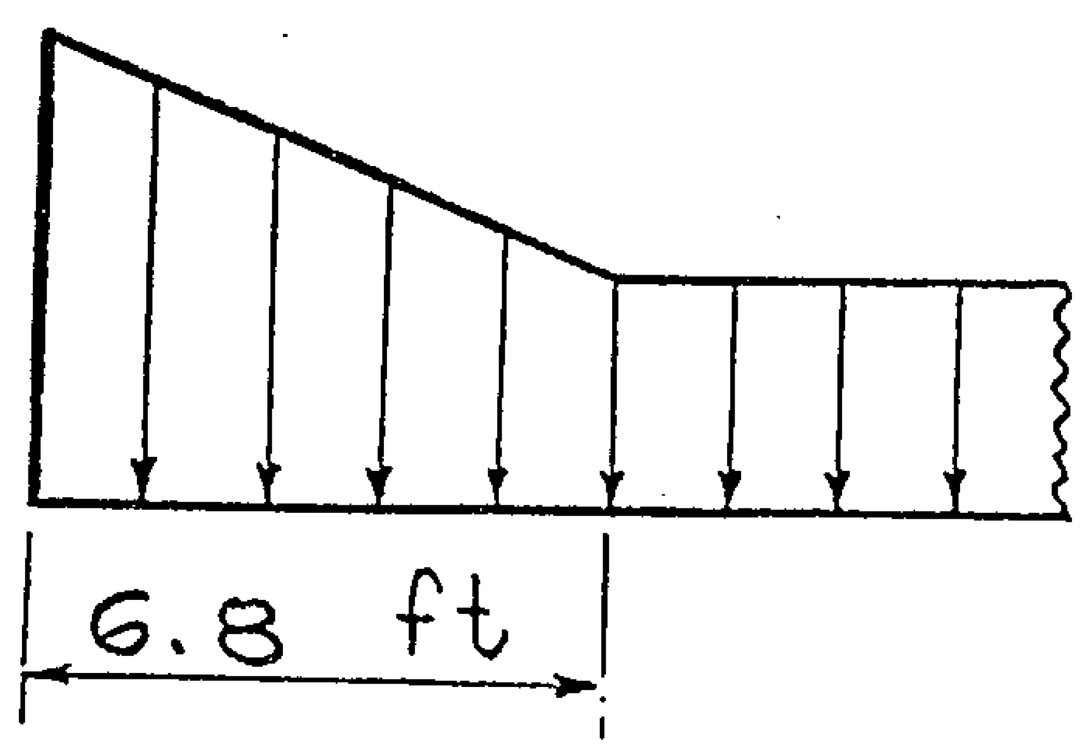
$hd = \frac{1}{2}(0.43\sqrt[3]{102}\sqrt[4]{25+10} - 1.5) = 1.69\ ft$

$P_m = 25 + 1.69 \times 17.25 = 54.15\ psf$

$$Wd = \min\begin{cases} 4 \times 1.69 = 6.76\ ft \\ 4(4 - 1.44) = 10.24\ ft \end{cases}$$

$Wd = 6.8\ ft$

54 psf



psf

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WESTGATE VILLAGE — Date FEB 28, 02 — Designed DT
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SNOW DRIFTS

A) AROUND PARAPETS (3'-0 HIGH, $W_b = 102$ ft

$h_r = 3.0$ ft

$h_b = 25/17.25 = 1.44$ ft

$h_d = \frac{1}{2}(0.43\sqrt[3]{W_b}\,\sqrt[4]{P_g + 10} - 1.5)$

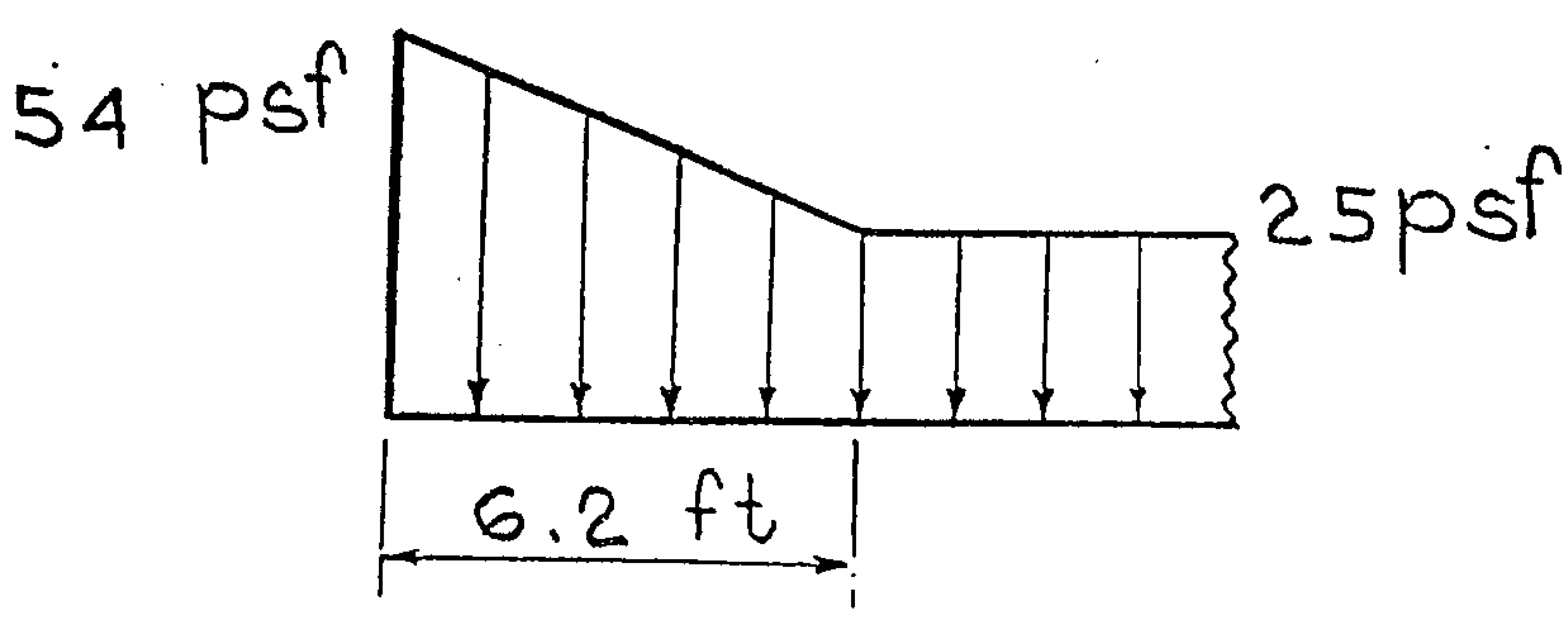
$\underline{W_b = 102 \text{ ft}}$

$h_d = \frac{1}{2}(0.43\sqrt[3]{102}\,\sqrt[4]{25+10} - 1.5) = 1.69$ ft

$P_m = 25 + 1.69 \times 17.25 = 54.15$ psf

$$W_d = \min \begin{cases} 4 \times 1.69 = 6.76 \text{ ft} \\ 4(3 - 1.44) = 6.24 \text{ ft} \end{cases}$$

$W_d = 6.2$ ft



Job No. 9919

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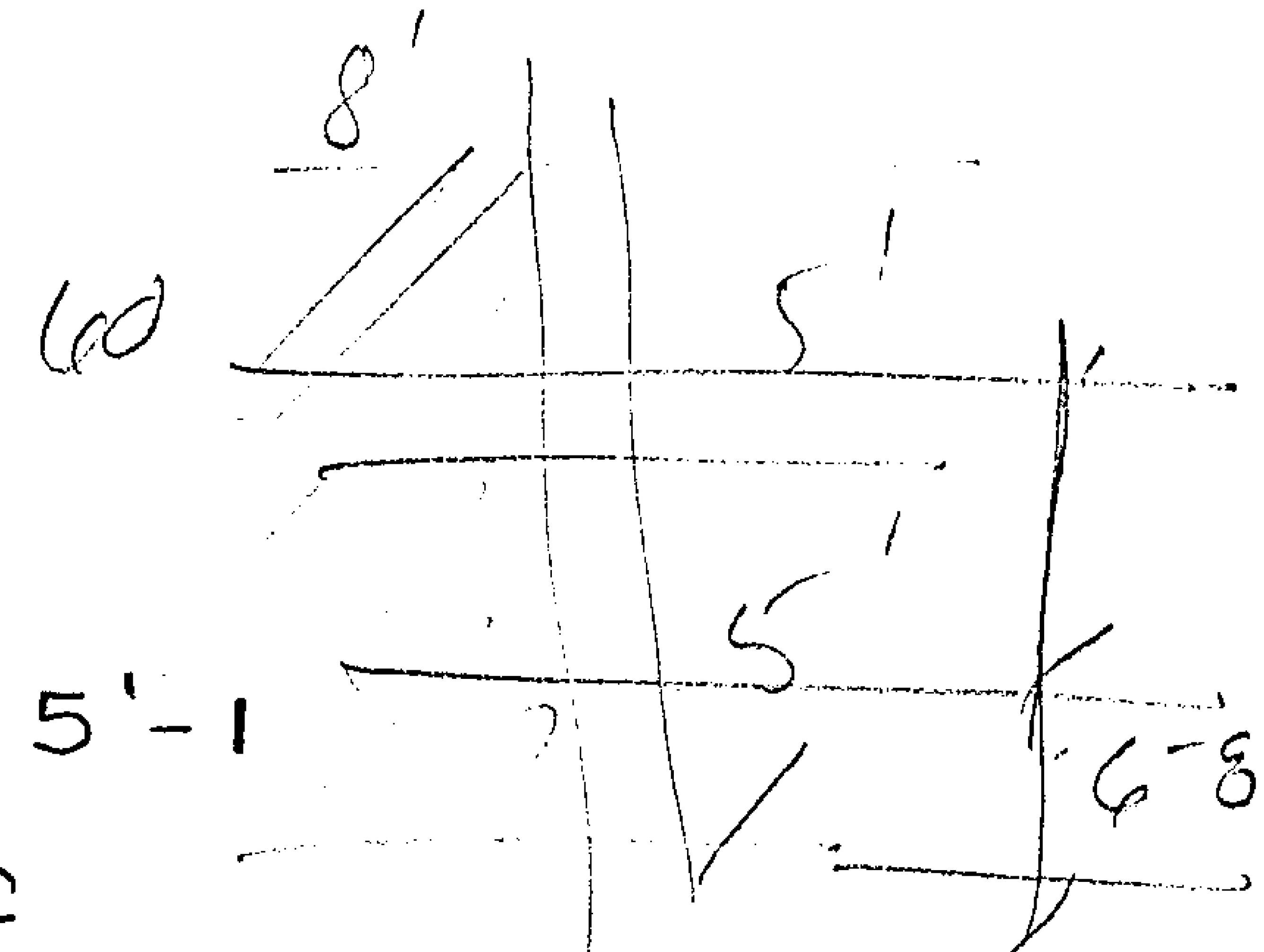
JOISTS & DECK

J1



- SPAN: 28'-6
- SPACING: (25'-5)/5 = 5'-1
- SNOW LOAD: 60 psf

60(5) = 300 #/ft LL

- ADDITIONAL SNOW LOAD: —
- ADDITIONAL MECH. LOAD: SEE PLAN
- DECK: 22GA

60 × 5 = 300

- JOIST FACTORED LINE LOAD:

$(5'-1) \times (60 \times 1.6 + 25 \times 1.2) = 640$ plf

- JOIST DEPTH: 24"

*NOTES:

- FOR ALL JOISTS DEAD LOAD IS 25 psf.
- DECK GA CAN CHANGE LATE IN THE DESIGN DUE TO SHEAR LOADS IN DIAPHRAGM.

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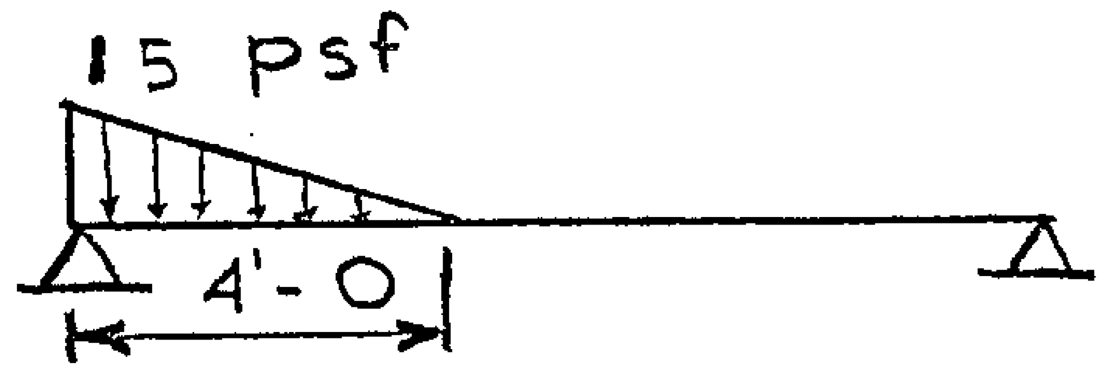
J2

- SPAN: 29'-6"16
- SPACING: (35'-1) / 6 = $5'\text{-}10\frac{3}{16}$
- SNOW LOAD: 45 psf
- ADD. SNOW LOAD:



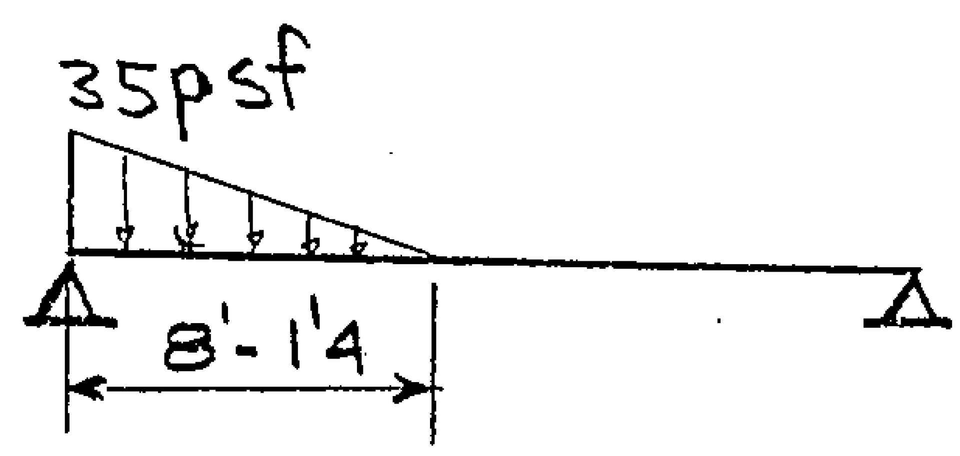
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'\text{-}10\frac{3}{16}) \times (45 \times 1.6 + 25 \times 1.2) = 574$ plf

- JOIST DEPTH: 24"

J3

- SPAN: VARIES FROM 29'-6"16 TO 34'-10
- SPACING: (35'-1) / 6 = $5'\text{-}10\frac{3}{16}$
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'\text{-}10\frac{3}{16}) \times (25 \times 1.6 + 25 \times 1.2) = 409$ plf

- JOIST DEPTH: 26"

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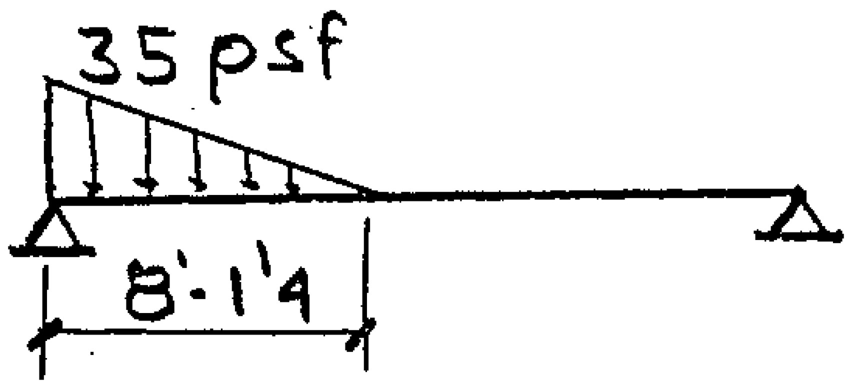
J4

- SPAN: 34'-10 $5(25) = 125$
- SPACING: (16'-6) / 3 = 5'-6
- SNOW LOAD: 25 psf $5 \times 60 = 300$
- ADD. SNOW LOAD:



- ADD. MECH LOAD: –
- DECK: 22 GA

$$\frac{60 - 25}{35} \times 5 = 175$$

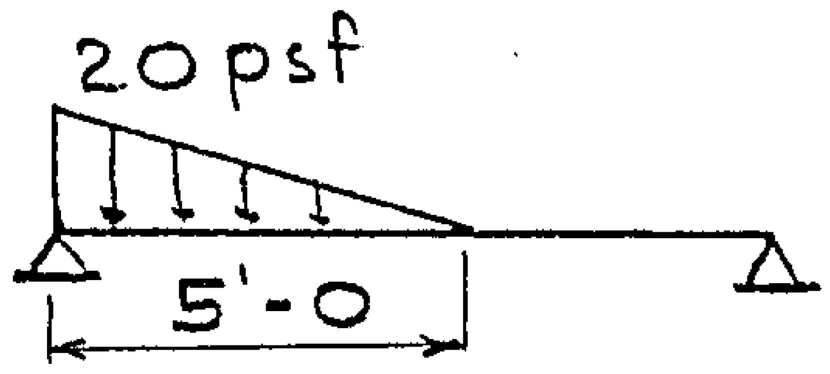
- JOIST FACTORED LINE LOAD: Why factored?

(5'-6) × (25 × 1.6 + 25 × 1.2) = 385 plf

- JOIST DEPTH: 26"

J5

- SPAN: 34'-10
- SPACING: (20'-0) / 4 = 5'-0
- SNOW LOAD: 40 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-0) × (40 × 1.6 + 25 × 1.2) = 470 plf

- JOIST DEPTH: 26"

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$\boxed{J6}$

- SPAN: 34'10 ✓

28-6
6-4
24-10"

- SPACING: (20'-0) /4/2 = 2'-6 ?

5/2 = 2'-6" ok

- SNOW LOAD: 50 psf ok

- ADD. SNOW LOAD: /

- ADD. MECH LOAD: /

- DECK: 22 GA

Service
(50 + 25)(2.5)
= 125 + 62.5
= 187.5

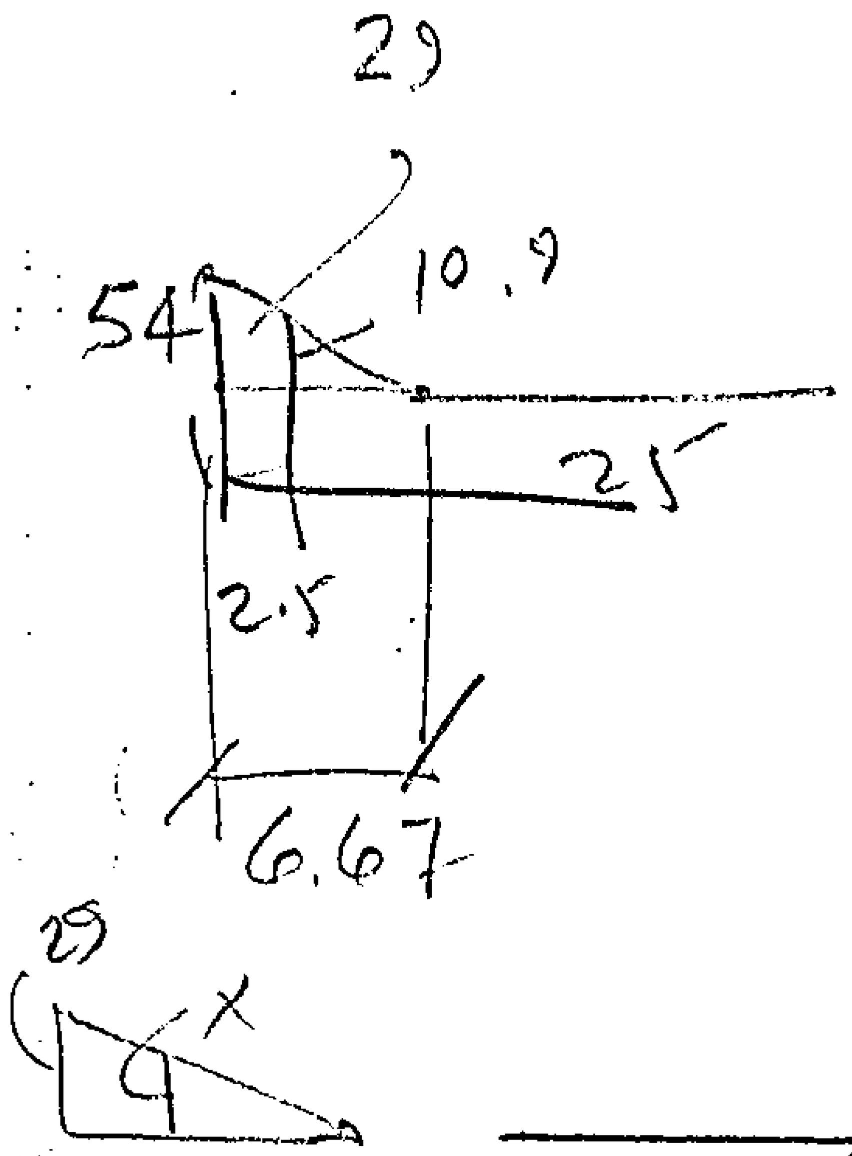
- JOIST FACTORED LINE LOAD:

(2'-6) × (50 × 1.6 + 25 × 1.2) = 275 plf

- JOIST DEPTH: 24"

$\left(\frac{29+10.2}{2}\right)2.5($

=



$\frac{x}{29} = \frac{2.5}{6.67}$

x = 49.8 plf

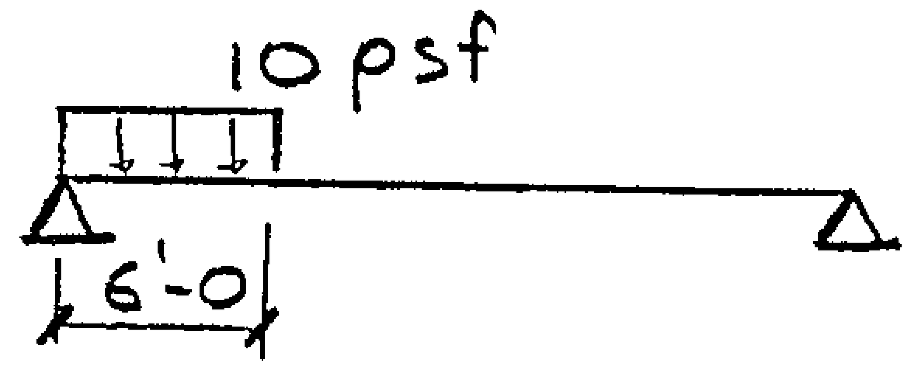
$\boxed{J7}$

- SPAN: 40'-0

- SPACING: (28'-2) /5/2 = 2'-9 13/16

- SNOW LOAD: 50 psf

- ADD. SNOW LOAD:



- ADD. MECH LOAD: /

- DECK: 22 GA

- JOIST FACTORED LINE LOAD:

(2'-9 13/16) × (50 × 1.6 + 25 × 1.2) = 310 plf

- JOIST DEPTH: 26"

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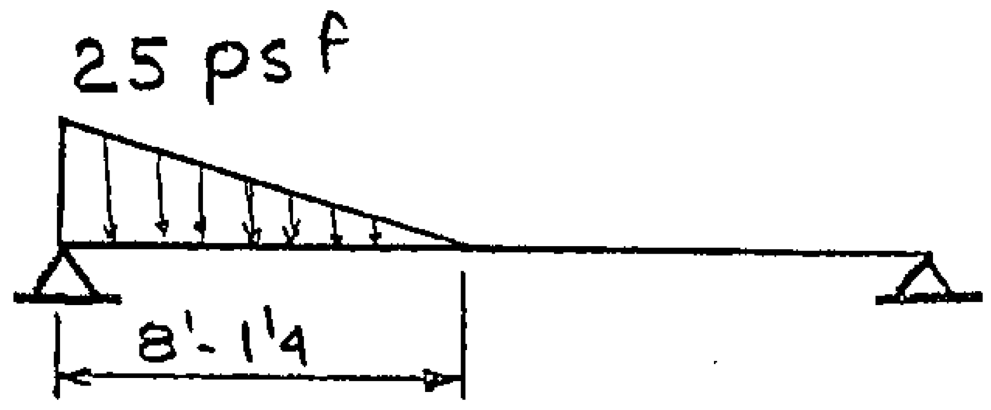
$60 - 25 = 35 \times 5'-6 = 1960$ #/ft

J8

- SPAN: 40'-0
- SPACING: (28'-2) / 5 = 5'-7 5/8
- SNOW LOAD: 40 psf
- ADD. SNOW LOAD:



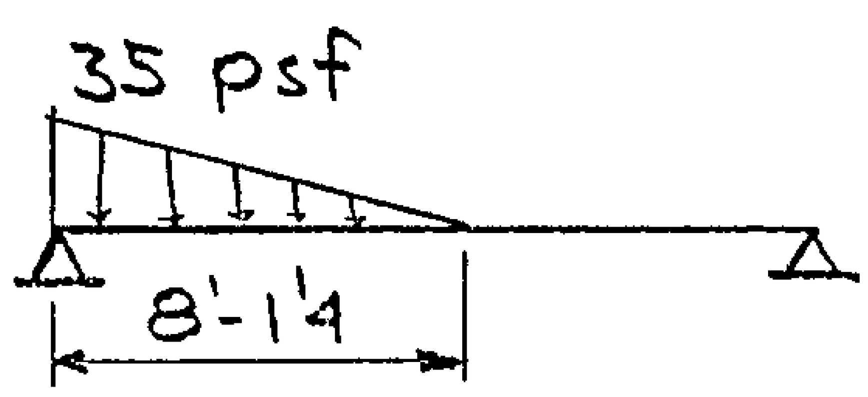
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-7 5/8) × (40 × 1.6 + 25 × 1.2) = 530 plf

- JOIST DEPTH: 28"

J9

- SPAN: 40'-0
- SPACING: (28'-2) / 5 = 5'-7 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-7 5/8) × (25 × 1.6 + 25 × 1.2) = 394 plf

- JOIST DEPTH: 28"

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J10

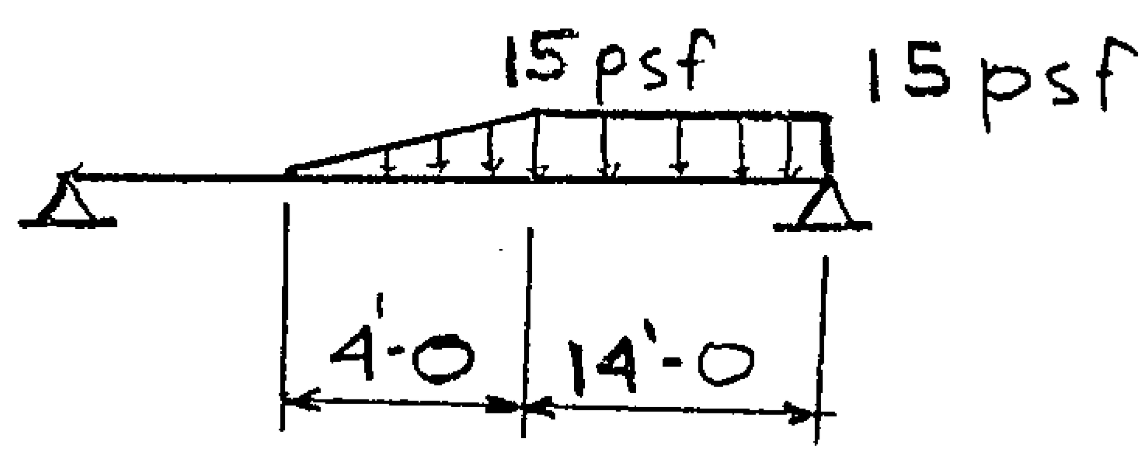
- SPAN: VARIES UP TO 40'-0
- SPACING: (32'-4) / 5 = 6'-5 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD: –
- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(6'-5 5/8) × (25 × 1.6 + 25 × 1.2) = 452 plf

- JOIST DEPTH: 28"

J11

- SPAN: ≈ 36'-10
- SPACING: (32'-4) / 5 = 6'-5 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(6'-5 5/8) × (25 × 1.6 + 25 × 1.2) = 452 plf

- JOIST DEPTH: 28"

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WESTGATE VILLAGE — Date MAR 4, 02 — Designed DT

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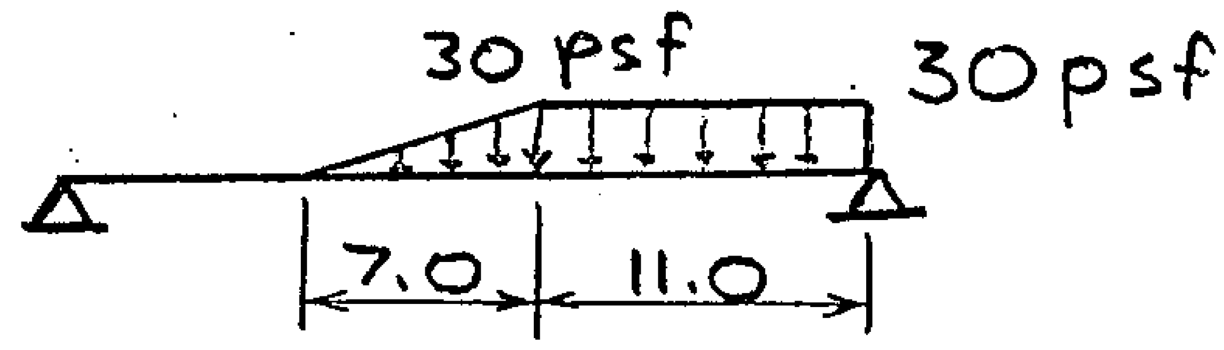
J12

- SPAN: ≃ 35'-0
- SPACING: $(32'-4)/5 = 6'\text{-}5\frac{5}{8}$
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



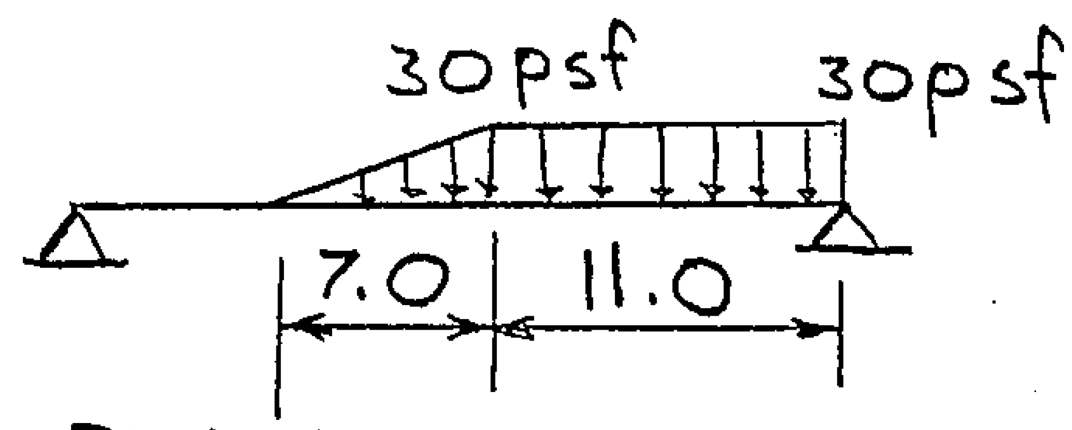
- ADD. MECH LOAD: SEE PLAN
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(6'\text{-}5\frac{5}{8}) \times (25 \times 1.6 + 25 \times 1.2) = 452$ plf

- JOIST DEPTH: 28"

J13

- SPAN: 33'-8
- SPACING: $(16'\text{-}6)/3 = 5'\text{-}6$
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: SEE PLAN
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'\text{-}6) \times (25 \times 1.6 + 25 \times 1.2) = 385$ plf

- JOIST DEPTH: 28"

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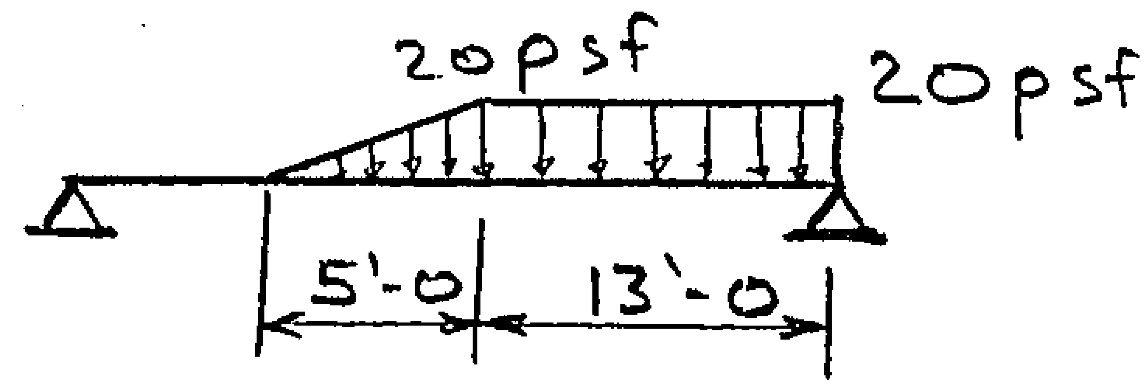
J14

- SPAN: 33'-8
- SPACING: (16'-6)/3 = 5'-6
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: ✓
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-6) × (25 × 1.6 + 25 × 1.2) = 385 plf

- JOIST DEPTH: 26"

J15

- SPAN: 33'-8
- SPACING: (16'-6)/3 = 5'-6
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD: ✓
- ADD. MECH LOAD: ✓
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-6) × (25 × 1.6 + 25 × 1.2) = 385 plf

- JOIST DEPTH: 26"

WESTGATE VILLAGE Date MAR 4, 02 Designed DT

Date Checked

$\boxed{J16}$

- SPAN: 33'-8
- SPACING: (20'-0)/4 = 5'-0
- SNOW LOAD: 40 psf
- ADD. SNOW LOAD: /
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-0) × (40 × 1.6 + 25 × 1.2) = 470 plf

- JOIST DEPTH: 26"

$\boxed{J17}$

- SPAN: 33'-8
- SPACING: (20'-0)/4/2 = 2'-6
- SNOW LOAD: 50 psf
- ADD. SNOW LOAD: /
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(2'-6) × (50 × 1.6 + 25 × 1.2) = 275 plf

- JOIST DEPTH: 24"

WESTGATE VILLAGE Date MAR 4, 02 Designed DT

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$\boxed{J18}$

- SPAN: 40'-0
- SPACING: (28'-2) /5/2 = 2'-9 13/16
- SNOW LOAD: 50 psf
- ADD. SNOW LOAD: –
- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(2'\text{-}9\tfrac{13}{16}) \times (50 \times 1.6 + 25 \times 1.2) = 310$ plf

- JOIST DEPTH: 26"

$\boxed{J19}$

- SPAN: 40'-0
- SPACING: (28'-2)/5 = 5'-7 5/8
- SNOW LOAD: 40 psf
- ADD. SNOW LOAD: –
- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'\text{-}7\tfrac{5}{8}) \times (40 \times 1.6 + 25 \times 1.2) = 530$ plf

- JOIST DEPTH: 28"

J20

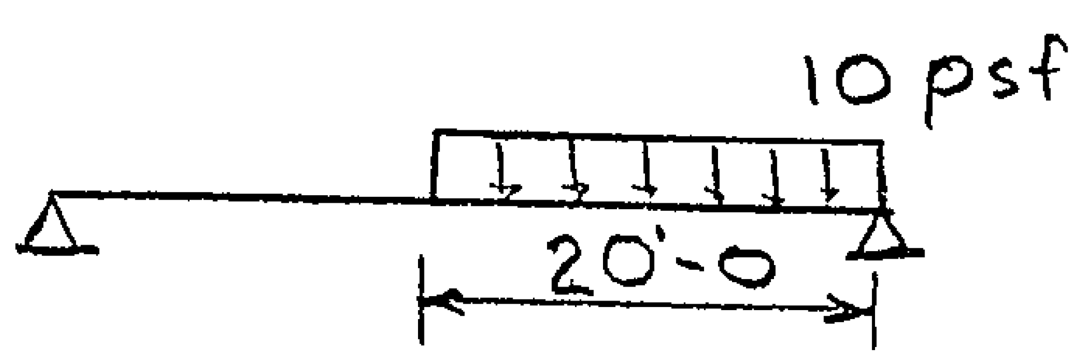
- SPAN: 40'-0
- SPACING: (28'-2)/5 $= 5'-7\frac{5}{8}$
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD: –
- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'-7\frac{5}{8}) \times (25 \times 1.6 + 25 \times 1.2) = 394$ plf

- JOIST DEPTH: 28"

J21

- SPAN: 40'-0
- SPACING: (28'-2)/5 $= 5'-7\frac{5}{8}$
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'-7\frac{5}{8}) \times (25 \times 1.6 + 25 \times 1.2) = 394$ plf

- JOIST DEPTH: 28"

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WESTGATE VILLAGE Date MAR 4, 02 Designed DT

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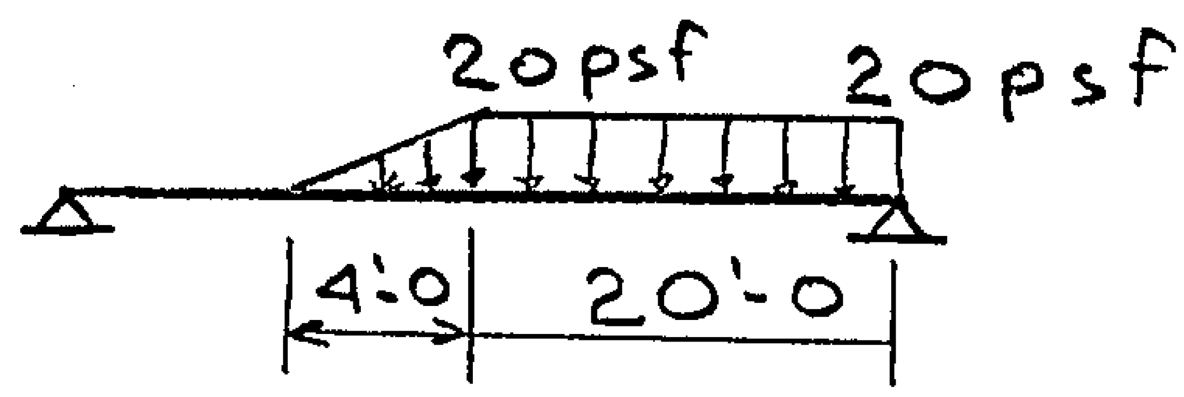
J22

- SPAN: 40'-0
- SPACING: (28'-2) / 5 = 5'-7 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



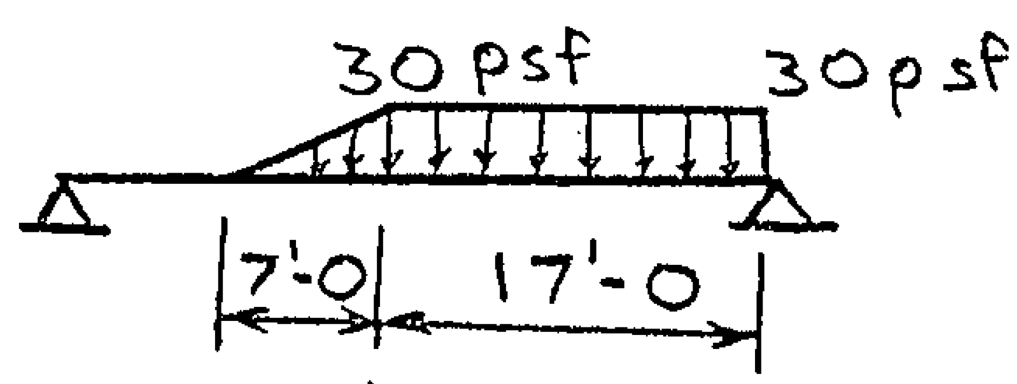
- ADD. MECH LOAD:
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-7 5/8) × (25 × 1.6 + 25 × 1.2) = 394 plf

- JOIST DEPTH: 28"

J23

- SPAN: 40'-0
- SPACING: (32'-4) / 5 = 6'-5 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: SEE PLAN
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(6'-5 5/8) × (25 × 1.6 + 25 × 1.2) = 452 plf

- JOIST DEPTH: 28"

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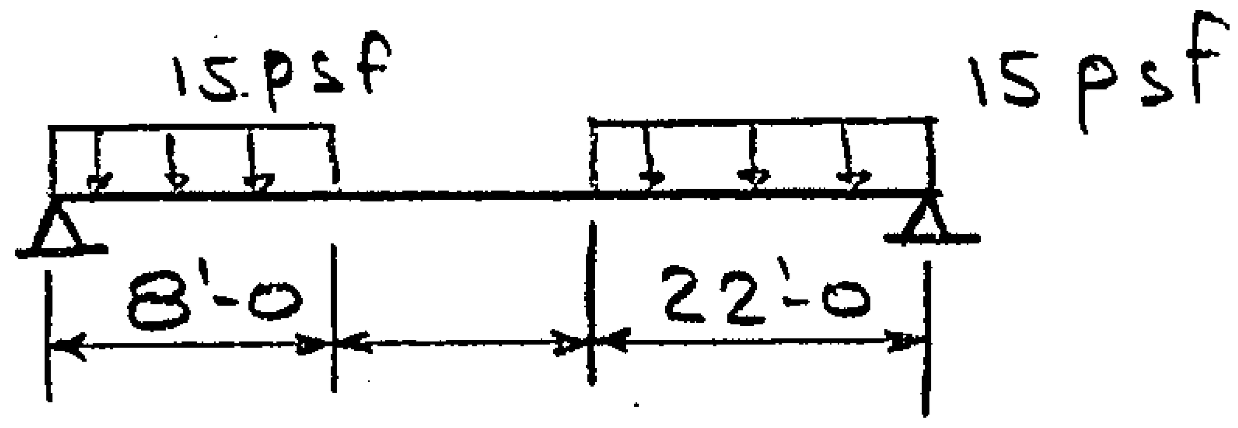
WESTGATE VILLAGE Date MAR 4, 02 Designed DT

Date Checked

J24

- SPAN: 40'-0
- SPACING: (32'-4) / 5 = 6'-5 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



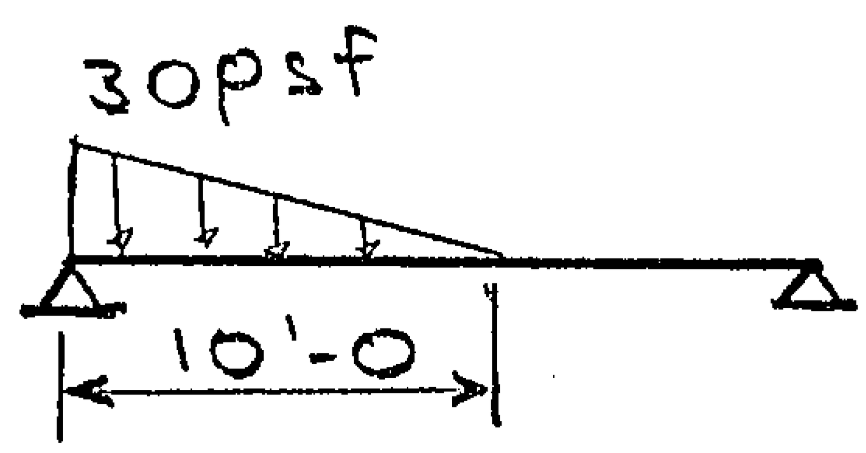
- ADD. MECH LOAD: ✓
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(6'-5 5/8) × (25 × 1.6 + 25 × 1.2) = 452 plf

- JOIST DEPTH: 28"

J25

- SPAN: 40'-0
- SPACING: (32'-4) / 5 = 6'-5 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: ✓
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(6'-5 5/8) × (25 × 1.6 + 25 × 1.2) = 452 plf

- JOIST DEPTH: 28"

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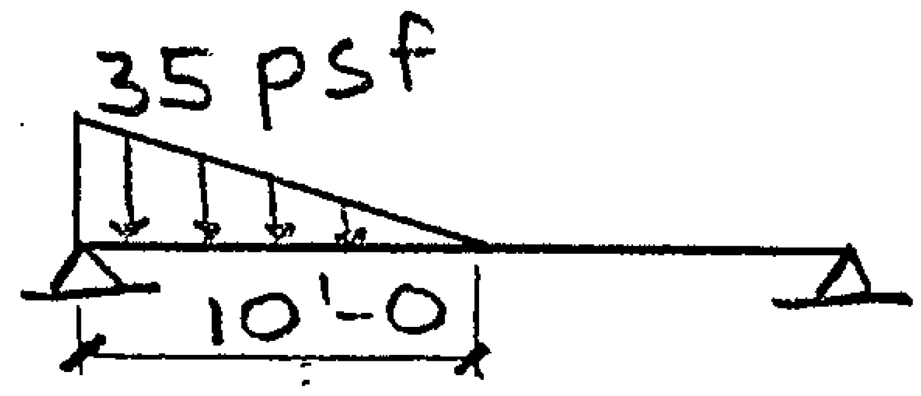
WESTGATE VILLAGE — Date MAR 4, 02 — Designed DT

Date ____ Checked ____

J26

- SPAN: 40'-0
- SPACING: (16'-6)/3 = 5'-6
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



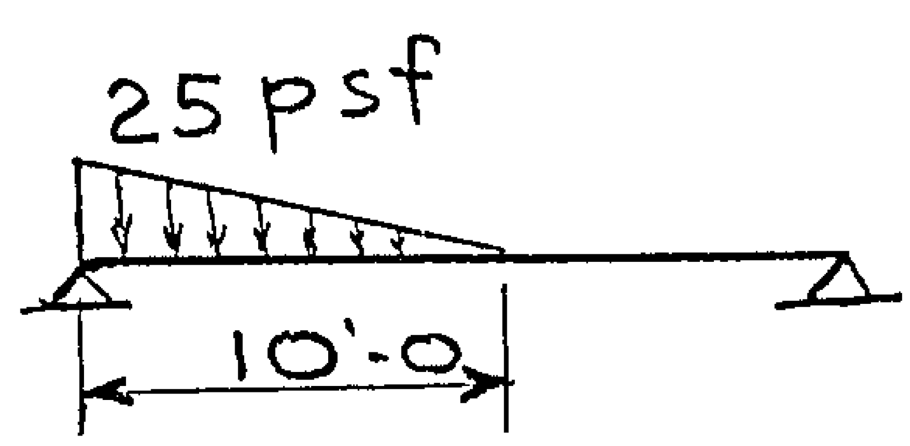
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-6) × (25 × 1.6 + 25 × 1.2) = 385 plf

- JOIST DEPTH: 28"

J27

- SPAN: 40'-0
- SPACING: (16'-6)/3 = 5'-6
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-6) × (25 × 1.6 + 25 × 1.2) = 385 plf

- JOIST DEPTH: 28"

WESTGATE VILLAGE Date MAR 4, 02 Designed DT

Date Checked

J28

- SPAN: 40'-0
- SPACING: (20'-0)/4 = 5'-0
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD: /
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'-0) \times (25 \times 1.6 + 25 \times 1.2) = 350$ plf

- JOIST DEPTH: 28"

J29

- SPAN: 40'-0
- SPACING: (20'-0)/4 = 5'-0
- SNOW LOAD: 40 psf
- ADD. SNOW LOAD: /
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'-0) \times (40 \times 1.6 + 25 \times 1.2) = 470$ plf

- JOIST DEPTH: 28"

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$\boxed{J30}$

- SPAN: 40'-0
- SPACING: (20'-0) /4/2= 2'-6
- SNOW LOAD: 50 psf
- ADD. SNOW LOAD: /
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(2'-6) \times (50 \times 1.6 + 25 \times 1.2) = 275$ plf

- JOIST DEPTH: 26"

$\boxed{J31}$

- SPAN: 23'-11 5/16
- SPACING: (28'-2) /5/2 = 2'-9 13/16
- SNOW LOAD: 50 psf
- ADD. SNOW LOAD: /
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(2'-9\tfrac{13}{16}) \times (50 \times 1.6 + 25 \times 1.2) = 310"$ plf

- JOIST DEPTH: 24"

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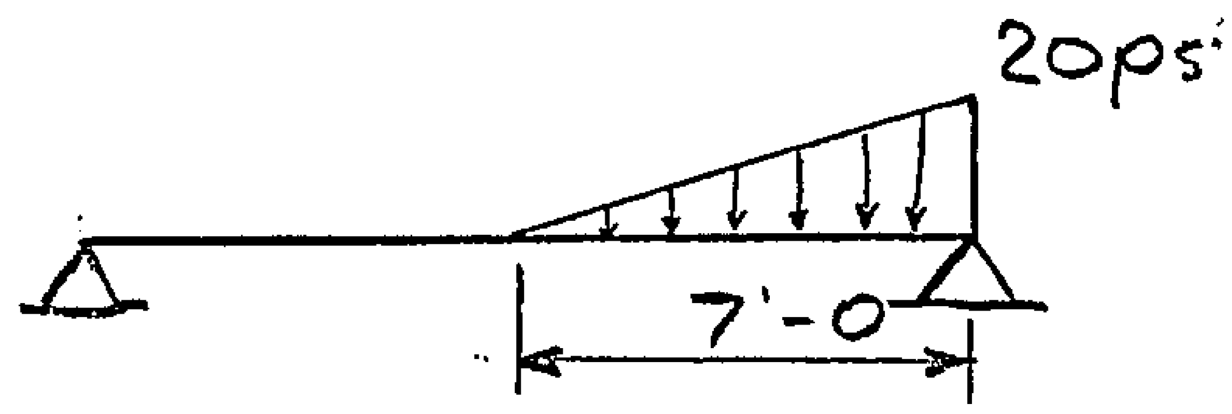
$\boxed{J32}$

- SPAN: 29'-1 9/16
- SPACING: (28'-2)/5 = 5'-7 5/8
- SNOW LOAD: 40 psf
- ADD. SNOW LOAD:



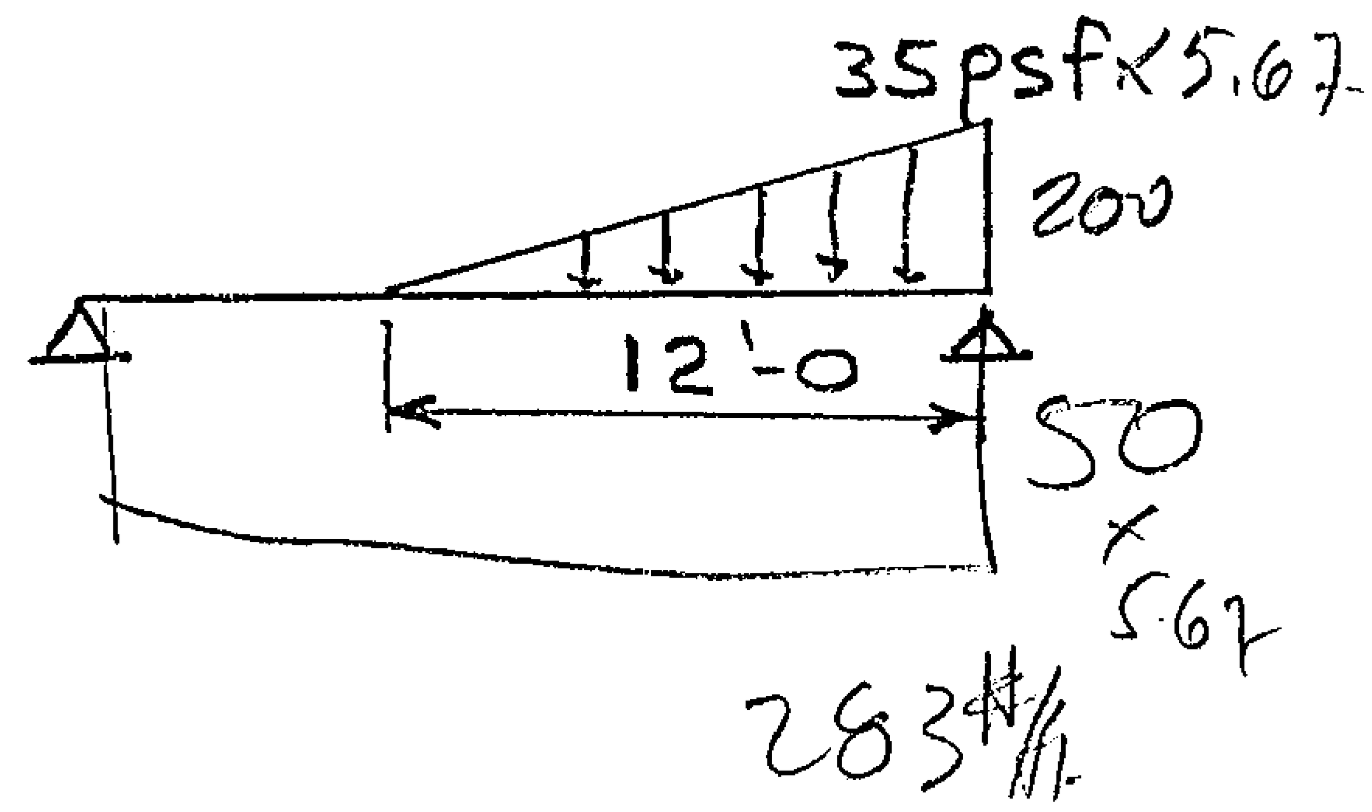
- ADD. MECH LOAD:
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-7 5/8) × (40 × 1.6 + 25 × 1.2) = 530 plf

- JOIST DEPTH: 26"

$\boxed{J33}$

- SPAN: 34'-3 3/4
- SPACING: (28'-2)/5 = 5'-7 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-7 5/8) × (25 × 1.6 + 25 × 1.2) = 394 plf

- JOIST DEPTH: 26"

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WESTGATE VILLAGE Date MAR 4, 02 Designed DT
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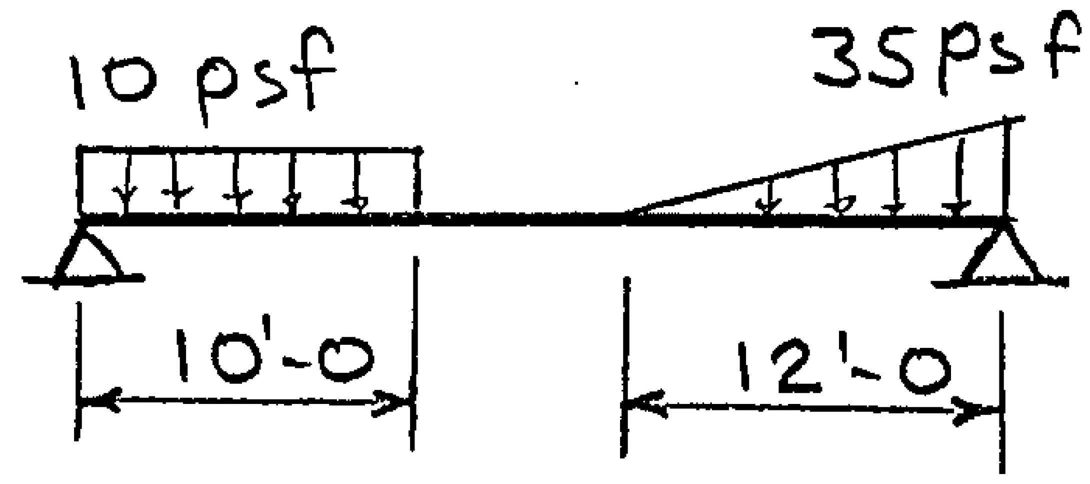
J34

- SPAN: 39'-6
- SPACING: (28'-2) / 5 = 5'-7 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



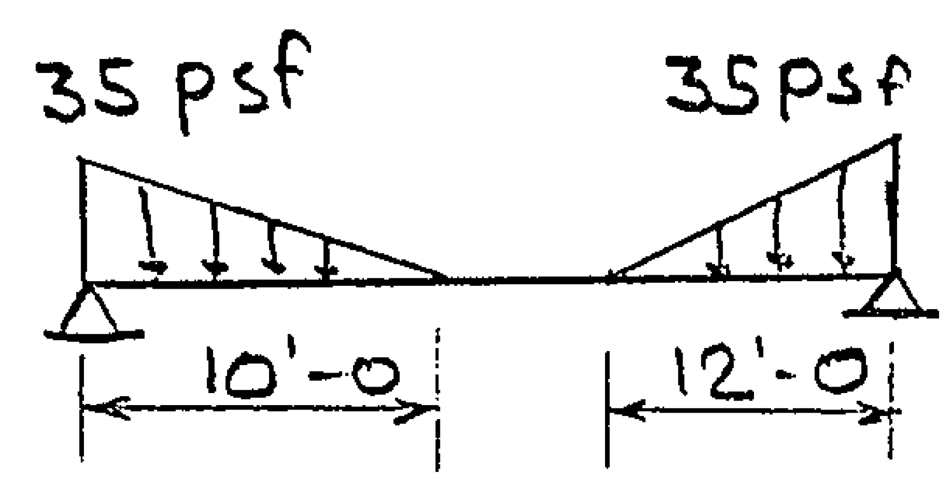
- ADD. MECH LOAD: ✓
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-7 5/8) × (25 × 1.6 + 25 × 1.2) = 394 plf

- JOIST DEPTH: 28"

J35

- SPAN: 39'-6
- SPACING: (28'-2) / 5 = 5'-7 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: ✓
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

(5'-7 5/8) × (25 × 1.6 + 25 × 1.2) = 394 plf

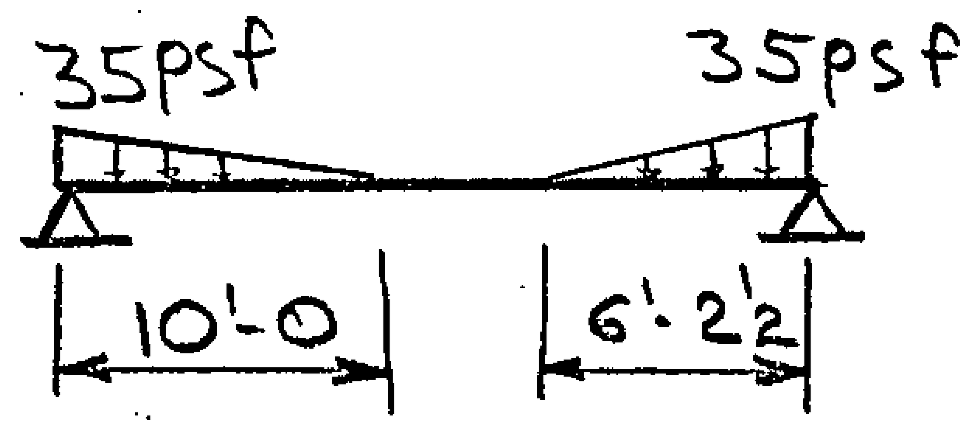
- JOIST DEPTH: 28"

WESTGATE VILLAGE Date MAR 4, 02 Designed DT

Date Checked

$\boxed{J36}$

- SPAN: 39'-6
- SPACING: (32'-4)/5 = 6'-5 5/8
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(6'\text{-}5\tfrac{5}{8}) \times (25 \times 1.6 + 25 \times 1.2) = 452$ plf

- JOIST DEPTH: 28"

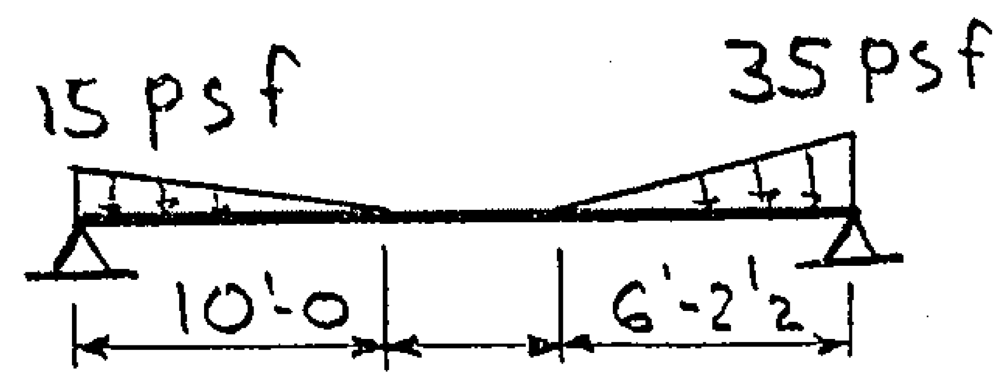
$\boxed{J37}$

- SPAN: 39'-6
- SPACING: (32'-4)/5 = 6'-5 5/8
- SNOW LOAD: 25 psf

187
54
25
29 × 6.45 = 187

60
-28
38 × 6.45
225

- ADD. SNOW LOAD:



- ADD. MECH LOAD: –
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(6'\text{-}5\tfrac{5}{8}) \times (25 \times 1.6 + 25 \times 1.2) = 452$ plf

- JOIST DEPTH: 28"

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WESTGATE VILLAGE Date MAR 4, 02 Designed DT

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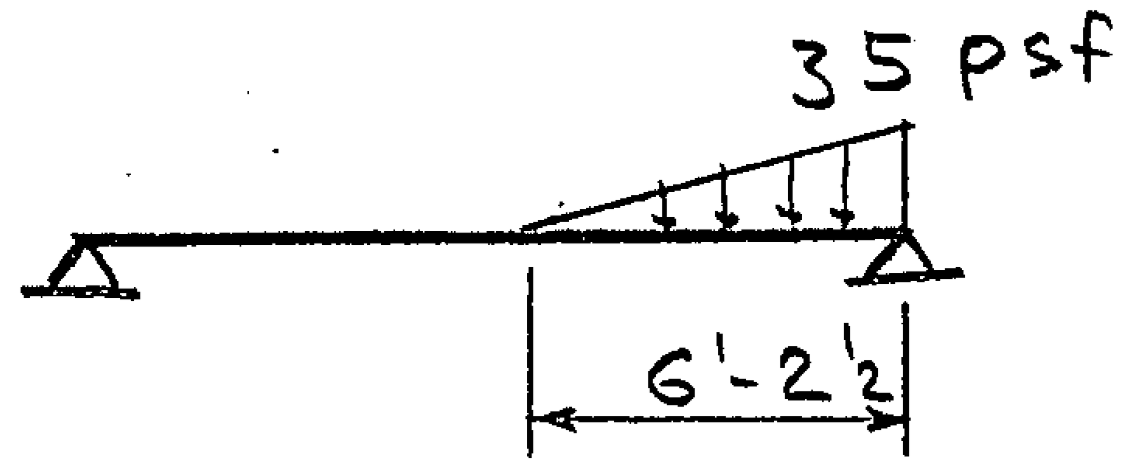
J38

- SPAN: 39'-6
- SPACING: (32'-4)/5 = 6'-5$\frac{5}{8}$
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



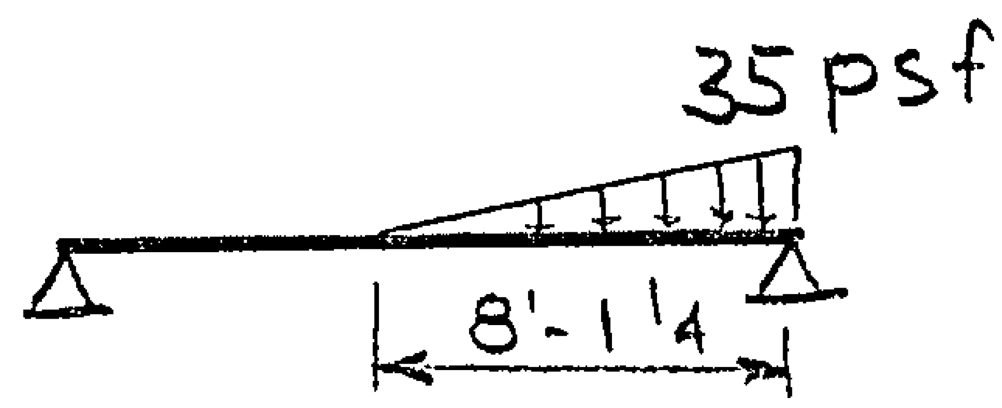
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(6'\text{-}5\tfrac{5}{8}) \times (25 \times 1.6 + 25 \times 1.2) = 452$ plf

- JOIST DEPTH: 28"

J39

- SPAN: 39'-6
- SPACING: (16'-6)/3 = 5'-6
- SNOW LOAD: 25 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'\text{-}6) \times (25 \times 1.6 + 25 \times 1.2) = 385$ plf

- JOIST DEPTH: 28"

WESTGATE VILLAGE Date MAR 4, 02 Designed DT

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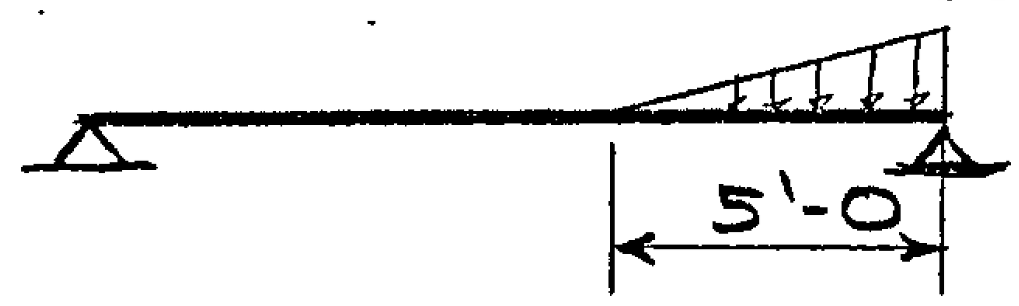
J40

- SPAN: 39'-6
- SPACING: (20'-0) / 4 = 5'-0
- SNOW LOAD: 40 psf
- ADD. SNOW LOAD:



- ADD. MECH LOAD:
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(5'\text{-}0) \times (40 \times 1.6 + 25 \times 1.2) = 470 \text{ plf}$

- JOIST DEPTH: 28"

J41

- SPAN: 39'-6
- SPACING: (20'-0)/4/2 = 2'-6
- SNOW LOAD: 50 psf
- ADD. SNOW LOAD: /
- ADD. MECH LOAD: /
- DECK: 22 GA
- JOIST FACTORED LINE LOAD:

$(2'\text{-}6) \times (50 \times 1.6 + 25 \times 1.2) = 275 \text{ plf}$

- JOIST DEPTH: 26"

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WESTGATE VILLAGE Date MAR 5, 02 Designed DT

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JOIST GIRDERS

JG1

28.5 | 33.67

155.4

$P = \left(\frac{28.5+33.67}{2}\right)(5)(50+25)$

$= 7.77^k + 3.89^k = 11.66^k$

- SPAN: 20'-0
- POINT LOADS: $P_{(1,2,3)D}$ = 2177 + 2104 = 4281 lbs

$P_{(1)L}$ = 3483 + 3366 = 6849 lbs

? $P_{(2,3)L}$ = 2177 + 2104 = 4281 lbs

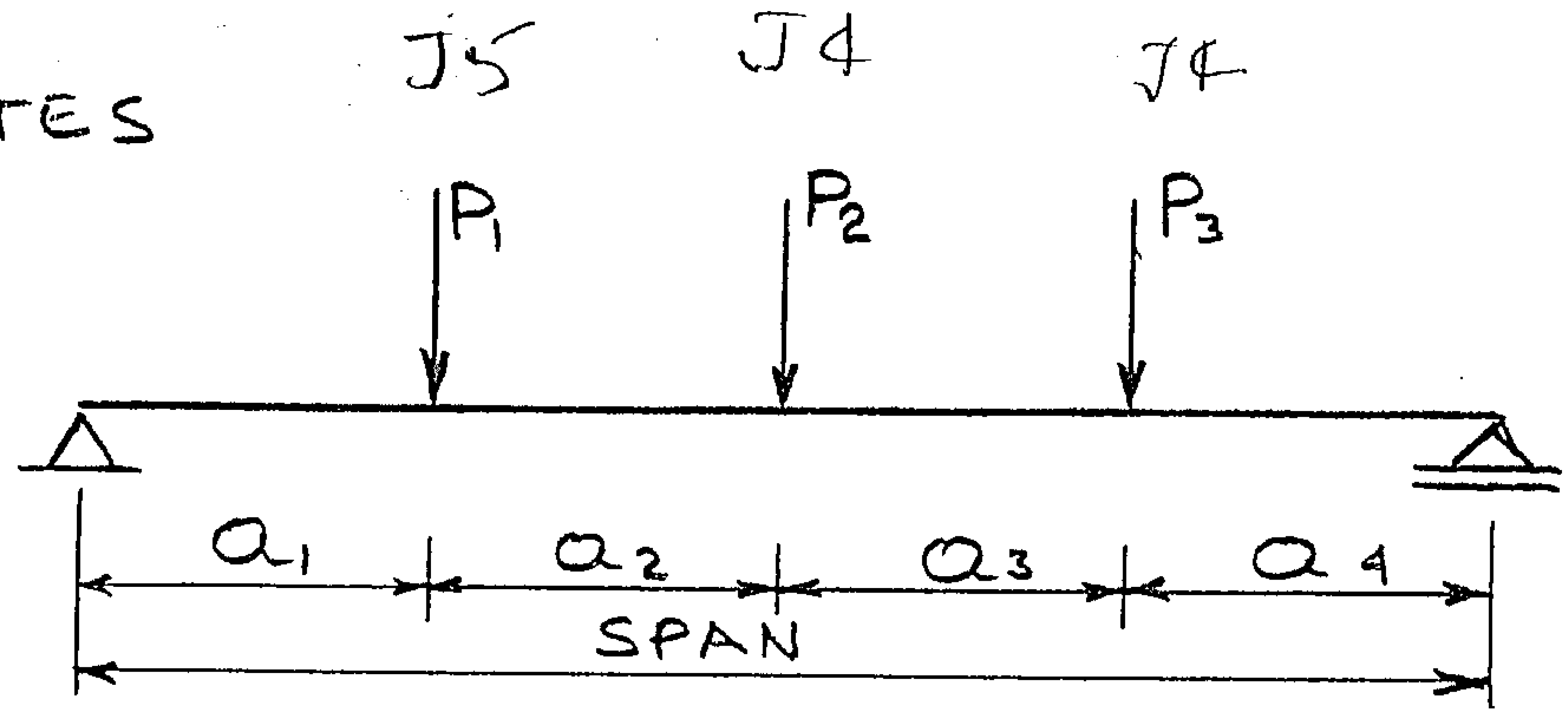
- SPACING BETWEEN LOAD POINTS:

$a_{(1,2,3,4)}$ = 5'-0

- JOIST GIRDER DEPTH: 30"

5(50 + 25) =
250 + 125
375#/ft
× 28.5/2 =
5.43^k × 2
10.86

* NOTES



WESTGATE VILLAGE Date MAR 5, 02 Designed DT

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$\boxed{JG2}$

- SPAN: 21'-6
- POINT LOADS:

$$P_{(1,2,3)D} = 2394 + 2314 = 4708 \text{ lbs}$$

$$P_{(1,2,3)L} = 2394 + 2314 = 4708 \text{ lbs}$$

- SPACING: 5'-6
- JOIST GIRDER DEPTH: 30"

$\boxed{JG3}$

- SPAN: 35'-8
- POINT LOADS:

$$P_{(1,2,3,4,5)D} = 2340 + 2778 = 5118 \text{ lbs}$$

$$P_{(1,2,3,4)L} = 2340 + 2778 = 5118 \text{ lbs}$$

$$P_{(5)L} = 3924 + 3788 \; 7712 \text{ lbs}$$

- SPACING: 5'-10 13/16
- JOIST GIRDER DEPTH: 40"

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Date Checked

JG4

- SPAN: 25'-5
- POINT LOADS:

$$P_{(1,2,3,4)D} = 1810 + 2542 = 4352 \text{ lbs}$$

$$P_{(1,2,3)L} = 4346 + 3262 = 7608 \text{ lbs}$$

$$P_{(4)L} = 4346 + 4582 = 8928 \text{ lbs}$$

- SPACING: 5'-1
- JOIST GIRDER DEPTH: 36"

JG5

- SPAN: 36'-6
- POINT LOADS:

$$P_{(1,2,3)D} = 2104 + 2500 = 4604 \text{ lbs}$$

$$P_{(4,5,6)D} = 2314 + 2750 = 5064 \text{ lbs}$$

$$P_{(1)L} = 3366 + 4000 = 7366 \text{ lbs}$$

$$P_{(2,3)L} = 2104 + 2500 = 4604 \text{ lbs}$$

$$P_{(4)L} = 2314 + 2750 = 5064 \text{ lbs}$$

$$P_{(5)L} = 4019 + 3437 = 7456 \text{ lbs}$$

$$P_{(6)L} = 4706 + 3712 = 8418 \text{ lbs}$$

- SPACING: $a_{(1,2,3,4)} = 5'\text{-}0$; $a_{(5,6,7)} = 5'\text{-}6$
- JOIST GIRDER DEPTH: 40"

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JG6

- SPAN: 37'-4

- POINT LOADS:

$P_{(1)D} = 2413 + 2867 = 5280$ Lbs

$P_{(2,3,4,5)D} = 3072 + 3234 = 6306$ Lbs

$P_{(1)L} = 4907 + 3871 = 8778$ Lbs

$P_{(2)L} = 5886 + 4204 = 10090$ Lbs

$P_{(3)L} = 4624 + 4010 = 8634$ Lbs

$P_{(4,5)L} = 3072 + 3234 = 6306$ Lbs

- SPACING: $a_{(1)} = 5'-0$; $a_{(2,3,4,5,6)} = 6'-5\frac{5}{8}$

- JOIST GIRDER DEPTH: 40"

JG7

- SPAN: 28'-2

- POINT LOADS:

$P_{(1,2,3,4)D} = 2818 + 2818 = 5636$ Lbs

$P_{(1,2,3)L} = 2818 + 2818 = 5636$ Lbs

$P_{(4)L} = 4508 + 4508 = 9016$ Lbs

- SPACING: $5'-7\frac{5}{8}$

- JOIST GIRDER DEPTH: 36"

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JG8 - SPAN: 36'-6

- POINT LOADS:

$P_{(1,2,3)D} = 2500 + 2468 = 4968$ lbs

$P_{(4,5,6)D} = 2750 + 2716 = 5466$ lbs

$P_{(1)L} = 4000 + 3950 = 7950$ lbs

$P_{(2,3)L} = 2500 + 2468 = 4968$ lbs

$P_{(4,5,6)L} = 2750 + 2716 = 5466$ lbs

- SPACING: $a_{(1,2,3,4)} = 5'-0$; $a_{(5,6,7)} = 5'-6$

- JOIST GIRDER DEPTH: 40"

JG9 - SPAN: 37'-4

- POINT LOADS:

$P_{(1)D} = 2867 + 2831 = 5698$ lbs

$P_{(2,3,4,5)D} = 3234 + 3194 = 6428$ lbs

$P_{(1)L} = 2867 + 2831 = 5698$ lbs

$P_{(2)L} = 3234 + 3194 = 6428$ lbs

$P_{(3)L} = 5368 + 3678 = 9046$ lbs

$P_{(4,5)L} = 7212 + 4326 = 11538$ lbs

- SPACING: $a_{(1)} = 5'-0$; $a_{(2,3,4,5,6)} = 6'-5\frac{5}{8}$

- JOIST GIRDER DEPTH: 40"

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$\boxed{JG10}$ - SPAN: 28'-2

- POINT LOAD:

$P_{(1,2)D} = 2818 + 2782 = 5600$ lbs

$P_{(3)D} = 2818 + 2418 = 5236$ lbs

$P_{(4)D} = 2818 + 2052 = 4870$ lbs

$P_{(1)L} = 5298 + 3768 = 9066$ lbs

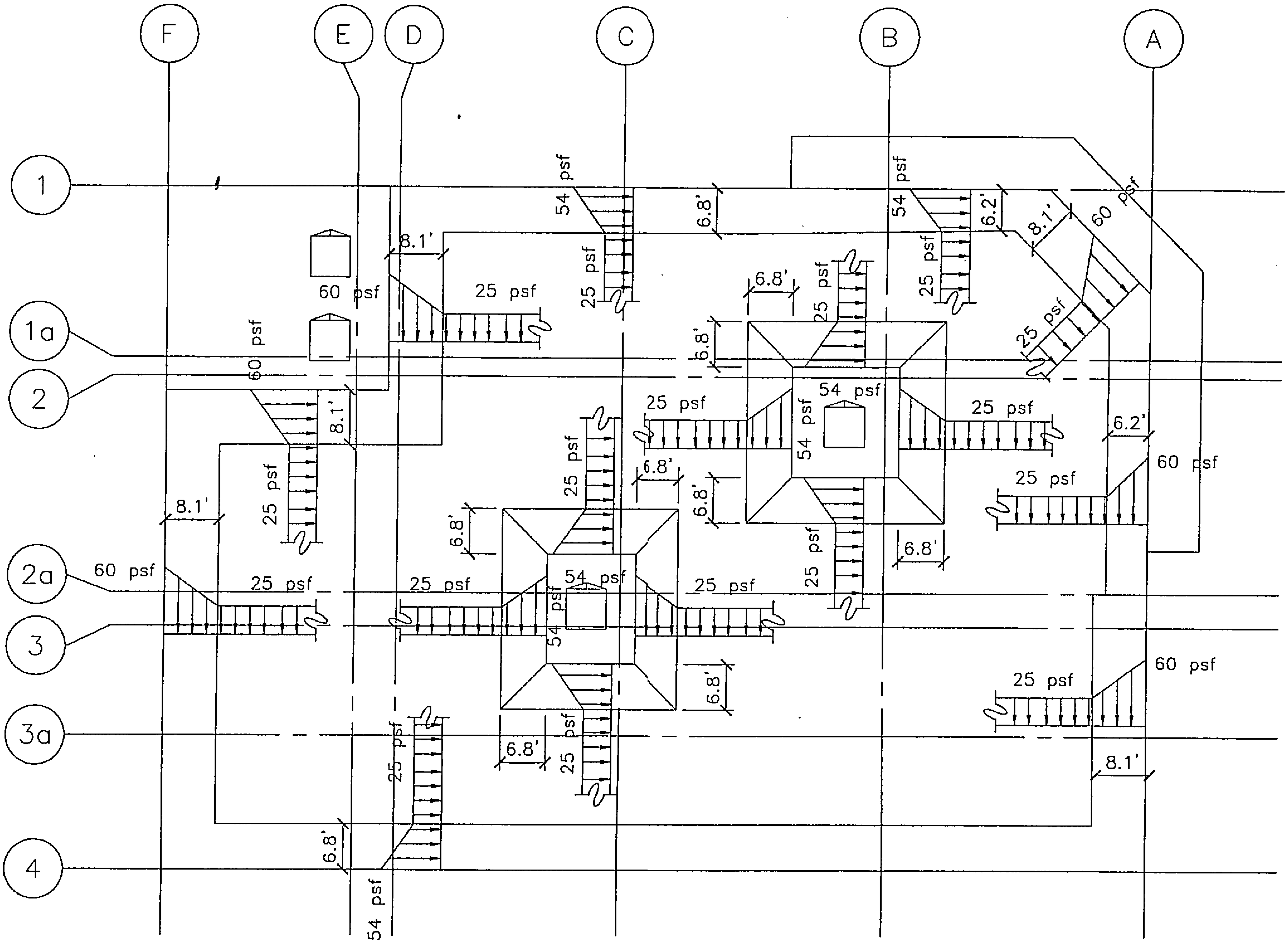
$P_{(2)L} = 3945 + 3345 = 7290$ lbs

$P_{(3)L} = 2818 + 2418 = 5236$ lbs

$P_{(4)L} = 4508 + 3284 = 7792$ lbs

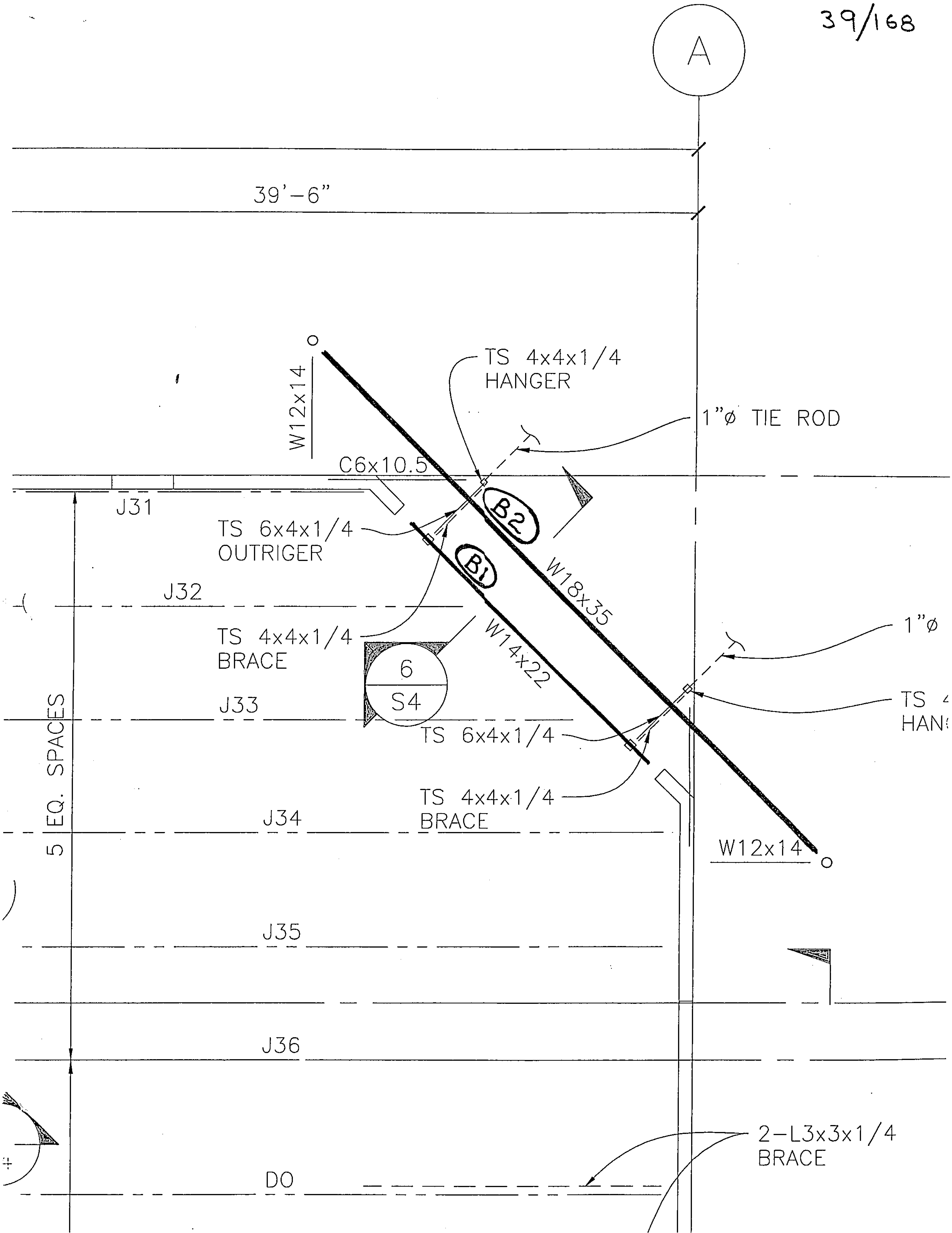
- SPACING: 5'-7 5/8

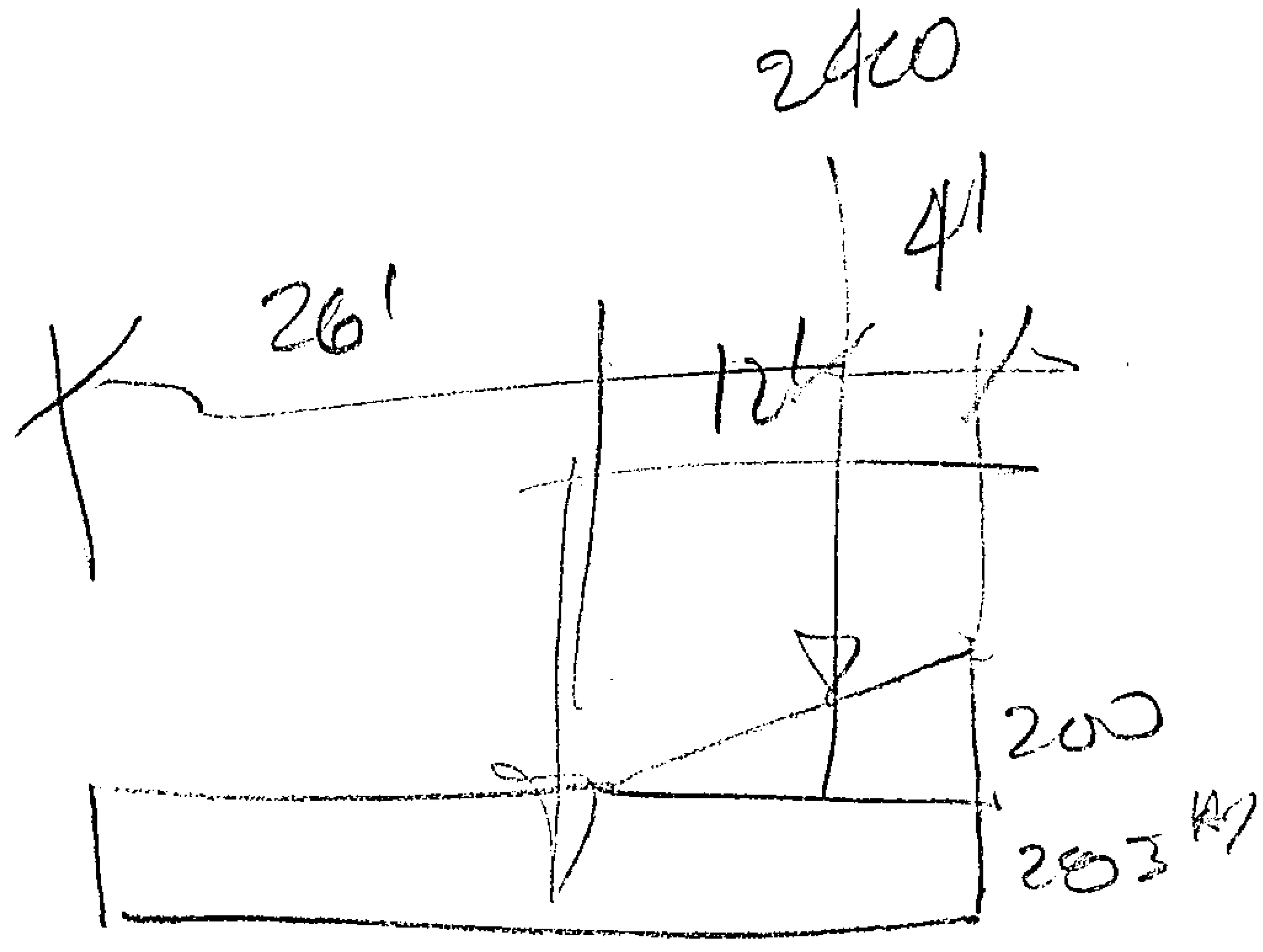
- JOIST GIRDER DEPTH: 36"



SNOW LOAD DIAGRAM

1'=20'-0"


A
39'-6"
W12x14
TS 4x4x1/4
HANGER
1"ø TIE ROD
C6x10.5
J31
B2
TS 6x4x1/4
OUTRIGER
B1
W18x35
J32
TS 4x4x1/4
BRACE
W14x22
1"ø
6
S4
J33
TS 6x4x1/4
TS 4
HAN
5 EQ. SPACES
TS 4x4x1/4
BRACE
J34
W12x14
J35
J36
2-L3x3x1/4
BRACE
DO



30'

~~8.5~~
4252#
320
(4961)

(35)

~~8.505~~
4253#
2080
6333
(4961)
2486
7888

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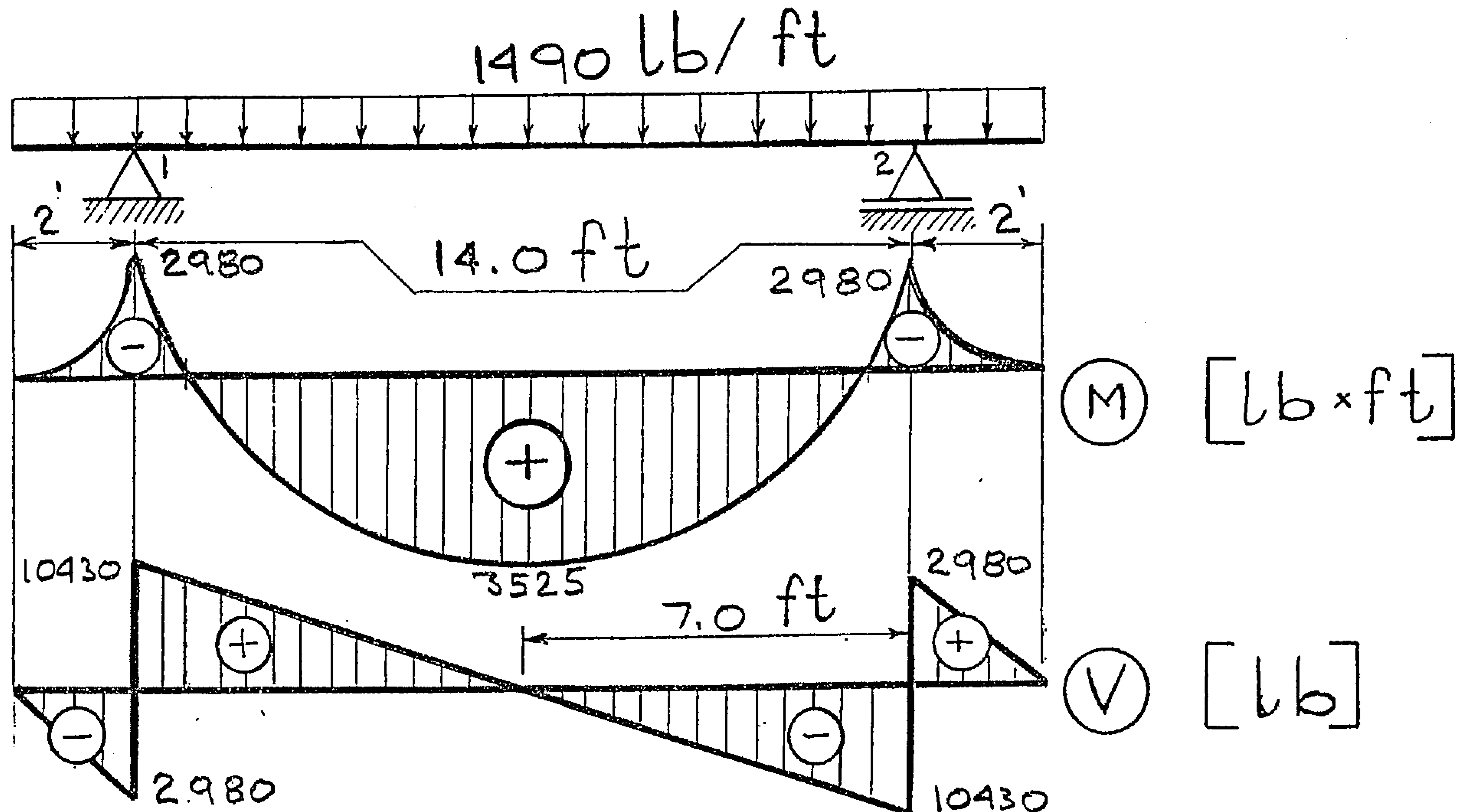
CANOPY FRAMING

B1

$$\omega = 23 \times (\overset{D.L.}{25} + \overset{L.L.}{25}) + \overset{ACC.\ SNOW}{8 \times 35 \times 0.5} + \overset{WALL}{200} = 1490 \ lb/f$$



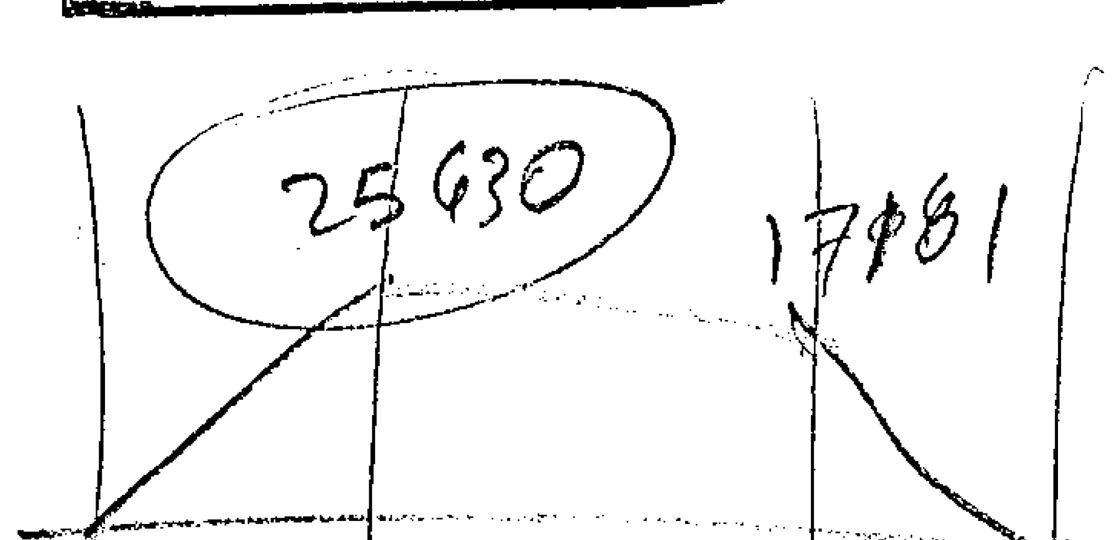
$M_{MIN} = 1490 \times 2^2/2 = 2980 \ lb \times ft$

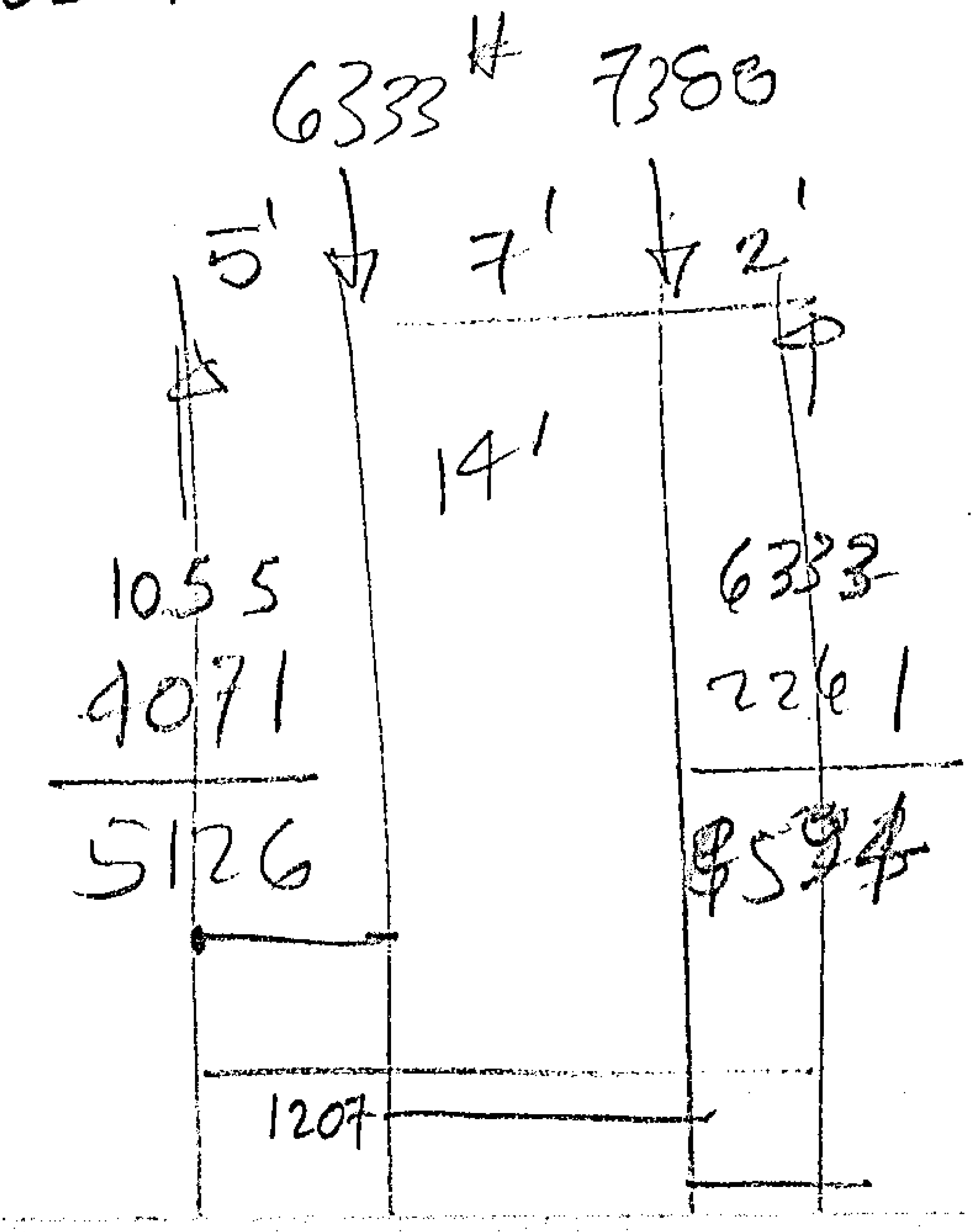
$M_{MAX} = 1490 \times 14^2/8 - 2980 = 33525 \ lb \times ft$

$R = 1440 \times 18/2 = 13410 \ lbs$

$S_{reqd} = \frac{33.6 \times 12}{30} = 13.44 \ in^3$ ok

USE $\boxed{W14 \times 22}$ ($29.0 \ in^3$)





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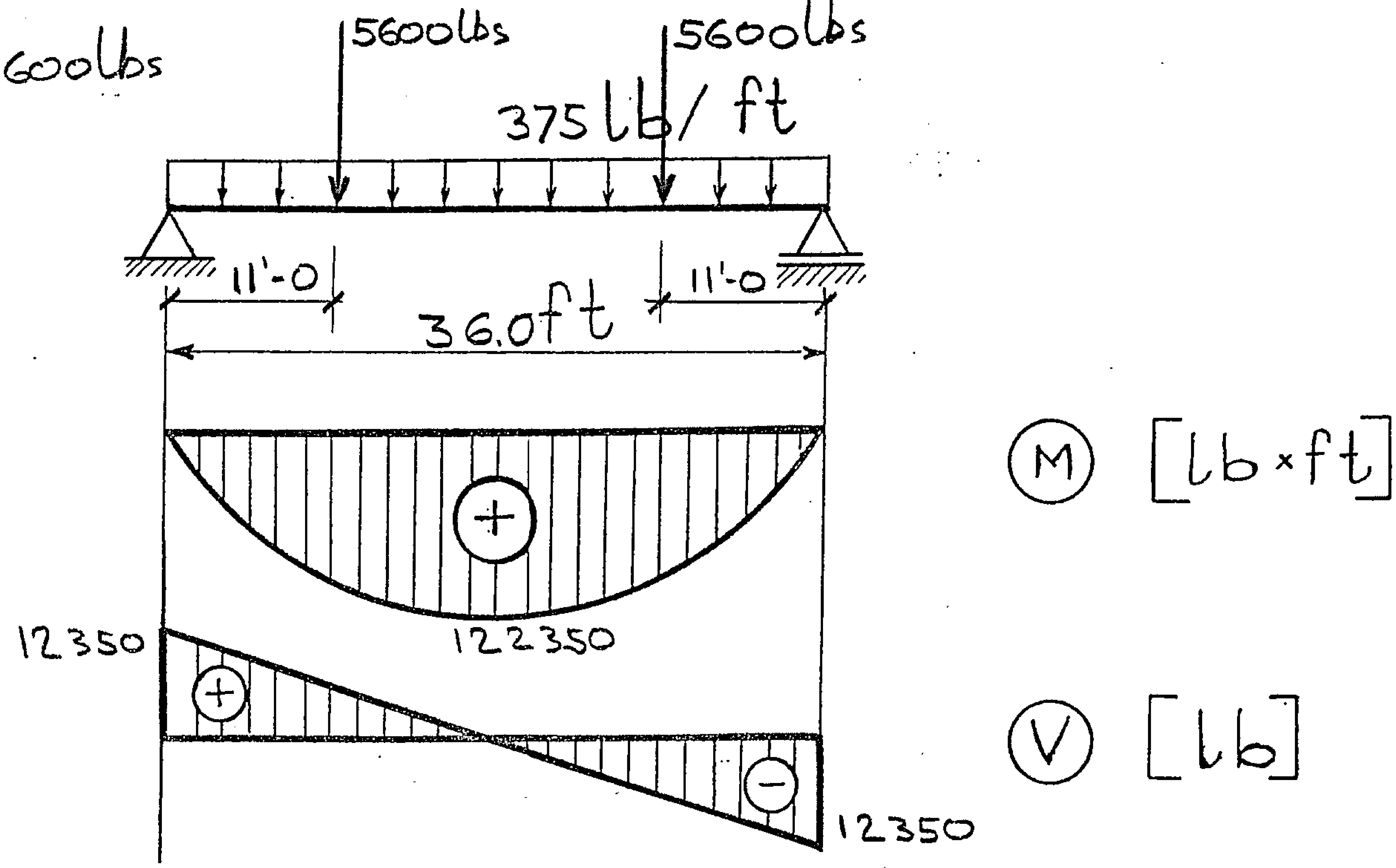
B2

$\omega = 5 \times (\underbrace{20}_{D.L.} + \underbrace{25}_{SNOW}) + \underbrace{150}_{WALL} = 375$ lb/f

P' = 5600lbs

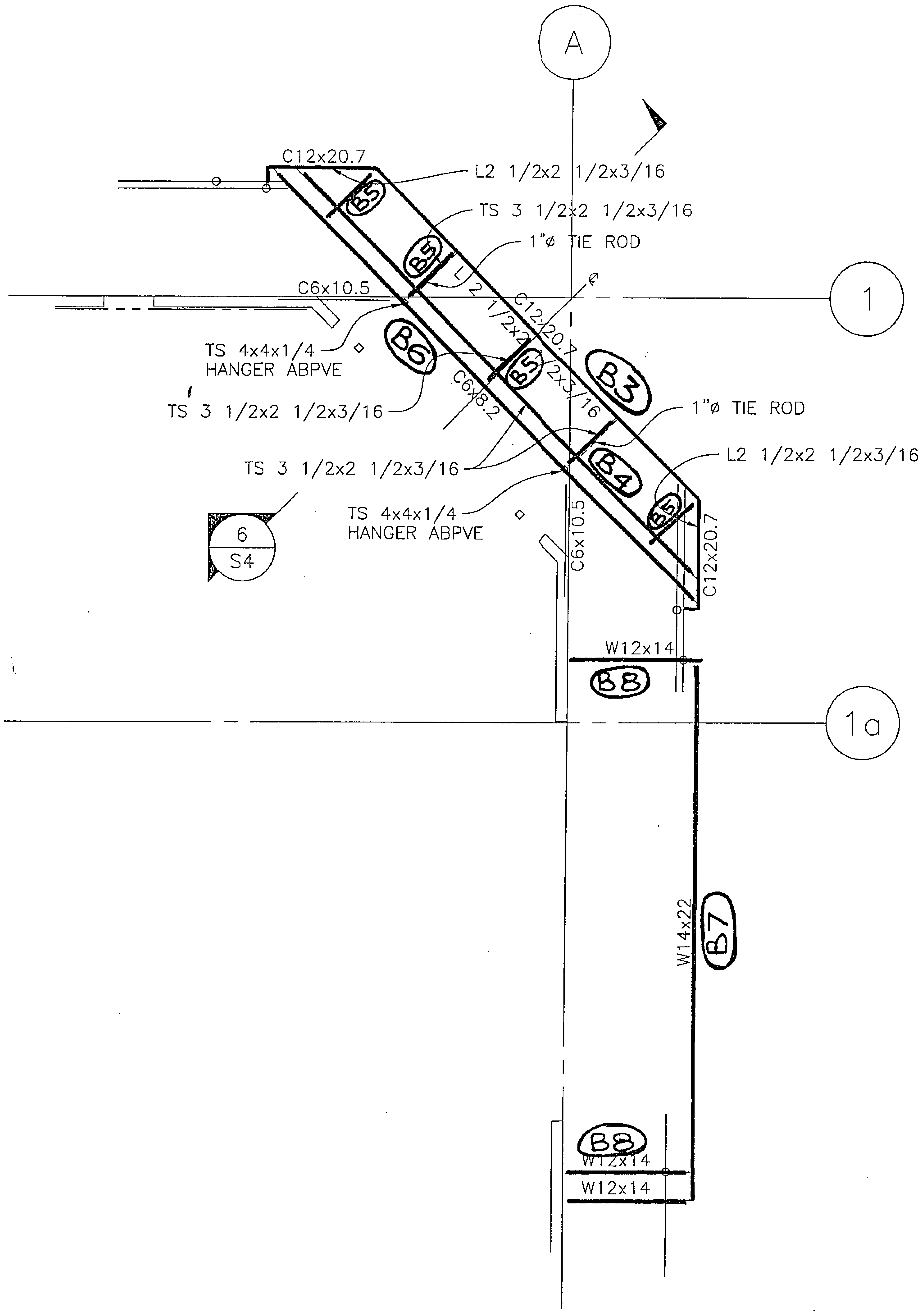


$$M = \frac{375 \times 36^2}{8} + 11 \times 5600 = 122350 \text{ lb×ft}$$

$$V = \frac{375 \times 36 + 2 \times 5600}{2} = 12350 \text{ lbs}$$

$$S_{reqd} = \frac{122.5 \times 12}{30} = 49 \text{ in}^3$$

USE $\boxed{W\ 18 \times 35}$ (57.6 in³)



PLAN ON FRAMING AT BASE
OF TIE ROD (CANOPY LEVEL)

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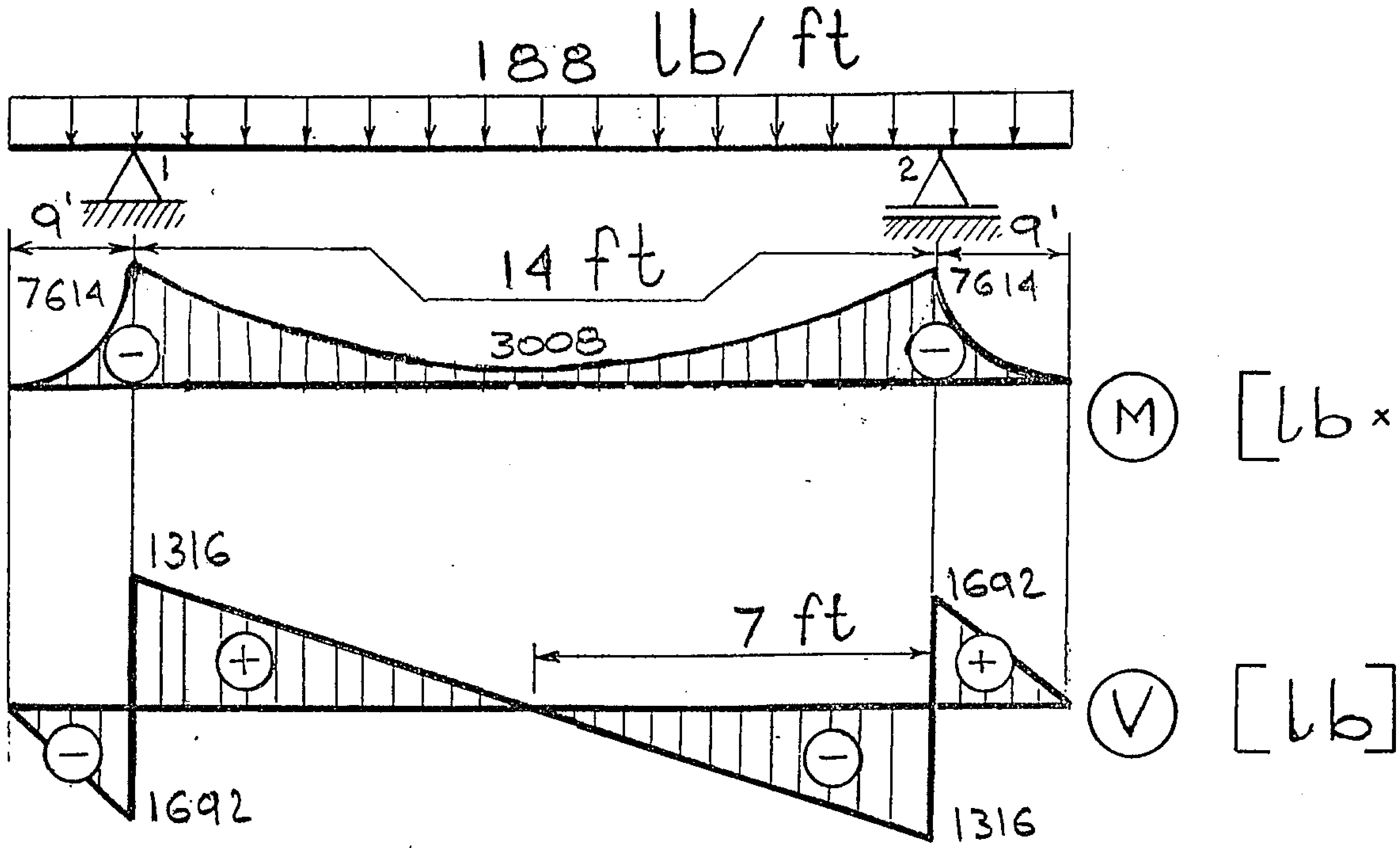
B3

$\omega = 2.5 \times (25 + 50) = 188$ lb/f



$M_{MIN} = 188 \times 9^2/2 = 7614$ lb×ft

$M_{MAX} = 188 \times 14^2/8 - 7614 = -3008$ lb×ft

(BEAM BOTT. CORD IS CONSTANTLY COMPRESSED, IT HAS TO BE BRACED)

$R = 188 \times 16 = 3008$ lbs

$S_{REQ'D} = \dfrac{8 \times 12}{30} = 3.2\ in^3$

USE $\boxed{C\ 12 \times 20.7}$ (21.5 in³) ARCH. REASONS

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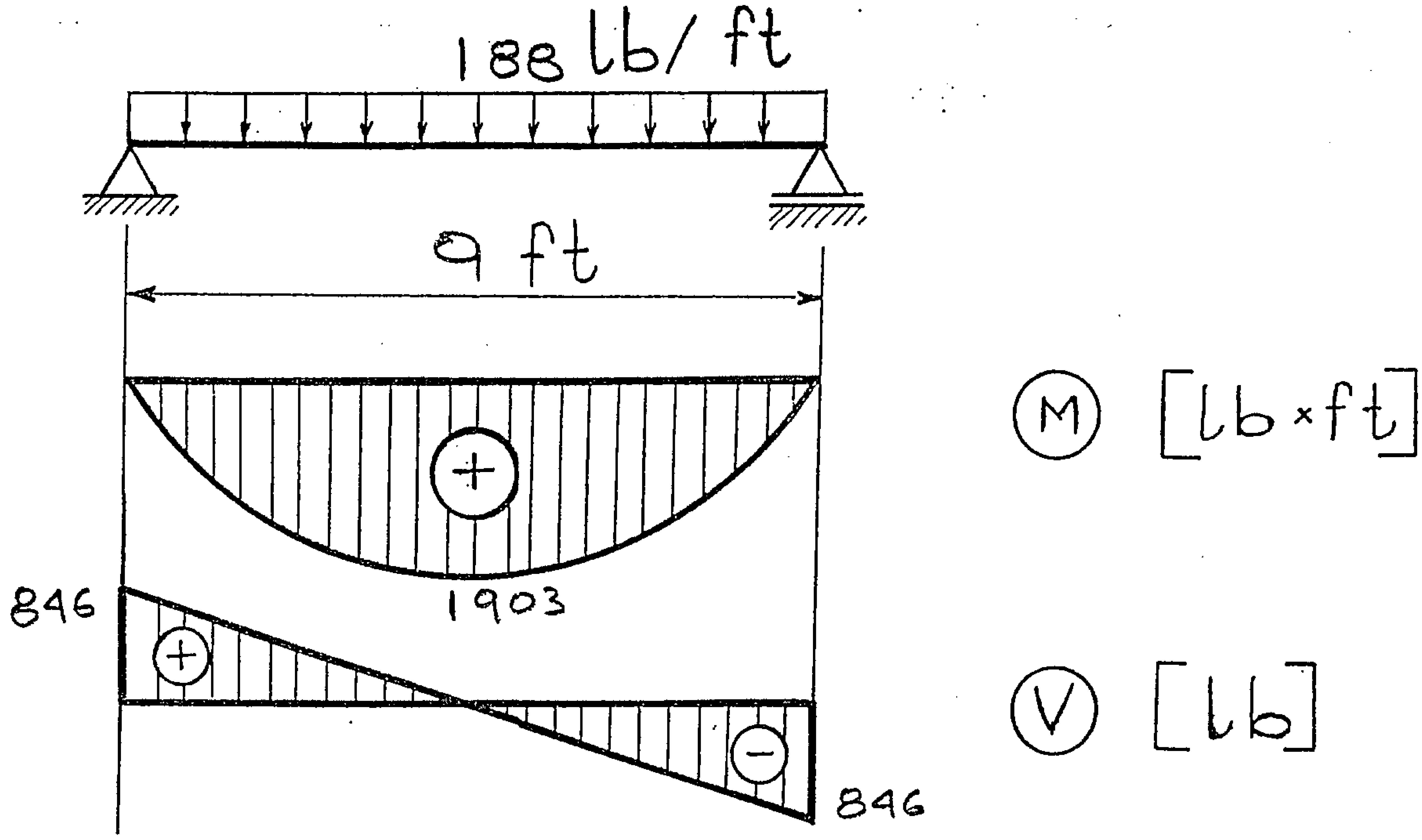
B4

$\omega = 188$ lb × ft



$$M = \frac{188 \times 9^2}{8} = 1903 \text{ lb} \times \text{ft}$$

$$V = \frac{188 \times 9}{2} = 846 \text{ lbs}$$

$$S_{reqd} = \frac{2 \times 12}{30} = 0.8 \text{ in}^3$$

USE $\boxed{\text{TS } 3\tfrac{1}{2} \times 2\tfrac{1}{2} \times \tfrac{3}{16}}$ (1.86 in³)

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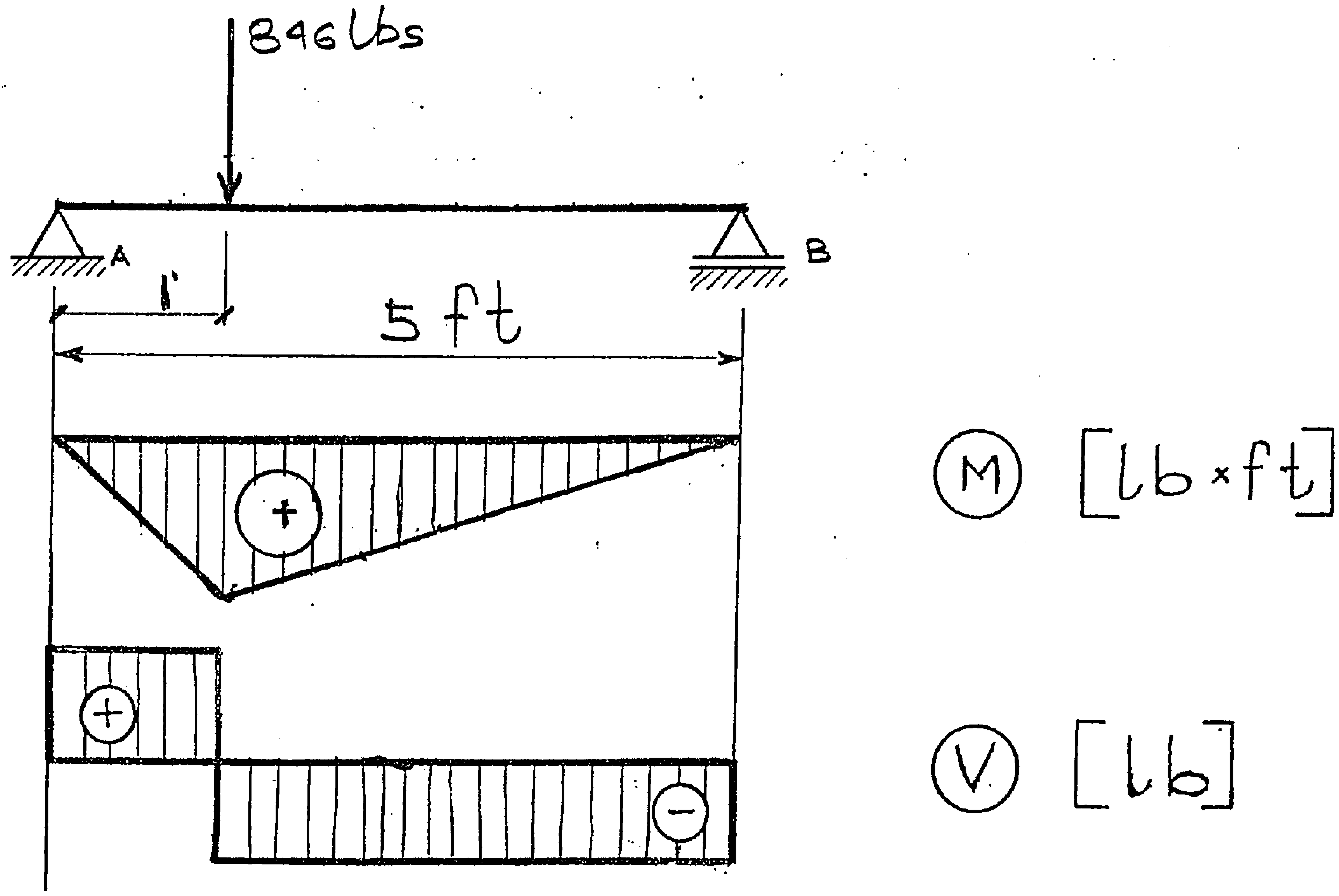
$\boxed{B5}$

P = 846 lbs



$$R_A = \frac{864 \times 4}{5} = 677 \text{ lbs}$$

$$M = 677 \times 1.0 = 677 \text{ lb} \times \text{ft}$$

$$S_{reqd} = \frac{1 \times 12}{30} = 0.4 \text{ in}^3$$

.303 for L $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{16}$

USE $\boxed{TS\ 3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{16}}$ (1.86 in^3)

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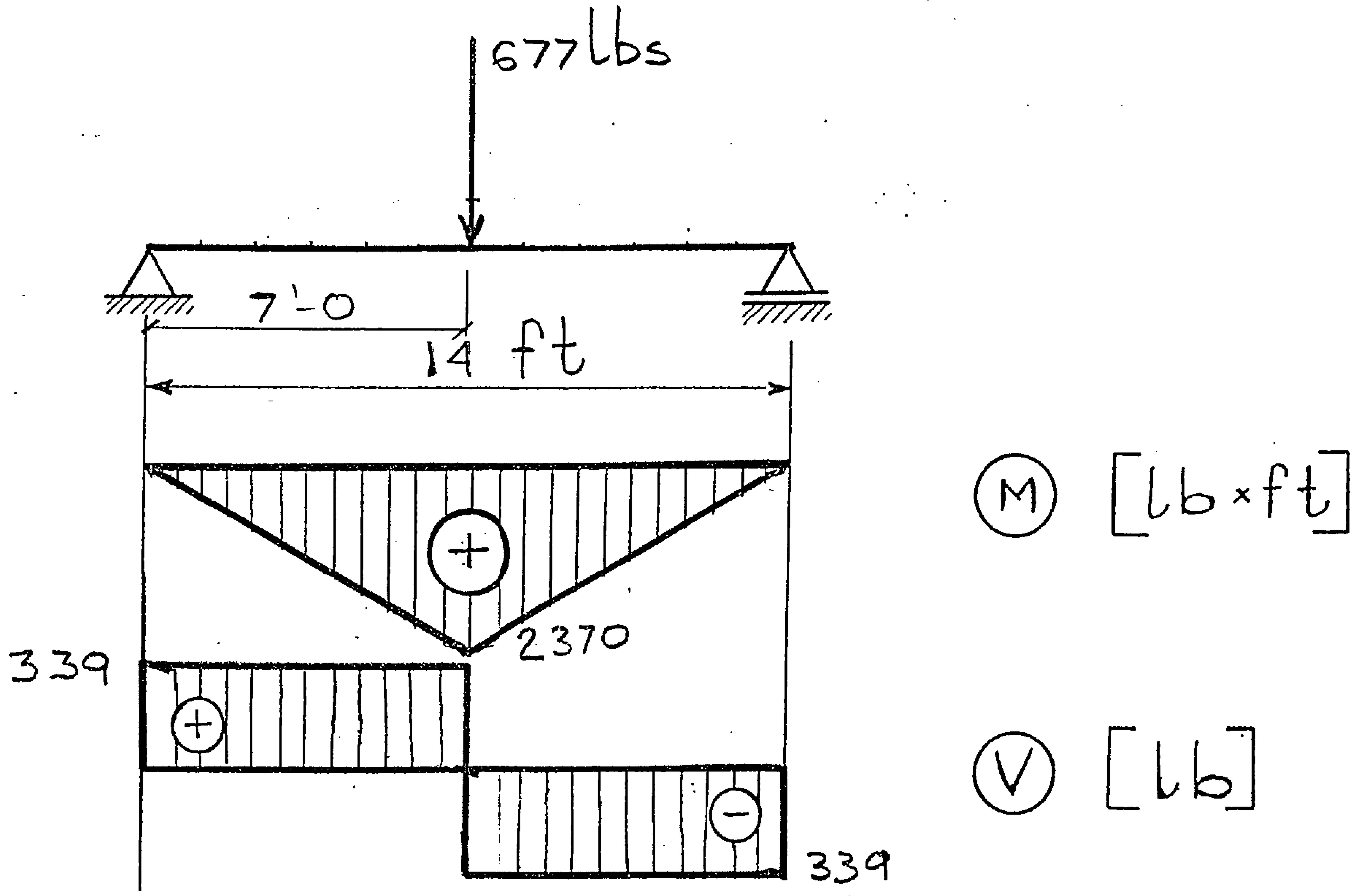
B6

P = 677 lbs



$$M = \frac{677 \times 14}{4} = 2370 \quad lb \times ft$$

$$V = \frac{677}{2} = 339 \quad lb$$

$$S_{reqd} = \frac{2.5 \times 12}{30} = 1.0 \ in^3$$

USE $\boxed{C\ 6 \times 8.2}$ ($4.38 in^3$)

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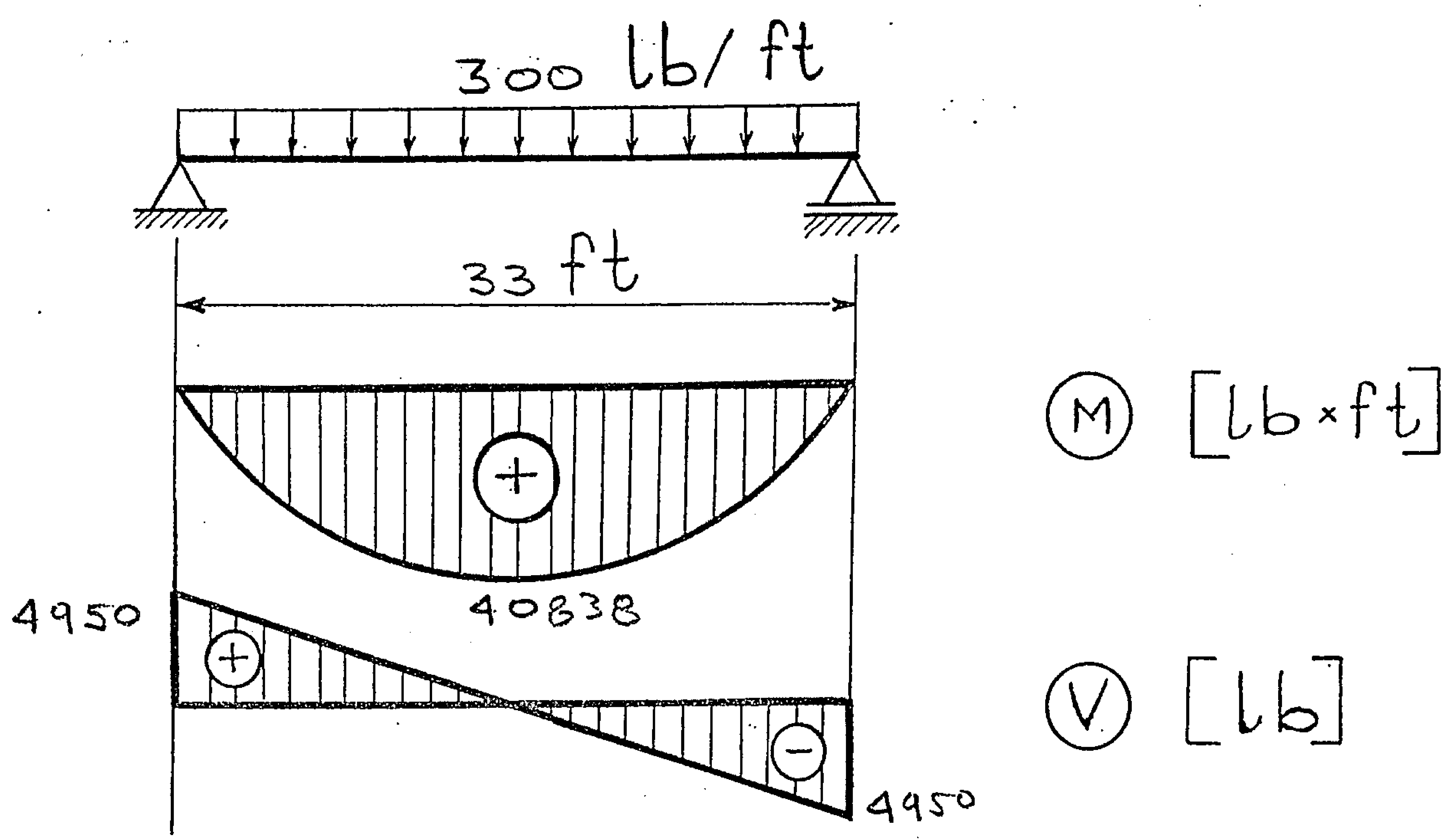
B7

$$\omega = 4 \times (\underbrace{25}_{\text{D.L.}} + \underbrace{50}_{\text{SNOW}}) = 300 \ lb/f$$



$$M = \frac{300 \times 33^2}{8} = 40838 \ lb \times ft$$

$$V = \frac{300 \times 33}{2} = 4950 \ lb$$

$$S_{reqd} = \frac{41 \times 12}{30} = 16.4 \ in^3$$

USE $\boxed{W\ 14 \times 22}$ $(29.0 \ in^3)$

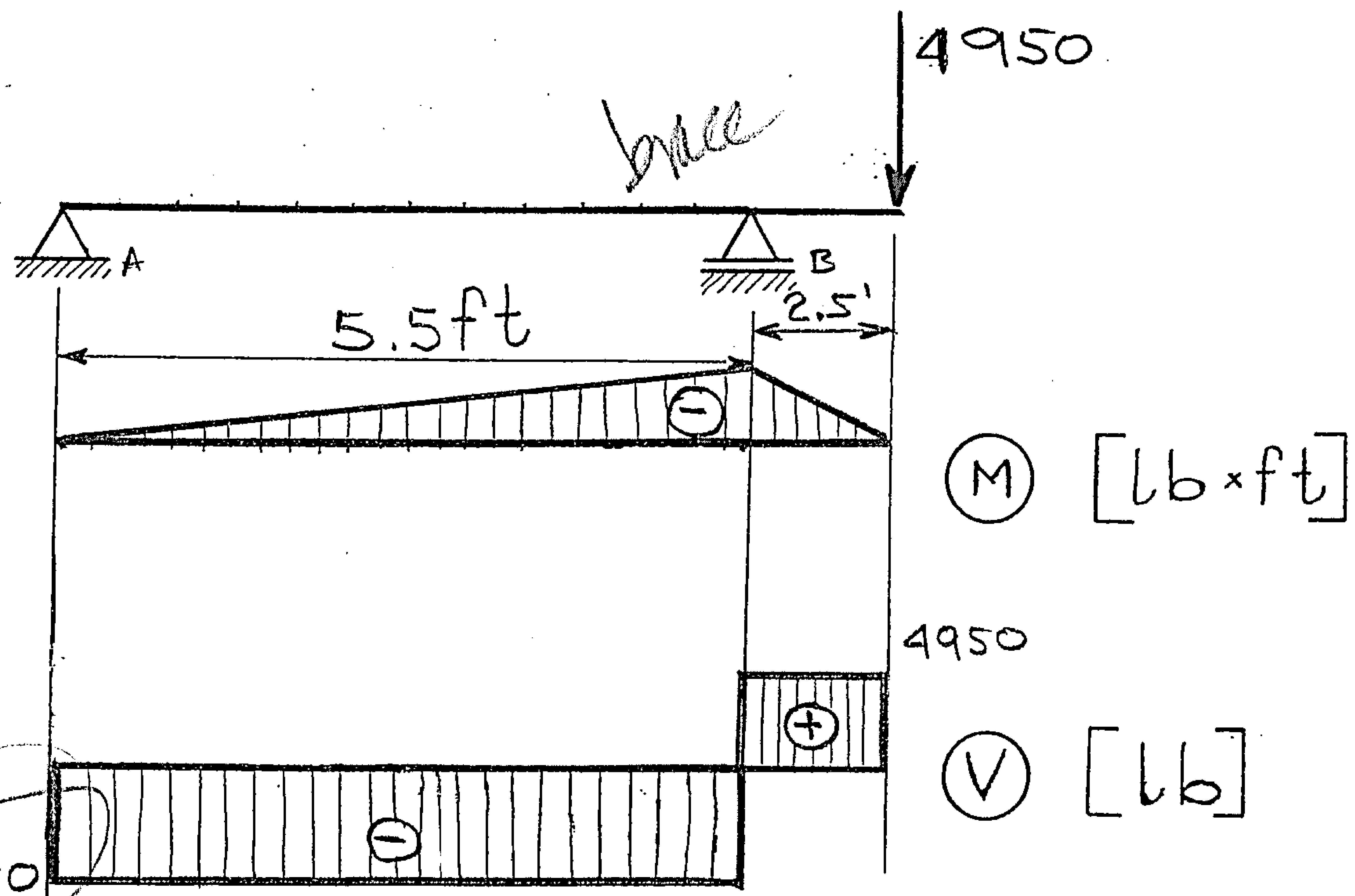
WESTGATE VILLAGE — Date MAR 12, 02 — Designed DT

Date ________ Checked ________

B8

$P = 4950\,lbs$

?, $4 \times 1100 = 4.4^k$



4 - 3/4" φ bolts in masonry

$M = 4950 \times 2.5 = 12375\ lb \times ft$

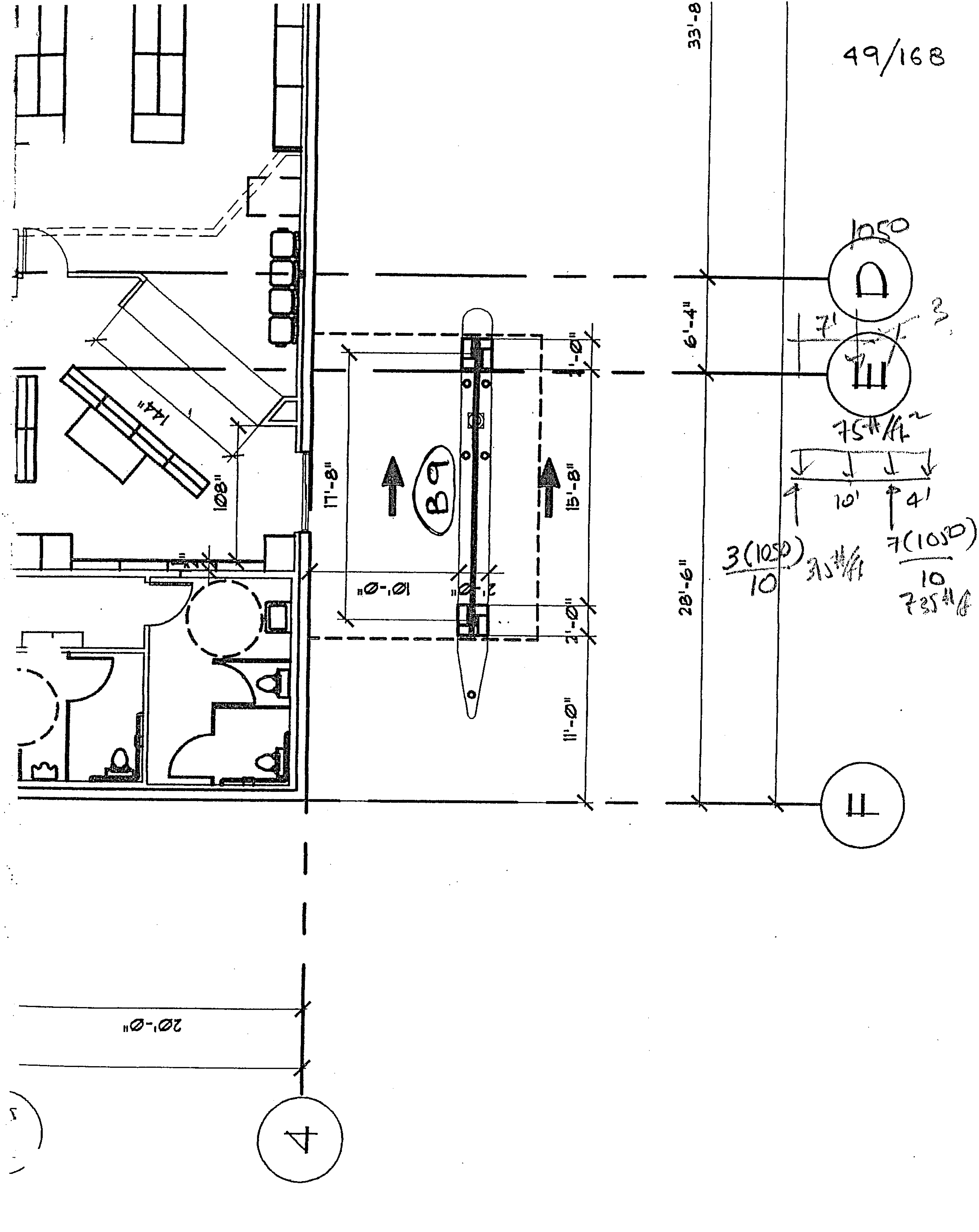
$R_B = \dfrac{4950 \times 8}{5.5} = 7200\ lb$

$S_{reqd} = \dfrac{12.5 \times 12}{30} = 5.0\ in^3$

unbraced length?

USE $\boxed{W\ 12 \times 14}$ ($14.9\,in^3$)

$39^{k'}$ ok



33'-8
1050
D
E
F
4
6'-4"
28'-6"
17'-8"
15'-8"
2'-0"
2'-0"
2'-0"
10'-0"
2'-0"
11'-0"
144"
108"
20'-0"
B9
7'
75#/ft
10'
4'
3(1050)/10
315#/ft
7(1050)/10
735#/ft

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WESTGATE VILLAGE Date MAR 12, 02 Designed DT

Date Checked

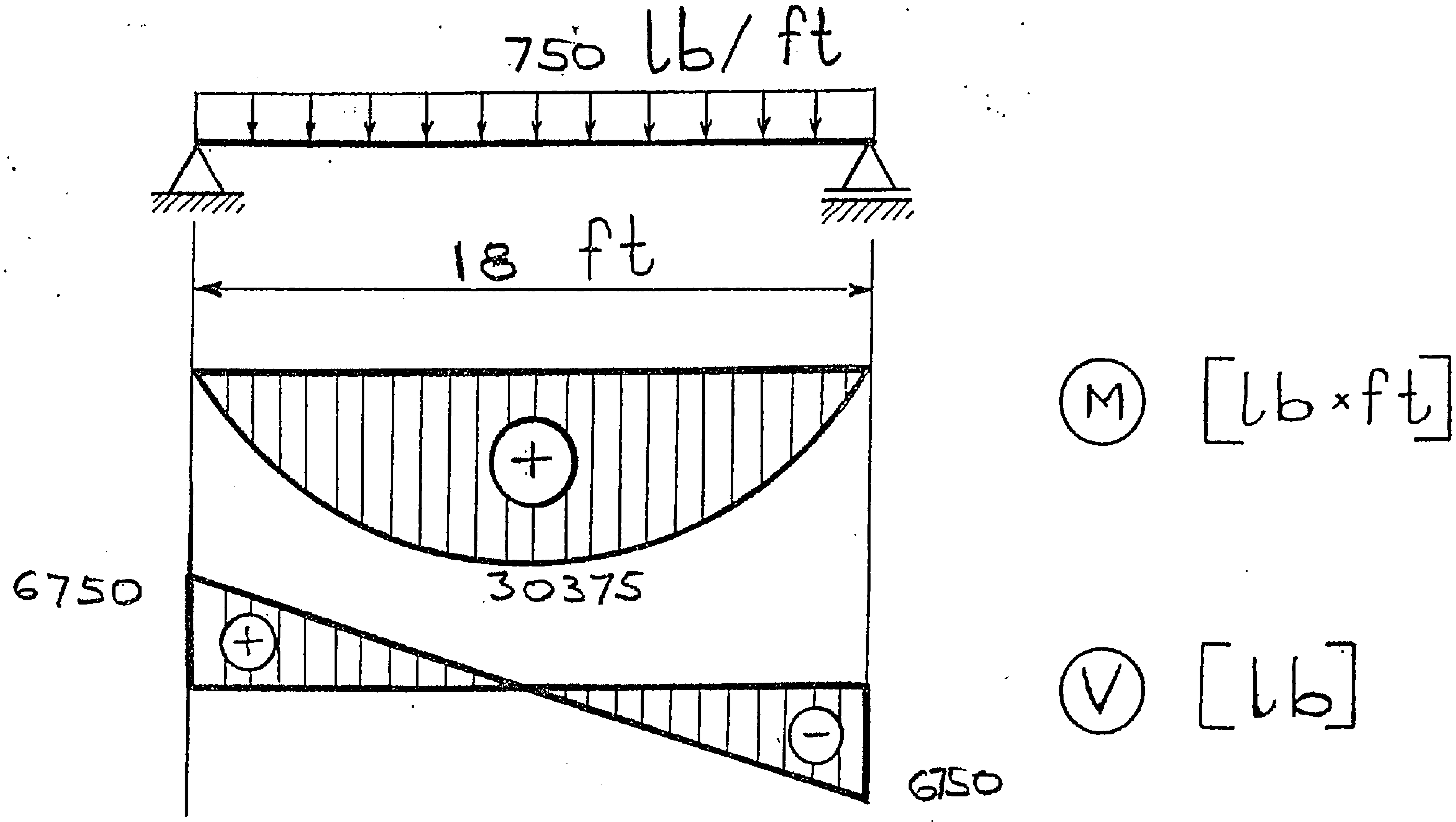
$\boxed{B9}$

D.L.= 25 ×10 = 250plf; LL=50×10=500plf

735 #/ft

ω = 250 + 500 = 750 lb/ft

ok



$$M = \frac{750 \times 18^2}{8} = 30375 \quad lb \times ft$$

$$V = \frac{750 \times 18}{2} = 6750 \quad lb$$

$$S_{reqd} = \frac{30.4 \times 12}{30} = 12.6 \ in^3$$

USE $\boxed{W\,12 \times 16}$ (17.1 in³)

$$\Delta = \frac{5}{384} \frac{0.75 \times (18 \times 12)^4}{29000 \times 103} = 0.59 \ in$$

Δ = ℓ/362 (O.K.)

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WESTGATE VILLAGE Date MAR 12, 02 Designed DT

Date Checked

TS HANGER

$$T = 5 \times 32 \times (\underset{\text{D.L.}}{20} + \underset{\text{SNOW}}{50}) \times 0.5 = 5600 \text{ lbs}$$

USE TS 4x4x¼ (A = 3.59 in²)

USE STEEL A500 GRADE B

$$f_{allow.} = 46/1.5 = 30.66 \text{ Ksi} \quad (30 \text{ ksi})$$

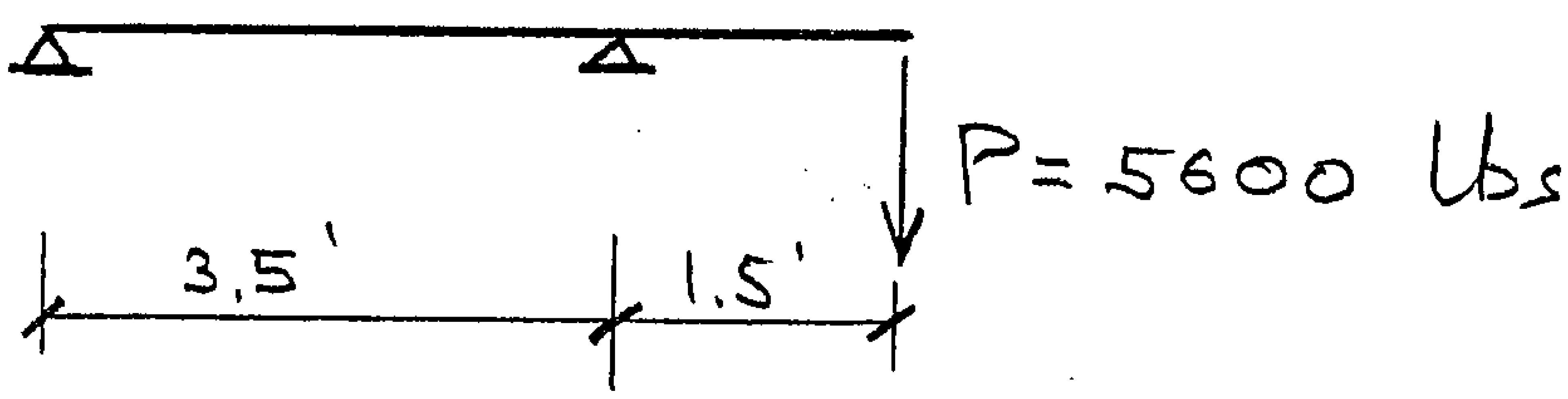
$$f = \frac{5.6}{3.6} = 1.55 \text{ ksi} \quad (\text{o.k.})$$

TS BRACE

$$T = 5492 \text{ lbs}$$

USE TS 4x4x¼

TS OUTRIGER



$$M = 1.5 \times 5600 = 8400 \text{ Lb} \times \text{ft}$$

$$S_{REQ^D} = \frac{8.4 \times 12}{30} = 3.36 \text{ in}^3$$

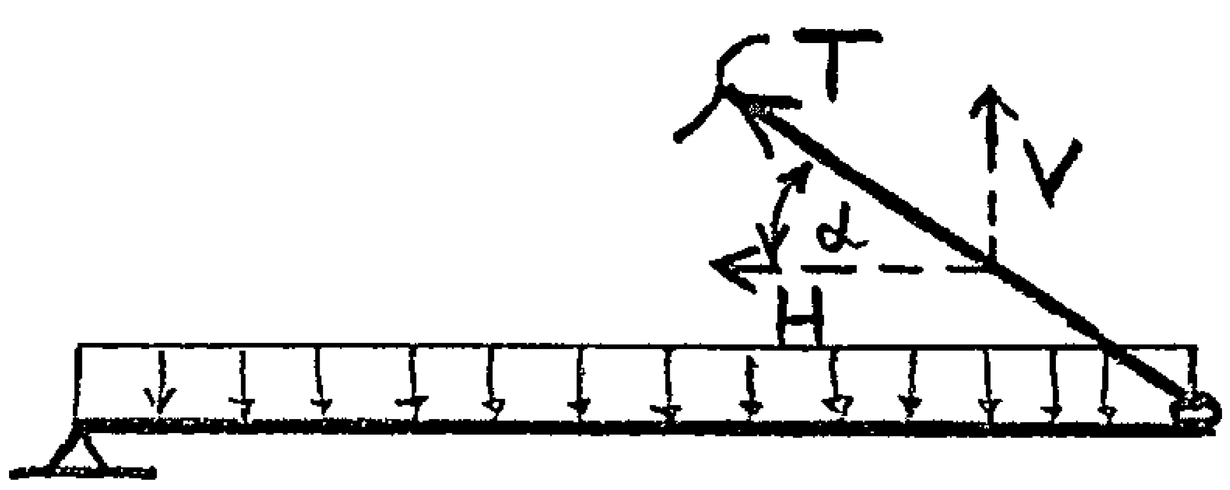
USE <u>TS 6x4x¼</u> (7.36 in³)

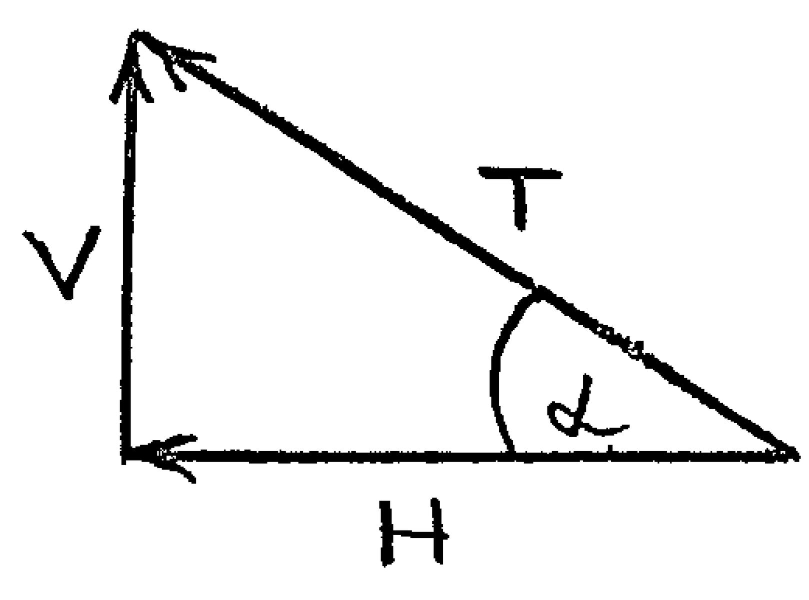
WESTGATE VILLAGE Date MAR 12, 08 Designed DT

Date Checked

TIE ROD

$\alpha = 35°$





$V = 18 \times (20+50) \times 5.0/2 = 3150$ Lbs

$H = V/tg\,\alpha = 4498$ Lbs

$T = V/\sin\alpha = 5492$ Lbs

$A\,reqd = \frac{5.5}{30} = 0.183\ in^2$

USE 1"ø ROD ($0.7854\ in^2$)

```
=================================================================
     M A S O N R Y   D E S I G N   S O F T W A R E  2 . 0 (C)
                      Fero Corporation
-----------------------------------------------------------------
 Masonry Beam and Lintel
=================================================================


Project     : Westgate Village
Job No.     : 9919
Description: Masory Beam
Designer    : Dragan Todorovic
Date        : Mar 12, 02
-----------------------------------------------------------------

Design Code : ACI 530-95/ASCE 5-95/TMS 402-95
Member Mark : Long Beam in East Wall
Run No.     : 1
Remarks     : 1.2D + 1.6S
=================================================================

Loading
=======

    Span                     = 300 (in)
    Boundary Conditions      = left pinned, right pinned
    Uniform load (Live)      = 0.634 (kip/ft)        Factor = 1.6
    Uniform load (Dead)      = 0.435 (kip/ft)        Factor = 1.2
    Triangular load (Dead)   = 0 (kip/ft)        Factor = 1
    Beam dead weight         = 1.2 (kip/ft)        Factor = 1.2

    Point loads:

            No:             x (in)     Dead (kip)     Live (kip)

    Point load factor (Dead) = 1 for all
    Point load factor (Live) = 1 for all

    Factored Moment:
    @ x =          150 (in)   Mf =       232.531 (kip-ft)
    @ x =          300 (in)   Mf = -1.11016e-13 (kip-ft)
```

55/168

Factored Shear:

@ x =	0	(in)	Vf =	37.205	(kip)
@ x =	300	(in)	Vf =	-37.205	(kip)

Design Shear = 29.2679 (kip)

Note: x = Distance from left support

Beam Properties:
================

Width = 7.625 (in)
Depth = 72 (in)

Beam classification = Concrete block masonry
Masonry strength f'm = 2000 (psi)
Steel yield strength fy = 60000 (psi)

Flexural Design:
================

Flexural Design - Positive Moment:
==================================

Factored moment, Mf(+ve) = 232.531 (kip-ft)
Factored resistance, Mr = 252.535 (kip-ft)
Tension steel:
4#7
Location of steel from tension face, dc (tension) = 8 (in)

?

Stresses - Actual

Masonry, fb = 666.667 (psi)
Tension steel, fs = 22140.1 (psi)

Stresses - Allowable

Masonry, Fb = 666.667 (psi)
Tension steel, Fs = 24000 (psi)

Flexural Design - Negative Moment:

Factored moment, Mf(-ve) = 1.11016e-13 (kip-ft)
Factored resistance, Mr = 252.535 (kip-ft)
Tension steel:
4#7
Location of steel from tension face, dc (tension) = 8 (in)

Stresses - Actual

Masonry, fb = 666.667 (psi)
Tension steel, fs = 22140.1 (psi)

Stresses - Allowable

Masonry, Fb = 666.667 (psi)
Tension steel, Fs = 24000 (psi)

Shear Design:

Shear reinforcement schedule:

From x (in)	To x (in)	Spacing, s (in)
0.0	64.0	8
60.0	240.0	Not req'd
237.0	301.0	8

s = shear reinforcement (stirrup) spacing

Stirrup type = Single Leg
Bar area(total) = 0.2 (in^2)
Yield strength = 60000 (psi)
Bar size = #4

```
Development Length:
===================

Development Length (Positive Moment):
=====================================

     Bar type                     = Straight bar
     Critical section @ x         = 150 (in)
     Min. bar cover (side)        = 1.5748 (in)

     Critical bar size (tension bar) = #7
     Required development length     = 31.5 (in)
     Tension bar spacing             = O.K.


Development Length (Negative Moment):
=====================================

     Bar type                 = Straight bar
     Critical section @ x     = 300 (in)
     Min. bar cover (side)    = 1.5748 (in)

     Critical bar size (tension bar) = #7
     Required development length     = 31.5 (in)
     Tension bar spacing             = O.K.



Bearing:
========

  Bearing area required:
@ x (in) =           0, Bf (kip)  =     37.205, A1 (in^2) =       74.41 (End Re
@ x (in) =         300, Bf (kip)  =     37.205, A1 (in^2) =       74.41 (End Re


Deflection:
===========
```

Maximum deflection	=	0.341945 (in)
Allowable deflection	=	0.5 (in)
Span/Deflection	=	877.335
Span/Deflection(Allowable)	=	600
Duration of sustained load	=	5 years or more
Percent sustained load	=	100%

COLUMNS

(h = 19'-0)

3/B

P = [(36'-6)/2 + (37'-4)/2] × [(40'-0)/2 + (39'-6)/2] × (25 + 25) = 73372 lbs

USE TS 6×6×1/4 (83 kips)

2/B

P = [(37'-4)/2 + (28'-2)/2] × [(40'-0)/2 + (39'-6)/2] × (25 + 25) + (24'-0) × (30'-0) × 30 = 86690 lbs

USE TS 6×6×5/16 (100 kips)

2/C

P = (40'-0) × [(28'-2)/2 + (37'-4)/2] × (25 + 25)
= 65500

USE TS 6×6×1/4 (83 kips)

CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE   Date MAR 7, 02   Designed DT

Date   Checked

$\boxed{3/C}$ $(h=19'-0)$

$$P = [(35'-0)/2 + (40'-0)/2] \times [(36'-6)/2 + (37'-4)/2] \times (25+25) + (20'-0) \times (30'-0) \times 30 = 87218 \text{ lbs}$$

USE TS $6 \times 6 \times \frac{5}{16}$ (100 kips)

$\boxed{3a/D}$

$$P = [(34'-10)/2 + (33'-8)/2] \times [(20'-0)/2 + (21'-6)/2] \times (25+25) = 35534 \text{ lbs}$$

USE TS $5 \times 5 \times \frac{1}{4}$ (49 kips)

$\boxed{2a/D}$

$$P = [(21'-6)/2 + (35'-1)/2] \times [(34'-10)/2 + (33'-8)/2] \times (25+25) = 48450 \text{ lbs}$$

USE TS $6 \times 6 \times \frac{1}{4}$ (83 kips)

$\boxed{1a/E}$

$$P = [(28'-6)/2 + (40'-0)/2] \times [(25'-5)/2 + (35'-1)/2] \times (25+25) + (26'-0) \times (28'-0) \times 35 = 77284 \text{ lbs}$$

USE TS $6 \times 6 \times \frac{5}{16}$ (100 kips)

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WESTGATE VILLAGE Date MAR 7, 02 Designed DT

Date Checked

4/B $(h = 10'-0)$

$P = (36'-6)/2 \times [(40'-0)/2 + (39'-6)/2] \times (25+25) + [(40'-0)/2 + (39'-6)/2] \times (7'-0) \times 15$

$= 40446$ lbs

USE TS 5×3×1/2 (76 kips)

4/C

$P = (36'-6)/2 \times [(40'-0)/2 + (33'-8)/2] \times (25+25) + [(40'-0)/2 + (33'-8)/2] \times (7'-0) \times 15$

$= 37478$ lbs

USE TS 5×3×1/2 (76 kips)

4/D

$P = (20'-0)/2 \times [(34'-10)/2 + (33'-8)/2] \times (25+25) + [(34'-10)/2 + (33'-8)/2] \times (7'-0) \times 15$

$= 20722$ lbs

USE TS 5×3×1/2 (76 kips)

WESTGATE VILLAGE Date MAR 7, 02 Designed DT

Date Checked

$(h = 10'-0)$

1/B

$$P = (28'-2)/2 \times [(40'-0)/2 + (39'-6)/2] \times (25+25) + [(40'-0)/2 + (39'-6)/2] \times (7'-0) \times 15$$

$= 32\,164$ lbs

USE TS 5×3×½ (76 kips)

1/C

$$P = (28'-2)/2 \times [(40'-0)/2 + (40'-0)/2] \times (25+25) + [(40'-0)/2 + (40'-0)/2] \times (7'-0) \times 15$$

$= 32\,366$ lbs

USE TS 5×3×½ (76 kips)

1/E

$$P = (25'-5)/2 \times [(28'-6)/2 + (40'-0)/2] \times (25+60)$$

$= 36998$ lbs

USE TS 5×3×½ (76 kips)

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WESTGATE VILLAGE Date MAR 7, 02 Designed DT

Date Checked

FOOTINGS

PAD FOOTINGS

[illegible] 5.7 K [illegible]

3/B

ASSUMED WEIGHT OF FOOT.

$P = 73500 + 10000 = 83500$ lbs

$a = \sqrt{83500/3000} = 5.27$ ft

USE FOOTING 5'-6 × 5'-6 × 1'-3

2/B

$P = 87000 + 10000 = 97000$ lbs

$a = \sqrt{97000/3000} = 5.69$ ft

USE FOOTING 6'-0 × 6'-0 × 1'-4

2/C

$P = 65500 + 8000 = 73500$ lbs

$a = \sqrt{73500/3000} = 4.94$ ft

USE FOOTING 5'-0 × 5'-0 × 1'-2

CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE — Date MAR 7, 02 — Designed DT
Date — Checked

$\boxed{3/C}$

$P = 87500 + 10000 = 97500 \text{ lbs}$

$a = \sqrt{97500/3000} = 5.70 \text{ ft}$

USE FOOTING 6'-0 × 6'-0 × 1'-4

$\boxed{3a/D}$

$P = 36000 + 5000 = 41000 \text{ Lbs}$

$a = \sqrt{41000/3000} = 3.69 \text{ Lbs}$

USE FOOTING 4'-0 × 4'-0 × 1'-0

$\boxed{2a/D}$

$P = 48500 + 6000 = 54500 \text{ Lbs}$

$a = \sqrt{54500/3000} = 4.26 \text{ ft}$

USE FOOTING 4'-6 × 4'-6 × 1'-1

$\boxed{1a/E}$

$P = 77500 + 10000 = 87500 \text{ Lbs}$

$a = \sqrt{87500/3000} = 5.40 \text{ ft}$

USE FOOTING 5'-6 × 5'-6 × 1'-3

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WESTGATE VILLAGE Date MAR 7, 02 Designed DT

Date Checked

LINES ① & ④

$$\max P = 40000 + 4000 = 44000 \text{ lbs}$$

$$a = \sqrt{44000/3000} = 3.82 \text{ ft}$$

USE FOOTING 4'-0 × 4'-0 × 1'-0

STRIP FOOTINGS

SW1

$$P = 148 \times (54 \times \overset{\text{WALL}}{24} + \overset{\text{WEIGHT OF FOOTING}}{250}) = 228808 \text{ lbs}$$

$$M = 702\,828 \text{ lb} \times \text{ft}$$

$$e = 702828/228808 = 3.07 \text{ ft}$$

$$p_1 = \frac{228808}{1.6\dot{6} \times 148}\left(1 + \frac{6 \times 3.07}{148}\right) = 1043 \text{ psf}$$

O.K.

$$p_2 = \frac{228808}{1.6\dot{6} \times 148}\left(1 - \frac{6 \times 3.07}{148}\right) = 812 \text{ psf}$$

O.K.

USE STRIP FOOTING 1'-8 WIDE & 1'-0 DEEP

CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date MAR 7, 02 Designed DT
Date Checked

SW2

DEAD + LIVE

$$P_D = (54 * 24) + 250 + 435 = 1981 \text{ lbs/ft}$$

$$P_L = 1045 \text{ Lbs/ft}$$

$$p = \frac{1981 + 1045}{1.8} = 1681 \text{ psf}$$

O.K.

SEISMIC

$$P = 1981 \times 102 = 202062 \text{ Lbs}$$

$$M = 702208 \text{ lb} \times \text{ft}$$

$$e = 702208 / 202062 = 3.47 \text{ ft}$$

$$p_1 = \frac{202062}{1.6\dot{6} \times 102}\left(1 + \frac{6 \times 3.47}{102}\right) = 1431 \text{ psf}$$

O.K.

$$p_2 = \frac{202062}{1.6\dot{6} \times 102}\left(1 - \frac{6 \times 3.47}{102}\right) = 945 \text{ psf}$$

O.K.

USE STRIP FOOTING 1'-8 WIDE & 1'-0 DEEP

WESTGATE VILLAGE Date MAR 7, 02 Designed DT

Date Checked

SW3

DEAD + LIVE

$$P_D = (54 * 24) + 250 + 435 = 1981 \text{ lbs/ft}$$

$$P_L = 634 \text{ Lbs/ft}$$

$$p = \frac{1981 + 634}{1.8} = 1452 \text{ psf}$$

O.K.

SEISMIC

$$P = 1981 \times 53 = 104993 \text{ Lbs}$$

$$M = 902272 \text{ Lb} \times \text{ft}$$

$$e = 902272 / 104993 = 8.59$$

$$p_1 = \frac{104993}{1.66 \times 53} \left(1 + \frac{6 \times 8.59}{53}\right) = 2344 \text{ psf}$$

O.K.

$$p_2 = \frac{104993}{1.66 \times 53} \left(1 - \frac{6 \times 8.59}{53}\right) = 32 \text{ psf}$$

O.K.

USE STRIP FOOTING 1'-8 WIDE & 1'-0 DEEP

WESTGATE VILLAGE Date MAR 19, 02 Designed DT

Date Checked

DETENTION POND

SLAB

250/62.4 = 10' water deep only 7-7' used.

1) LIVE LOAD .....250 psf (TABLE 16-A LINE 17; UBC)

2) SUPERIMPOSED DEAD LOAD

$1.5 \times 140 = 210$ psf

* DECK WILL ALSO BE CHECKED FOR POINT LOAD OF 12,000 Lbs

F L O O R D E S v. 4.01

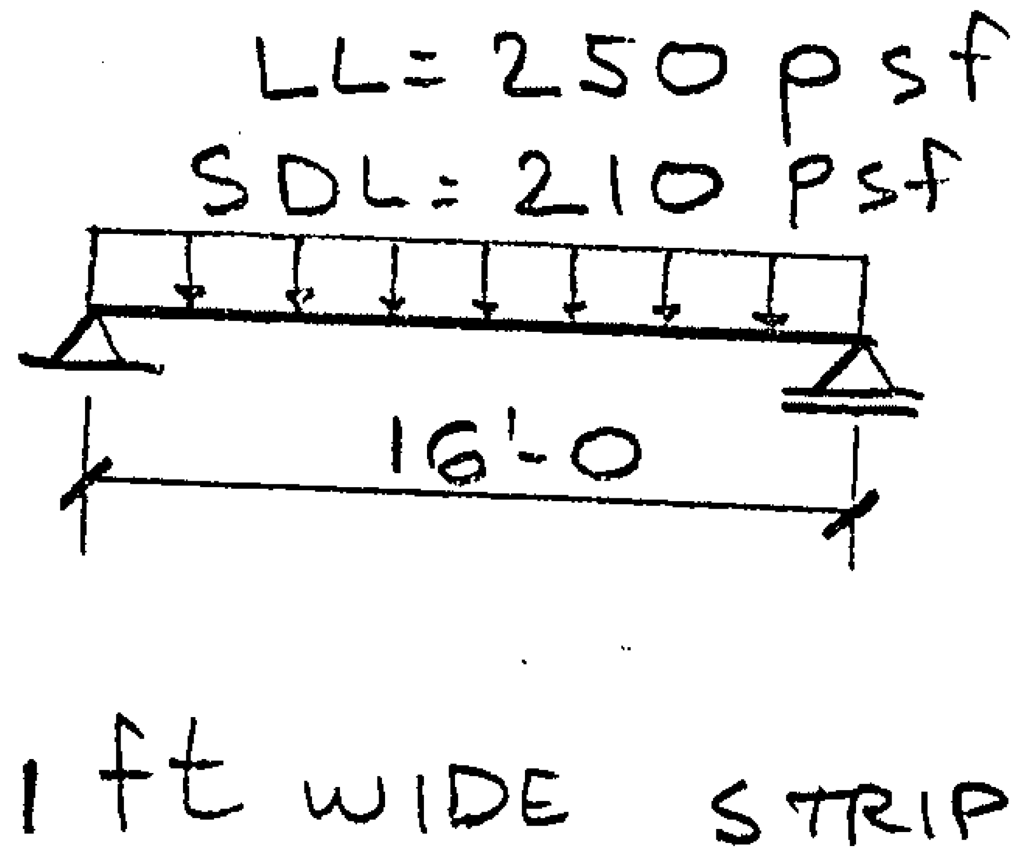
Date: 03-20-2002 Time: 10:48:15

Design Code: ACI 318-89 --- Data File: slab1

```
************************************************
***  Detention Pond Slab - Slab1          #9919
************************************************

**********************************
***  STRIP Slab1  - INPUT DATA  ***
**********************************

No. of Joints              =  2
Upper Column Height           No Column
Gen. Lower Column Height   = 999.00  ft
----------------------------------------
Concrete Strength          =   4.00  ksi
Concrete Weight            =    150  pcf
Flexural Steel Strength    =  60.00  ksi
Stirrup Steel  Strength    =  60.00  ksi
----------------------------------------
Top Cover to CL Reinf.     =   2.00  in
Bottom Cover to CL Reinf.  =   2.00  in
----------------------------------------
Modulus of Elasticity      =   3857  ksi
```



JOINT DATA

JT NO	Vert. Joint Freedom	Rota-tional Freedom	LOWER COLUMN Perp. (in)	LOWER COLUMN Paral. (in)	UPPER COLUMN Perp. (in)	UPPER COLUMN Paral. (in)	Upper Col-Ht. (ft)	Lower Col-Ht. (ft)
1	Supp	Free	0.00	0.00	-	-	-	999.00
2	Supp	Free	0.00	0.00	-	-	-	999.00

MEMBER TYPES

BM=Beam - SB=Slab Band - 1S=One-Way-Slab - 2SI/2SB=Two-Way-Slab (Int/Ext)

MB NO	MEMB TYPE	MB LGT (ft)	MEMB THICK-NESS (in)	WEB WIDTH (in)	FLG. THICK-NESS (in)	O.HANG FLANGE WIDTH (in)	SERV. AUTO. DL (K/ft)	SERV. SUPERIMP. DL (K/ft)	SERV. LL LC 1 (K/ft)
1	1S	16.00	12.00	12.00	0.00	0.00	0.15	0.21	0.25

```
*******************************************************
***  STRIP Slab1  - Factored END MOMENTS AND SHEARS  ***
*******************************************************
```

MB NO	L C	LEFT END MOM (Kft)	LEFT END SHEAR (K)	*	RIGHT END SHEAR (K)	RIGHT END MOM (Kft)
1	1	0.00	7.46	*	-7.46	0.00

```
*****************************************
***  STRIP Slab1  - FLEXURAL DESIGN  ***
*****************************************
```

Check ACI 318-89 Clause 12.10.5 before terminating any bar in a Tension Zone
>> or << - Indicates that Reinforcing Steel shown is Code Minimum
99999 - Indicates a Section that cannot be reinforced for the calculated moment

MB NO	L C	FACT. LEFT DES MOM (Kft)	DIST TO INFL (ft)	LEFT MOM REINF (in2)	*	DIST TO MMAX (ft)	FACT. INT. MAX. MOMENT (Kft)	MOM REINF (in2)	*	RIGHT MOM REINF (in2)	DIST TO INFL (ft)	FACT. RIGHT DES MOM (Kft)
1	1	0.00	-	0.00	*	8.01	29.85	0.70	*	0.00	-	0.00

```
**************************************
***  STRIP Slab1  - SHEAR DESIGN  ***
**************************************
```

- Shear Reinf. calcs. assume that no Flexural Reinf. is terminated in a Tension Zone otherwise increase the Shear Reinf. as per ACI 318-89 Clause 12.10.5
- 9999 - Indicates that the Concrete Shear exceeds maximum Code value

MB NO	L C	PHI*Vc (K)	*	DIST TO CRIT SECT (in)	ULT SHEAR CRIT SECT (K)	MAX SPAC (in)	Left Av/ft (in2/ft)	Right Av/ft (in2/ft)	MAX SPAC (in)	ULT SHEAR CRIT SECT (K)	DIST TO CRIT SECT (in)	*	PHI*Vc (K)
1	1	15.24	*	10.0	6.69	-	0.00	0.00	-	6.69	-10.0	*	15.24

****
*** STRIP Slab1 - SUPPORT LOADS ***
****

Loads positive downwards - Moments positive clockwise

JT NO	L C	SERVICE DEAD LOAD (K)	SERVICE LIVE LOAD (K)	FACTORED TOTAL LOAD (K)	FACT UPPER COLUMN MOM (KFT)	FACT LOWER COLUMN MOM (KFT)
1	1	2.90	2.00	7.46	0.00	0.00
2	1	2.90	2.00	7.46	0.00	0.00

****
*** STRIP Slab1 - DEFLECTIONS ***
****

- All Deflections are for service loads on the Cracked Section
- Refer to ACI 813-89 Table 9.5(b) for code limitations on Deflections
- Deflections positive downwards

SP NO	L C	MAX. AVER. Ig/Ie	DIST TO MAX DEFL (ft)	IMMEDIATE LL-DEFLECTION (in)	IMM DL-DEF (in)	ADD. DL-DEF (in)	IMM. LL-DEF + ADD. DL-DEF (in)
1	1	2.34	8.00	0.24 (L/810)	0.08	0.16	0.40 (L/483)

FLOORDES v. 4.01

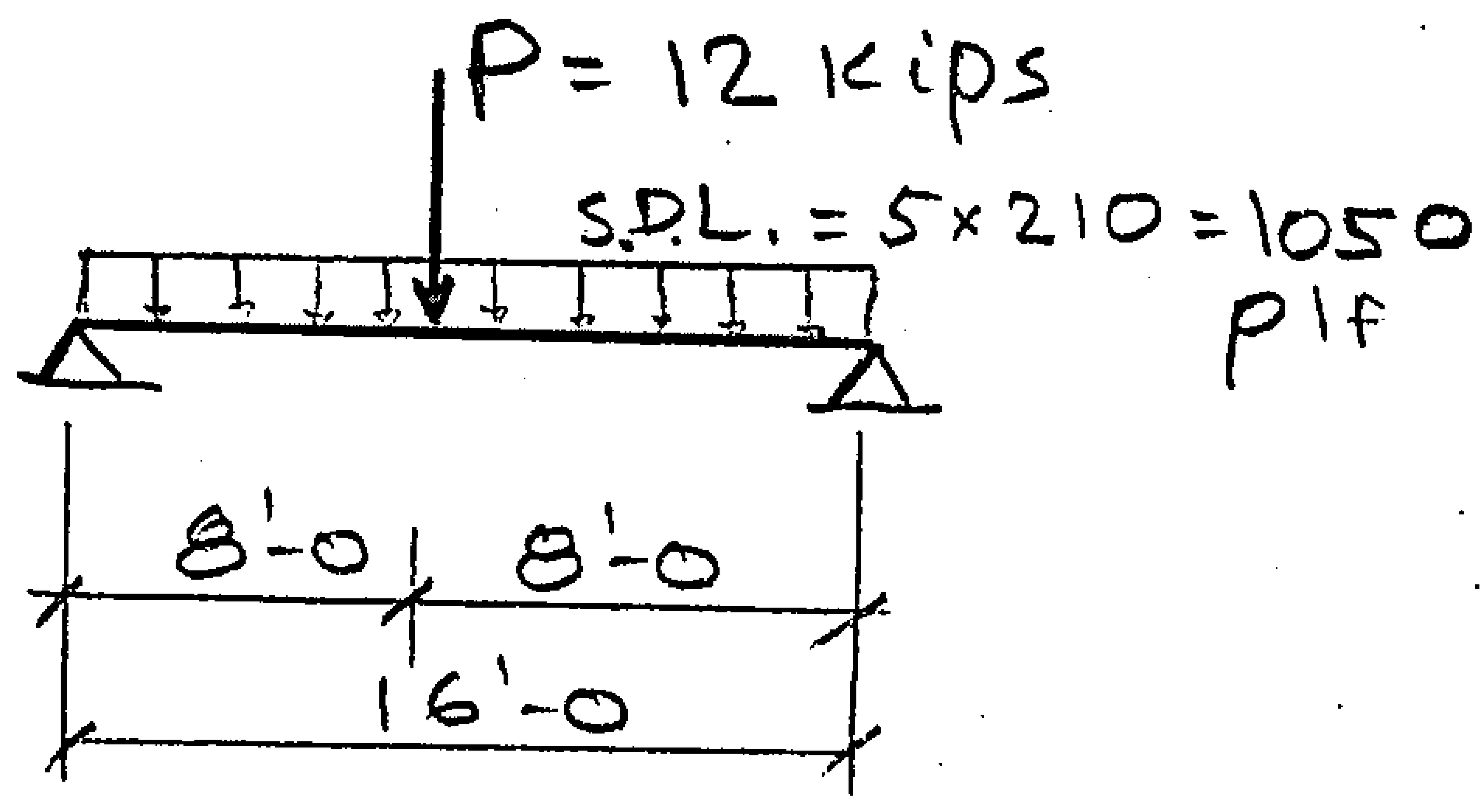
Date: 03-20-2002 Time: 10:42:09

Design Code: ACI 318-89 --- Data File: slab2

```
*************************************************
***  Slab at Detention Pond - Slab2          #9919
*************************************************

***********************************
***  STRIP Slab2  - INPUT DATA  ***
***********************************

No. of Joints                =  2
Upper Column Height             No Column
Gen. Lower Column Height     = 999.00  ft
-------------------------------------------
Concrete Strength            =   4.00  ksi
Concrete Weight              =    150  pcf
Flexural Steel Strength      =  60.00  ksi
Stirrup Steel  Strength      =  60.00  ksi
-------------------------------------------
Top Cover to CL Reinf.       =   2.00  in
Bottom Cover to CL Reinf.    =   2.00  in
-------------------------------------------
Modulus of Elasticity        =   3857  ksi
```



5'-0 WIDE STRIP

JOINT DATA

JT NO	Vert. Joint Freedom	Rota-tional Freedom	LOWER COLUMN Perp. (in)	LOWER COLUMN Paral. (in)	UPPER COLUMN Perp. (in)	UPPER COLUMN Paral. (in)	Upper Col-Ht. (ft)	Lower Col-Ht. (ft)
1	Supp	Free	0.00	0.00	-	-	-	999.00
2	Supp	Free	0.00	0.00	-	-	-	999.00

MEMBER TYPES

BM=Beam - SB=Slab Band - 1S=One-Way-Slab - 2SI/2SE=Two-Way-Slab (Int/Ext)

MB NO	MEMB TYPE	MB LGT (ft)	MEMB THICK-NESS (in)	WEB WIDTH (in)	FLG. THICK-NESS (in)	O.HANG FLANGE WIDTH (in)	SERV. AUTO. DL (K/ft)	SERV. SUPERIMP. DL (K/ft)	SERV. LL LC 1 (K/ft)
1	1S	16.00	12.00	60.00	0.00	0.00	0.76	1.05	0.00

MEMBER POINT LOADS (Serv.)

MB NO	DIST TO LOAD (ft)	DL (K)	LL LC 1 (K)
1	8.00	0.00	12.00

****
*** STRIP Slab2 - Factored END MOMENTS AND SHEARS ***
****

MB NO	L C	LEFT END MOM (Kft)	LEFT END SHEAR (K)	*	RIGHT END SHEAR (K)	RIGHT END MOM (Kft)
1	1	0.00	30.52	*	-30.52	0.00

****
*** STRIP Slab2 - FLEXURAL DESIGN ***
****

Check ACI 318-89 Clause 12.10.5 before terminating any bar in a Tension Zone
>> or << - Indicates that Reinforcing Steel shown is Code Minimum
99999 - Indicates a Section that cannot be reinforced for the calculated moment

MB NO	L C	FACT. LEFT DES MOM (Kft)	DIST TO INFL (ft)	LEFT MOM REINF (in2)	*	DIST TO MMAX (ft)	FACT. INT. MAX. MOMENT (Kft)	MOM REINF (in2)	*	RIGHT MOM REINF (in2)	DIST TO INFL (ft)	FACT. RIGHT DES MOM (Kft)
1	1	0.00	-	0.00	*	8.01	162.88	3.83	*	0.00	-	0.00

$\rightarrow$ 3.83/5 = 0.766 in² per ft

****
*** STRIP Slab2 - SHEAR DESIGN ***
****

- Shear Reinf. calcs. assume that no Flexural Reinf. is terminated in a Tension Zone otherwise increase the Shear Reinf. as per ACI 318-89 Clause 12.10.5
- 9999 - Indicates that the Concrete Shear exceeds maximum Code value

MB NO	L C	PHI*Vc (K)	*	DIST TO CRIT SECT (in)	ULT SHEAR CRIT SECT (K)	MAX SPAC (in)	Left Av/ft (in2/ft)	Right Av/ft (in2/ft)	MAX SPAC (in)	ULT SHEAR CRIT SECT (K)	DIST TO CRIT SECT (in)	*	PHI*Vc (K)
1	1	76.19	*	10.0	28.41	-	0.00	0.00	-	28.40	-10.0	*	76.19

```
************************************
***  STRIP Slab2  - SUPPORT LOADS  ***
************************************
```

Loads positive downwards - Moments positive clockwise

JT NO	L C	SERVICE DEAD LOAD (K)	SERVICE LIVE LOAD (K)	FACTORED TOTAL LOAD (K)	FACT UPPER COLUMN MOM (KFT)	FACT LOWER COLUMN MOM (KFT)
1	1	14.52	6.00	30.52	0.00	0.00
2	1	14.52	6.00	30.52	0.00	0.00

```
************************************
***  STRIP Slab2  - DEFLECTIONS  ***
************************************
```

- All Deflections are for service loads on the Cracked Section
- Refer to ACI 813-89 Table 9.5(b) for code limitations on Deflections
- Deflections positive downwards

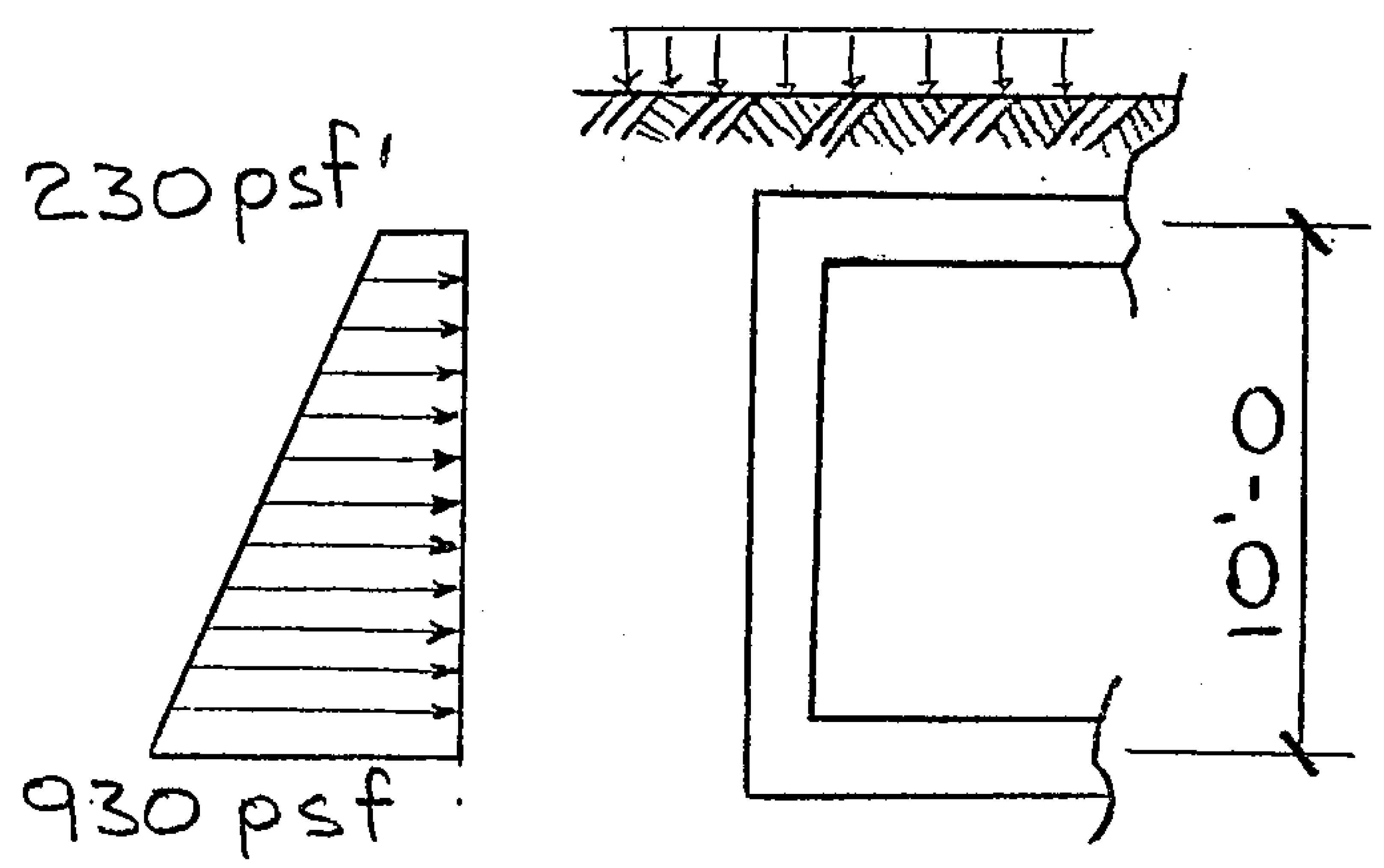
SP NO	L C	MAX. AVER. Ig/Ie	DIST TO MAX DEFL (ft)	IMMEDIATE LL-DEFLECTION (in)	IMM DL-DEF (in)	ADD. DL-DEF (in)	IMM. LL-DEF + ADD. DL-DEF (in)
1	1	2.54	8.00	0.25 (L/753)	0.08	0.16	0.42 (L/462)

WESTGATE VILLAGE Date MAR 19, 02 Designed DT

Date Checked

WALL

A) EMPTY TANK



LOAD @ TOP

$p = 250 + 210 = 460 \text{ psf}$

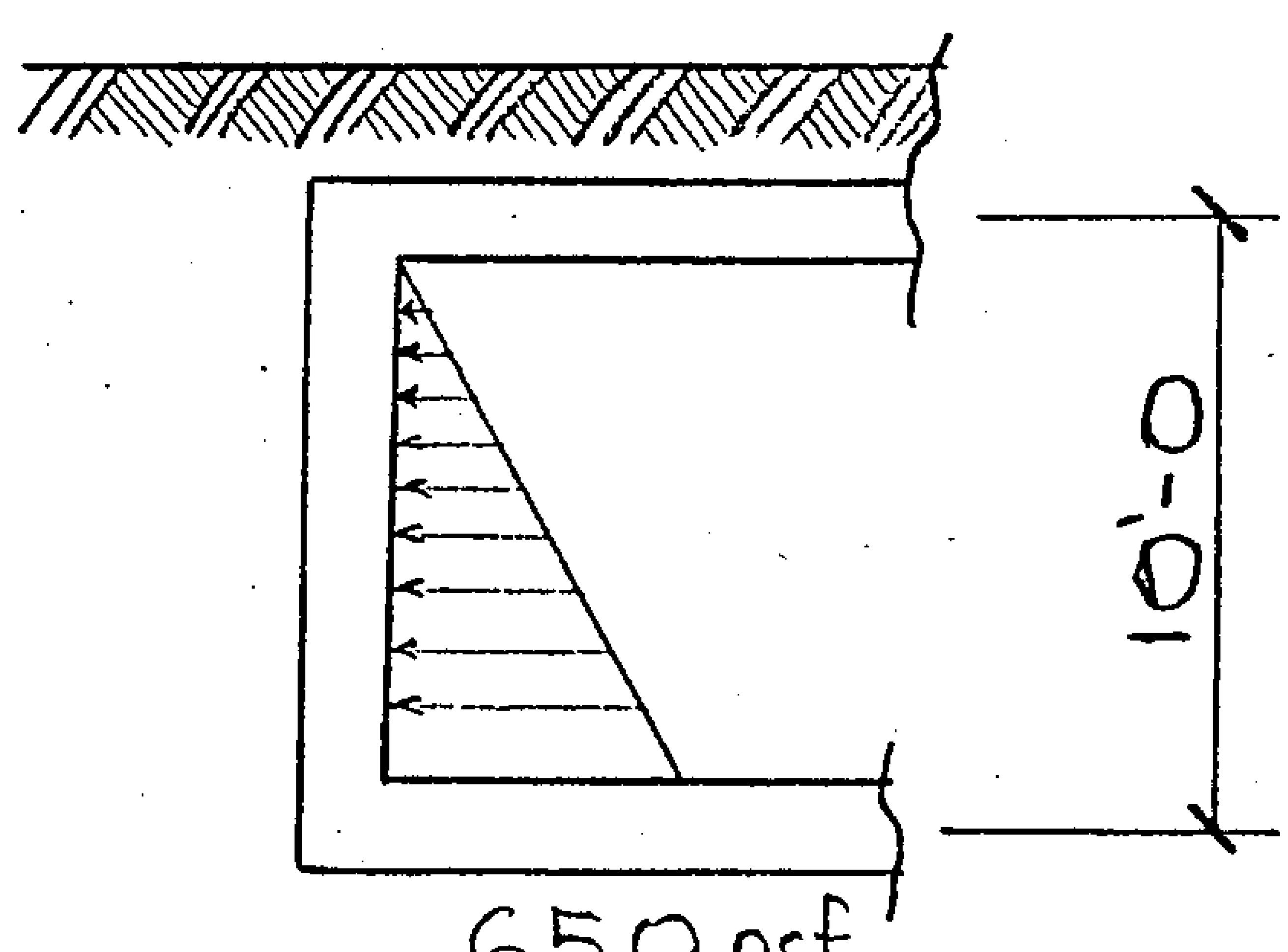
PER GEOTECH. REP.

$q = 70/140 = 0.5$

$p_1 = 460 \times 0.5 = 230 \text{ psf}$

$p_2 = 230 + 70 \times 10 = 930 \text{ psf}$

B) FULL TANK



$P_1 = 0$

$P_2 = 62.5 \times 10 = 650 \text{ psf}$

* PASIVE EARTH PESSURE IGNORED

F L O O R D E S v. 4.01

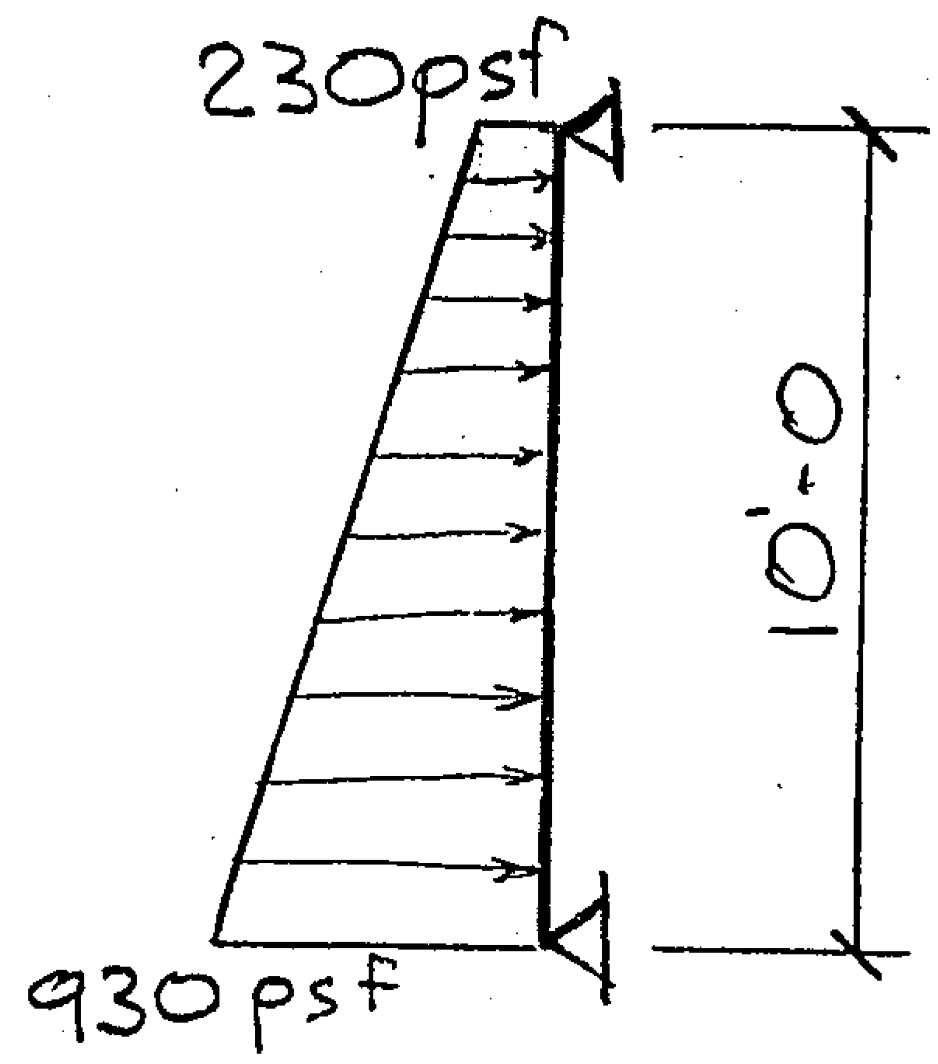
Date: 03-20-2002 Time: 10:52:36

Design Code: ACI 318-89 --- Data File: wall1

```
************************************************
***  Wall - Detention Pond                 # 9919
************************************************

**********************************
***  STRIP wall1  - INPUT DATA ***
**********************************

No. of Joints              =  2
Upper Column Height           No Column
Gen. Lower Column Height   = 999.00  ft
---------------------------------------
Concrete Strength          =   4.00  ksi
Concrete Weight            =    150  pcf
Flexural Steel Strength    =  60.00  ksi
Stirrup Steel  Strength    =  60.00  ksi
---------------------------------------
Top Cover to CL Reinf.     =   2.00  in
Bottom Cover to CL Reinf.  =   2.00  in
---------------------------------------
Modulus of Elasticity      =   3857  ksi
```



1 ft WIDE STRIP

JOINT DATA

JT NO	Vert. Joint Freedom	Rota-tional Freedom	LOWER COLUMN Perp. (in)	LOWER COLUMN Paral. (in)	UPPER COLUMN Perp. (in)	UPPER COLUMN Paral. (in)	Upper Col-Ht. (ft)	Lower Col-Ht. (ft)
1	Supp	Free	0.00	0.00	-	-	-	999.00
2	Supp	Free	0.00	0.00	-	-	-	999.00

MEMBER TYPES

BM=Beam - SB=Slab Band - 1S=One-Way-Slab - 2SI/2SE=Two-Way-Slab (Int/Ext)

MB NO	MEMB TYPE	MB LGT (ft)	MEMB THICK-NESS (in)	WEB WIDTH (in)	FLG. THICK-NESS (in)	O.HANG FLANGE WIDTH (in)	SERV. DL (K/ft)	SERV. LL LC 1 (K/ft)
1	1S	10.00	10.00	12.00	0.00	0.00	0.00	0.00

UNIFORMLY VARYING LOADS (Serv.)

MB NO	DL W-LEFT (K/ft)	DL W-RIGHT (K/ft)	LL-LC 1 W-LEFT (K/ft)	LL-LC 1 W-RIGHT (K/ft)
1	0.00	0.00	0.23	0.23

****

*** STRIP wall1 - Factored END MOMENTS AND SHEARS ***

****

MB NO	L C	LEFT END MOM (Kft)	LEFT END SHEAR (K)	*	RIGHT END SHEAR (K)	RIGHT END MOM (Kft)
1	1	0.00	5.92	*	-3.94	0.00

80/168

```
*****************************************
***  STRIP wall1  - FLEXURAL DESIGN  ***
*****************************************
```

Check ACI 318-89 Clause 12.10.5 before terminating any bar in a Tension Zone
>> or << - Indicates that Reinforcing Steel shown is Code Minimum
99999 - Indicates a Section that cannot be reinforced for the calculated moment

MB NO	L C	FACT. LEFT DES MOM (Kft)	DIST TO INFL (ft)	LEFT MOM REINF (in2)	*	DIST TO MMAX (ft)	FACT. INT. MAX. MOMENT (Kft)	MOM REINF (in2)	*	RIGHT MOM REINF (in2)	DIST TO INFL (ft)	FACT. RIGHT DES MOM (Kft)
1	1	0.00	-	0.00	*	4.53	12.44	0.36	*	0.00	-	0.00

```
*************************************
***  STRIP wall1  - SHEAR DESIGN  ***
*************************************
```

- Shear Reinf. calcs. assume that no Flexural Reinf. is terminated in a Tension Zone otherwise increase the Shear Reinf. as per ACI 318-89 Clause 12.10.5
- 9999 - Indicates that the Concrete Shear exceeds maximum Code value

MB NO	L C	PHI*Vc (K)	*	DIST TO CRIT SECT (in)	ULT SHEAR CRIT SECT (K)	MAX SPAC (in)	Left Av/ft (in2/ft)	Right Av/ft (in2/ft)	MAX SPAC (in)	ULT SHEAR CRIT SECT (K)	DIST TO CRIT SECT (in)	*	PHI*Vc (K)
1	1	12.19	*	8.0	4.89	-	0.00	0.00	-	3.65	-8.0	*	12.19

```
***********************************
***  STRIP wall1  - SUPPORT LOADS  ***
***********************************
```

Loads positive downwards - Moments positive clockwise

JT NO	L C	SERVICE DEAD LOAD (K)	SERVICE LIVE LOAD (K)	FACTORED TOTAL LOAD (K)	FACT UPPER COLUMN MOM (KFT)	FACT LOWER COLUMN MOM (KFT)
1	1	0.00	3.48	5.92	0.00	0.00
2	1	0.00	2.32	3.94	0.00	0.00

```
***********************************
***  STRIP wall1  - DEFLECTIONS  ***
***********************************
```

- All Deflections are for service loads on the Cracked Section
- Refer to ACI 813-89 Table 9.5(b) for code limitations on Deflections
- Deflections positive downwards

SP NO	L C	MAX. AVER. Ig/Ie	DIST TO MAX DEFL (ft)	IMMEDIATE LL-DEFLECTION (in)	IMM DL-DEF (in)	ADD. DL-DEF (in)	IMM. LL-DEF + ADD. DL-DEF (in)
1	1	1.00	5.00	0.03 (L/999)	0.00	0.00	0.03 (L/999)

FLOORDES v. 4.01

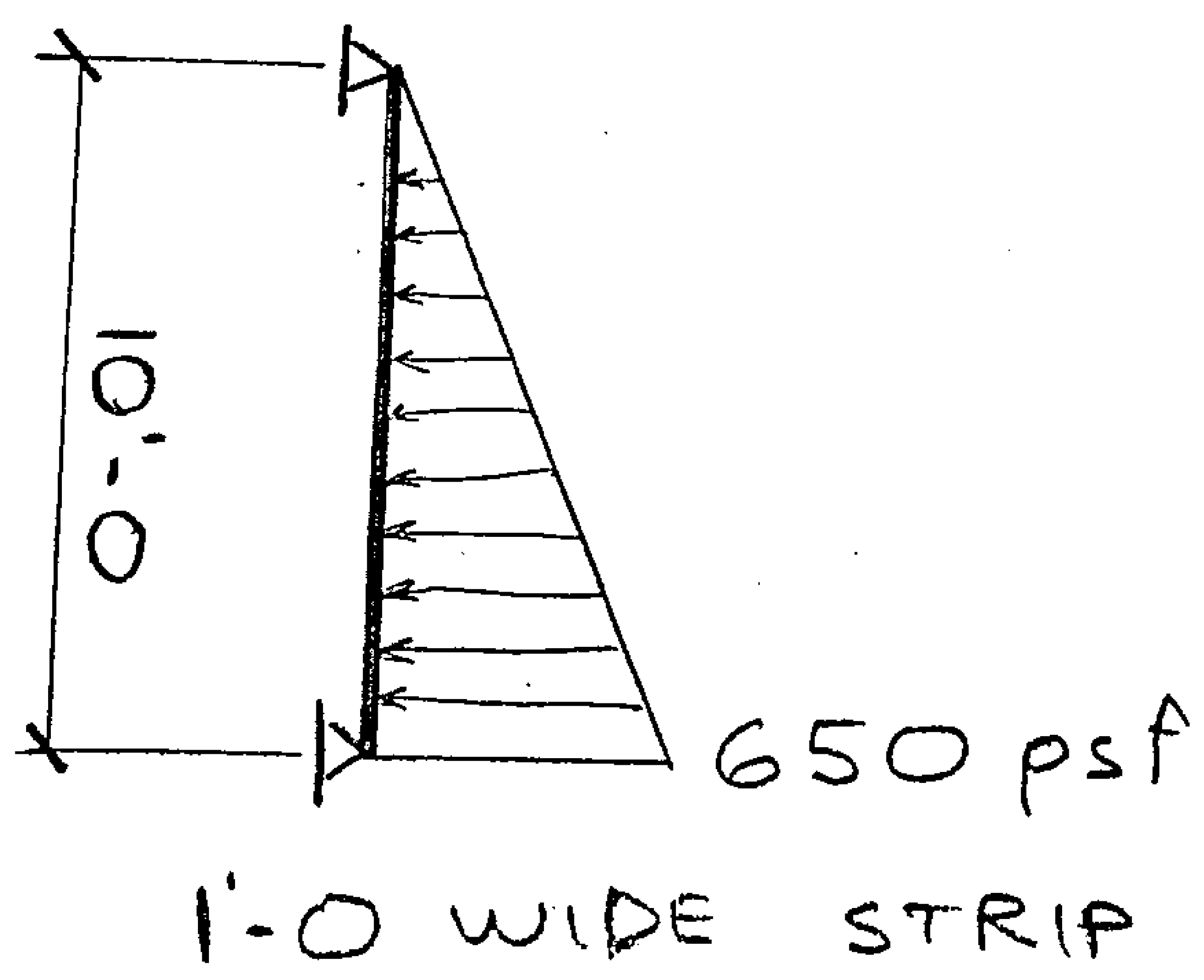
Date: 03-20-2002 Time: 10:54:49

Design Code: ACI 318-89 --- Data File: wall2

```
**************************************************
***  Wall - Detention Pond                 # 9919
**************************************************

***********************************
***  STRIP wall2  - INPUT DATA  ***
***********************************

No. of Joints                =  2
Upper Column Height             No Column
Gen. Lower Column Height     = 999.00  ft
-----------------------------------------
Concrete Strength            =   4.00  ksi
Concrete Weight              =    150  pcf
Flexural Steel Strength      =  60.00  ksi
Stirrup Steel  Strength      =  60.00  ksi
-----------------------------------------
Top Cover to CL Reinf.       =   2.00  in
Bottom Cover to CL Reinf.    =   2.00  in
-----------------------------------------
Modulus of Elasticity        =   3857  ksi
```



JOINT DATA

JT NO	Vert. Joint Freedom	Rota-tional Freedom	LOWER COLUMN Perp. (in)	LOWER COLUMN Paral. (in)	UPPER COLUMN Perp. (in)	UPPER COLUMN Paral. (in)	Upper Col-Ht. (ft)	Lower Col-Ht. (ft)
1	Supp	Free	0.00	0.00	-	-	-	999.00
2	Supp	Free	0.00	0.00	-	-	-	999.00

83/168

MEMBER TYPES

BM=Beam - SB=Slab Band - 1S=One-Way-Slab - 2SI/2SE=Two-Way-Slab (Int/Ext)

MB NO	MEMB TYPE	MB LGT (ft)	MEMB THICK-NESS (in)	WEB WIDTH (in)	FLG. THICK-NESS (in)	O.HANG FLANGE WIDTH (in)	SERV. DL (K/ft)	SERV. LL LC 1 (K/ft)
1	1S	10.00	10.00	12.00	0.00	0.00	0.00	0.00

UNIFORMLY VARYING LOADS (Serv.)

MB NO	DL W-LEFT (K/ft)	DL W-RIGHT (K/ft)	LL-LC 1 W-LEFT (K/ft)	LL-LC 1 W-RIGHT (K/ft)
1	0.00	0.00	0.65	0.00

****
*** STRIP wall2 - Factored END MOMENTS AND SHEARS ***
****

MB NO	L C	LEFT END MOM (Kft)	LEFT END SHEAR (K)	*	RIGHT END SHEAR (K)	RIGHT END MOM (Kft)
1	1	0.00	3.68	*	-1.84	0.00

84/168


```
******************************************
***  STRIP wall2  - FLEXURAL DESIGN  ***
******************************************
```

Check ACI 318-89 Clause 12.10.5 before terminating any bar in a Tension Zone
>> or << - Indicates that Reinforcing Steel shown is Code Minimum
99999 - Indicates a Section that cannot be reinforced for the calculated moment

MB NO	L C	FACT. LEFT DES MOM (Kft)	DIST TO INFL (ft)	LEFT MOM REINF (in2)	*	DIST TO MMAX (ft)	FACT. INT. MAX. MOMENT (Kft)	MOM REINF (in2)	*	RIGHT MOM REINF (in2)	DIST TO INFL (ft)	FACT. RIGHT DES MOM (Kft)
1	1	0.00	-	0.00	*	4.23	7.09	0.22<<	*	0.00	-	0.00

```
**************************************
***  STRIP wall2  - SHEAR DESIGN  ***
**************************************
```

- Shear Reinf. calcs. assume that no Flexural Reinf. is terminated in a Tension Zone otherwise increase the Shear Reinf. as per ACI 318-89 Clause 12.10.5
- 9999 - Indicates that the Concrete Shear exceeds maximum Code value

							Left	Right					
MB NO	L C	PHI*Vc (K)	*	DIST TO CRIT SECT (in)	ULT SHEAR CRIT SECT (K)	MAX SPAC (in)	Av/ft (in2/ft)	Av/ft (in2/ft)	MAX SPAC (in)	ULT SHEAR CRIT SECT (K)	DIST TO CRIT SECT (in)	*	PHI*Vc (K)
1	1	12.19	*	8.0	2.97	-	0.00	1.00	-	1.82	-8.0	*	12.19

```
**************************************
***  STRIP wall2  - SUPPORT LOADS  ***
**************************************
```

Loads positive downwards - Moments positive clockwise

JT NO	L C	SERVICE DEAD LOAD (K)	SERVICE LIVE LOAD (K)	FACTORED TOTAL LOAD (K)	FACT UPPER COLUMN MOM (KFT)	FACT LOWER COLUMN MOM (KFT)
1	1	0.00	2.17	3.68	0.00	0.00
2	1	0.00	1.08	1.84	0.00	0.00

```
************************************
***  STRIP wall2  - DEFLECTIONS  ***
************************************
```

- All Deflections are for service loads on the Cracked Section
- Refer to ACI 813-89 Table 9.5(b) for code limitations on Deflections
- Deflections positive downwards

SP NO	L C	MAX. AVER. Ig/Ie	DIST TO MAX DEFL (ft)	IMMEDIATE LL-DEFLECTION (in)	IMM DL-DEF (in)	ADD. DL-DEF (in)	IMM. LL-DEF + ADD. DL-DEF (in)
1	1	1.00	5.00	0.02 (L/999)	0.00	0.00	0.02 (L/999)

WESTGATE VILLAGE Date MAR 20.02 Designed DT
Date Checked

FOOTING

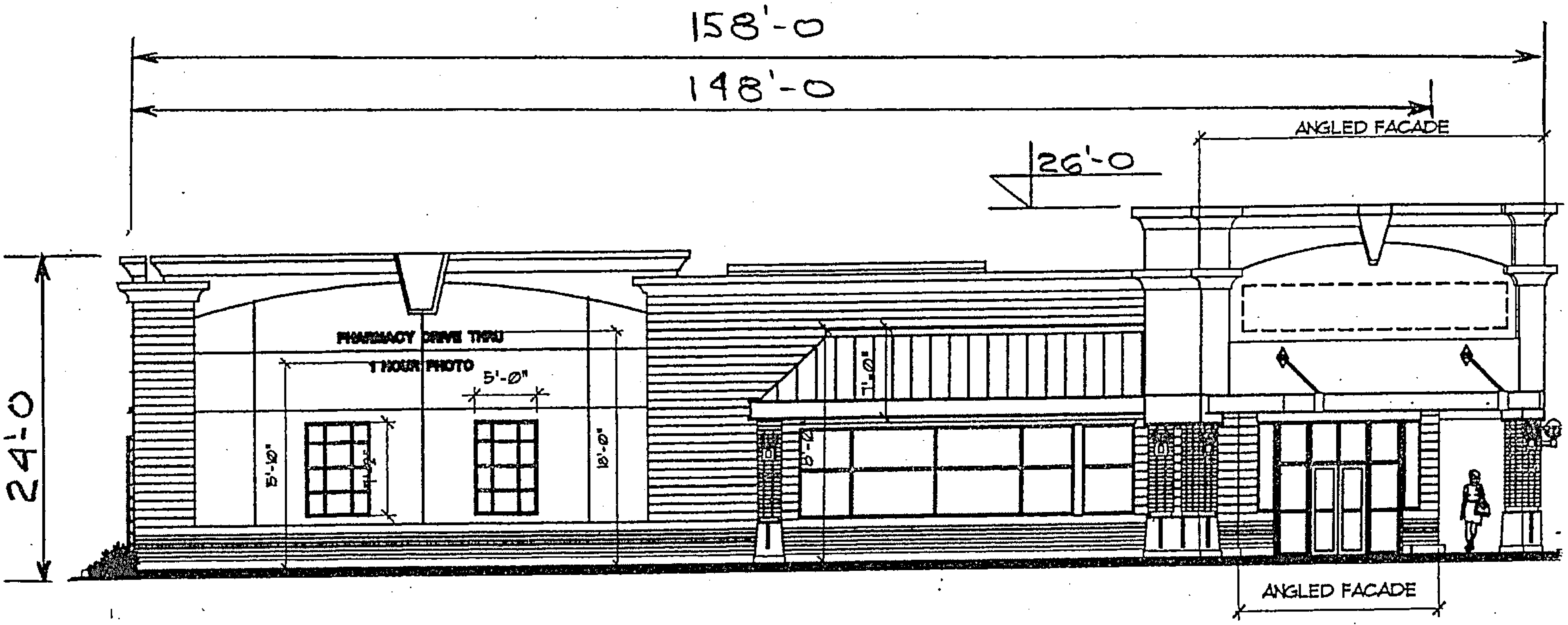
L.L.	250×9	$= 2250$	lb/ft
S.D.L.	210×9	$= 1890$	lb/ft
SLAB	$1.0 \times 150 \times 9$	$= 1350$	lb/ft
WALLS	$10 \times 10/12 \times 150$	$= 1250$	lb/ft
FOOTING	$3.0 \times 1.0 \times 150$	$= 450$	lb/ft
		$\Sigma = 7190$	lb/ft

$$f = \frac{7190}{3.0 \times 1.0} = 2396 \text{ psf}$$ O.K.

CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date. MAR 6, 02 Designed DT
Date Checked

WIND AND SEISMIC COMPERISON



EAST ELEVATION

A) WIND

- EXPOSURE B
- PROJECTED AREA METHOD

$$P = C_e \times C_q \times q_s \times I_w$$

$C_e = 0.76$ (TABLE 16-G ; UBC)

$C_q = 1.3$ (TABLE 16-H ; UBC)

BASIC WIND SPEED 80 MPH (TABLE 16-1 ; UBC)

$q_s = 16.4$ psf (TABLE 16-F ; UBC)

$I_w = 1.0$ (TABLE 16-K ; UBC)

$P = 0.76 \times 1.3 \times 16.4 \times 1.0 = 16.2$ psf

TOTAL WIND LOAD AT FOUND. LEVEL

$W = 24 \times 102 \times 16.2 = \underline{39657 \text{ lbs}}$ (↔ DIRECTION)

$W = 24 \times 158 \times 16.2 = \underline{61430 \text{ lbs}}$ (↕ DIRECTION)

$W_{F\,MAX} = 61430 \times 1.6 = \underline{98288 \text{ lbs}}$

actually should be ½ this to compare w/seismic

B) SEISMIC

interior walls mechanical equipt.? Spol?

WEIGHT OF THE BUILDING:

- ROOF 102 × 148 × 25 .... 377400 lbs
- CANOPY 10 × 120 × 20 .... 24000 lbs
- WALLS (PERP. TO SEISM. FORCE)

26/2 × 148 × 54 × 2 .... 207792 lbs

not grouted solid

W = 609192 lbs

WESTGATE VILLAGE Date MAR 6,02 Designed DT

Date Checked

-STATIC FORCE PROCEDURE SHALL BE USED

-SEISMIC ZONE 3

$E = \rho E_h + E_v$

$V = \frac{C_v I}{RT} W$

4.5 masonry

$R = 5.5$ (TABLE 16-N; UBC)

$I = 1.0$ (TABLE 16-K; UBC)

$Z = 0.3$ (TABLE 16-I; UBC); (SOIL REP)

check soils report

$C_v = 0.45$ (SOIL S_C; TABLE 16 R; UBC)
$C_a = 0.33$ (SOIL S_C; TABLE 16Q; UBC) (SOIL REP.)

$T = C_t (h_n)^{3/4}$ (METHOD A)

$C_t = 0.02$

$h_n = 22$ ft

$T = 0.02 \times 22^{3/4} = 0.203$ sec

$V = \frac{0.45 \times 1.0}{\cancel{5.5}\ 4.5 \times 0.203} \times 703\,000$

$V = 0.403$ W check R

$V_{max} = \frac{2.5 \times C_a I}{R\ 4.5} = 0.\cancel{15}\ .183$ W

$V_{min} = 0.11 C_a I W = 0.04$ W

ultimate

$V = 0.15$ W $= 0.\cancel{15} \times 609192 = \cancel{91378}$ 111685 lbs

WESTGATE VILLAGE Date MAR 6, 02 Designed DT
Date Checked

$$r_{max} = \frac{1}{2} = 0.5$$

$$\rho = 2 - \frac{20}{0.5\sqrt{102 \times 148}} = 1.67 \implies \rho = 1.5$$

$$E_v = 0.5 C_a I D = 0.5 \times 0.33 \times 1.0 D = 0.165D$$

$$E = 1.5V + 0.165D = V_f = 1.5V = 1.5 \times \cancel{91378} = 137062 \text{ lbs}$$

167,527

SEISMIC GOVERNS FOR ↔ DIRECTION

SEISMIC GOVERNS FOR ↕ DIRECTION

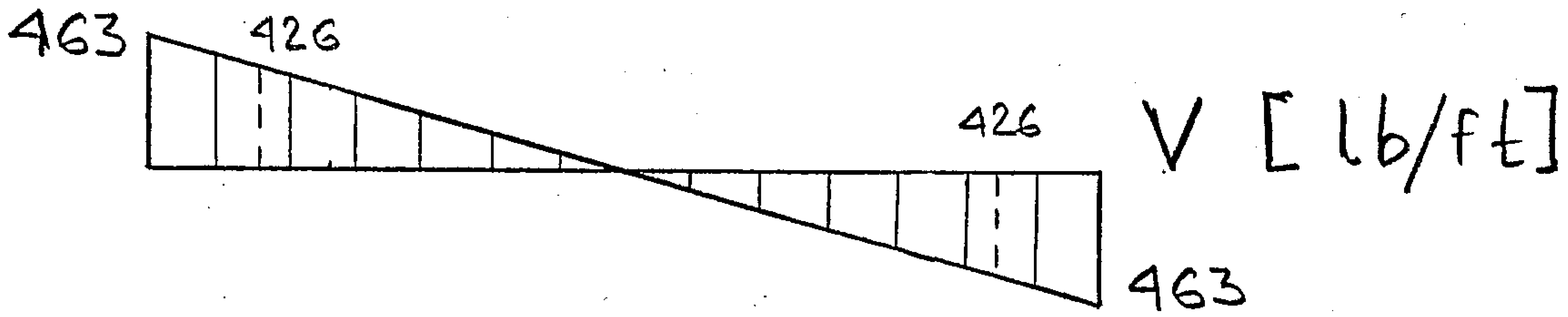
DIAPHRAGM

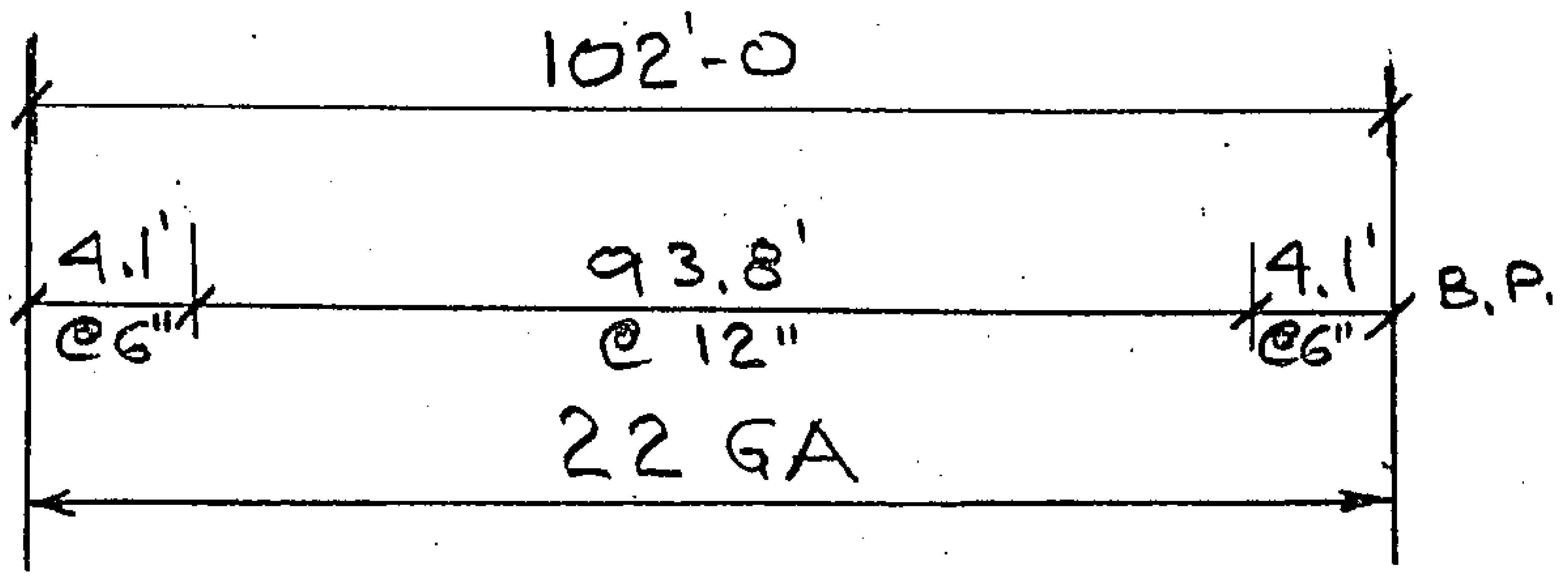
⇆ DIRECTION

$l = 148$ ft

$V = 137062$ lbs

$v = 137062/2/148 = 463$ lb/ft





- 22 GA DECK
- PUDDLE WELD @12"
- BUTTON PUNCH @ 6" (6'-0 @ SUPPORTS)
  @ 12" (MIDDLE)

CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date MAR 6, 02 Designed DT

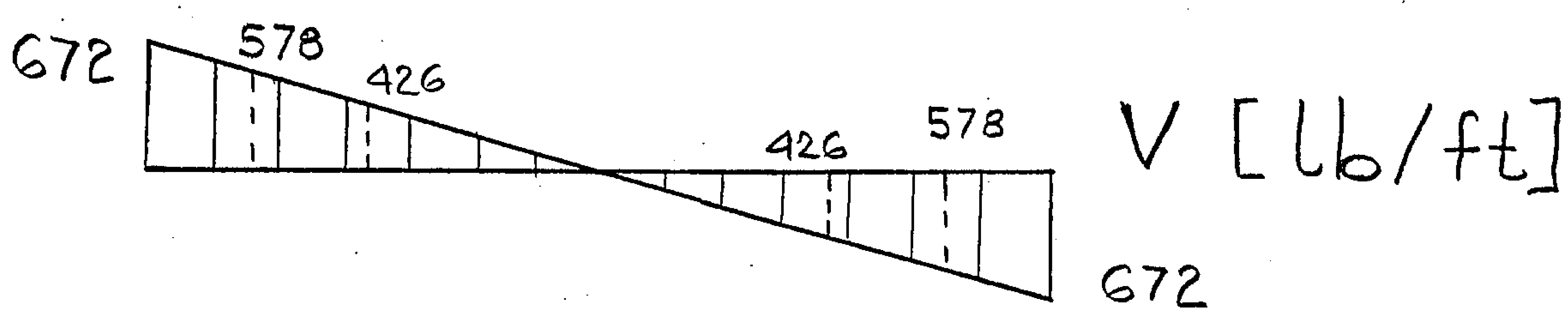
Date Checked

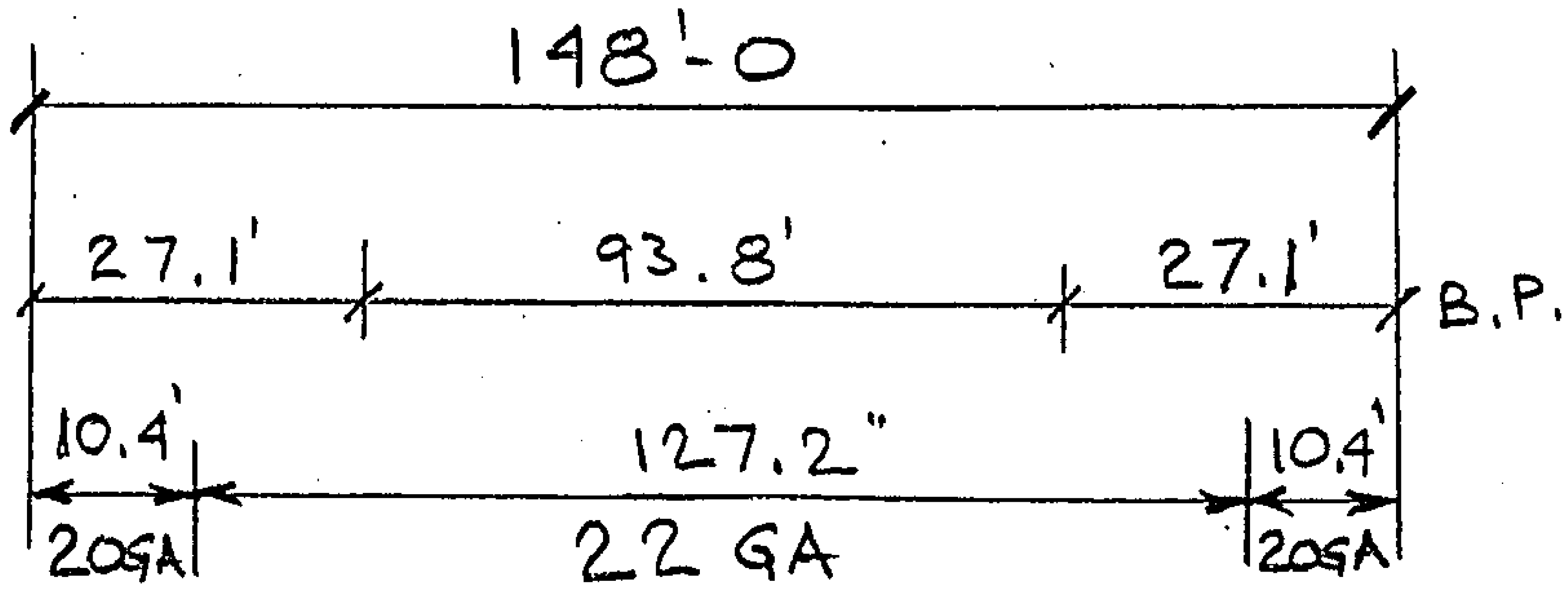
↕ DIRECTION

$l = 102$ ft

$V = 137062$ lbs

$v = 137062/2/102 = 672$ lb/ft



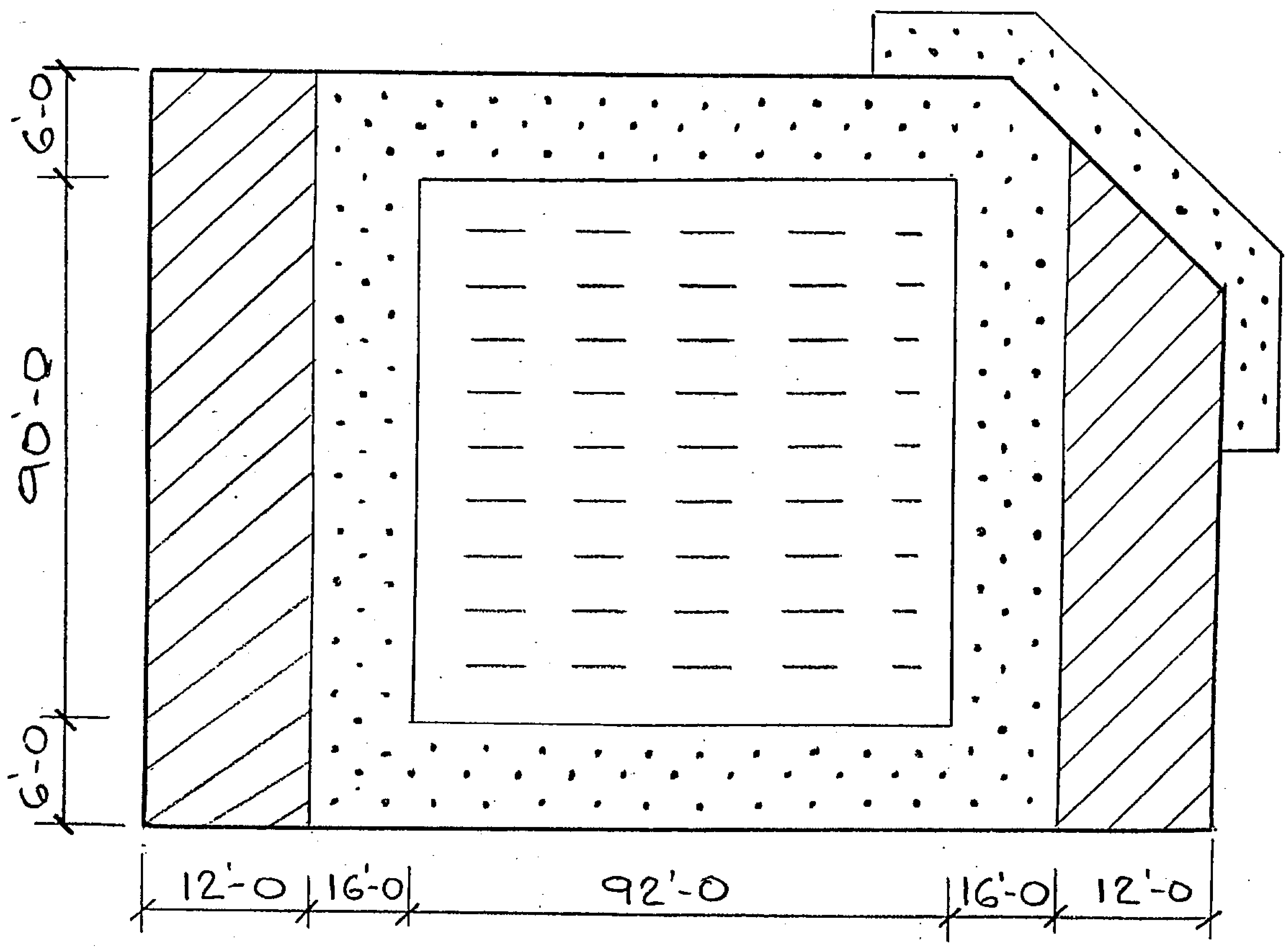


- 20 GA DECK (12'-0 @ SUPPORTS)
- 22 GA DECK (MIDDLE)
- PUDDLE WELD @ 12"
- BUTTON PUNCH @ 6" (28'-0 @ SUPPORT)
  @ 12" (MIDDLE)

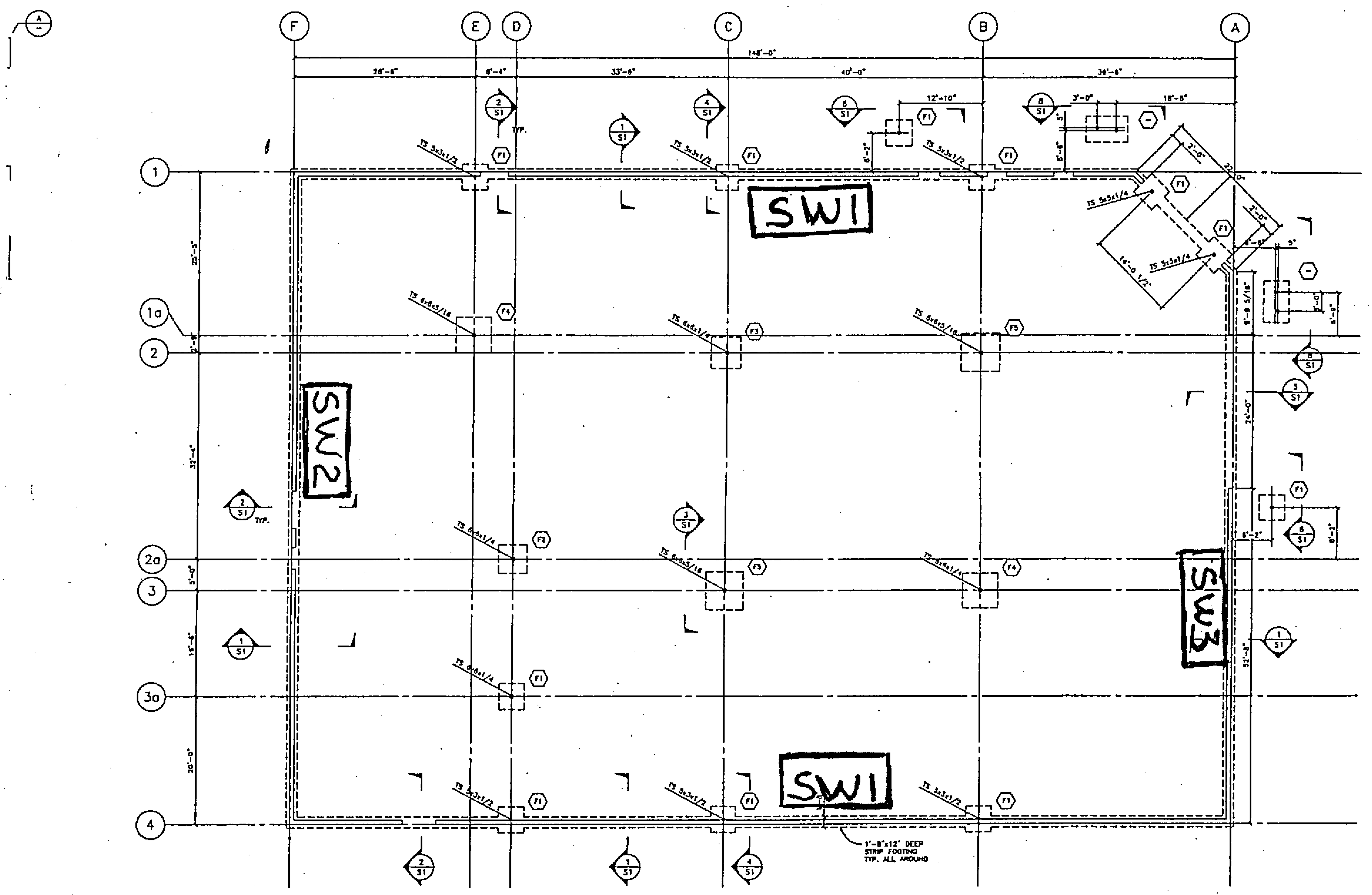
CWMM CONSULTING ENGINEERS LTD.

WESTGATE VILLAGE Date MAR 7, 02 Designed DT

Date Checked



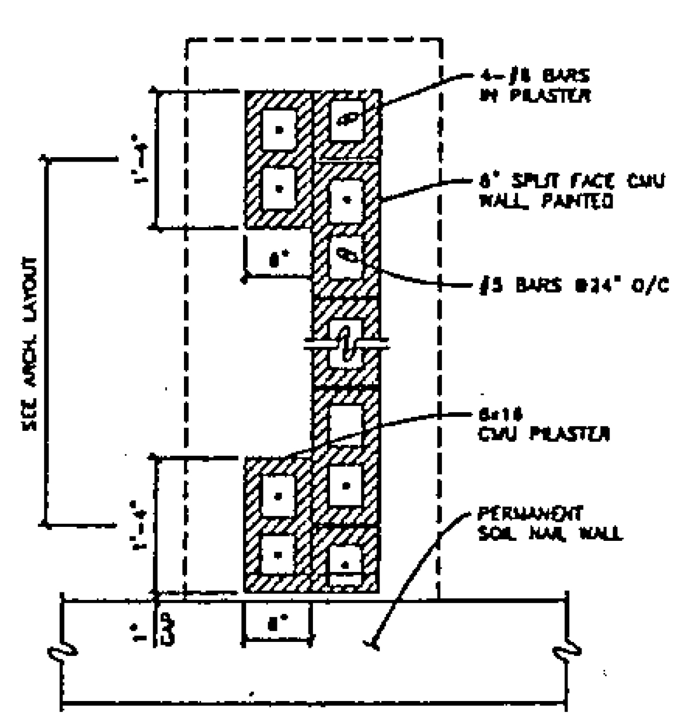
DIAPHRAGM SCHEDULE				
MARK	DECK	PUDDLE WELD	BUTTON PUNCH	NOTE
	20 GA	@12"	@6"	
	22 GA	@12"	@6"	
	22 GA	@12"	@12"	



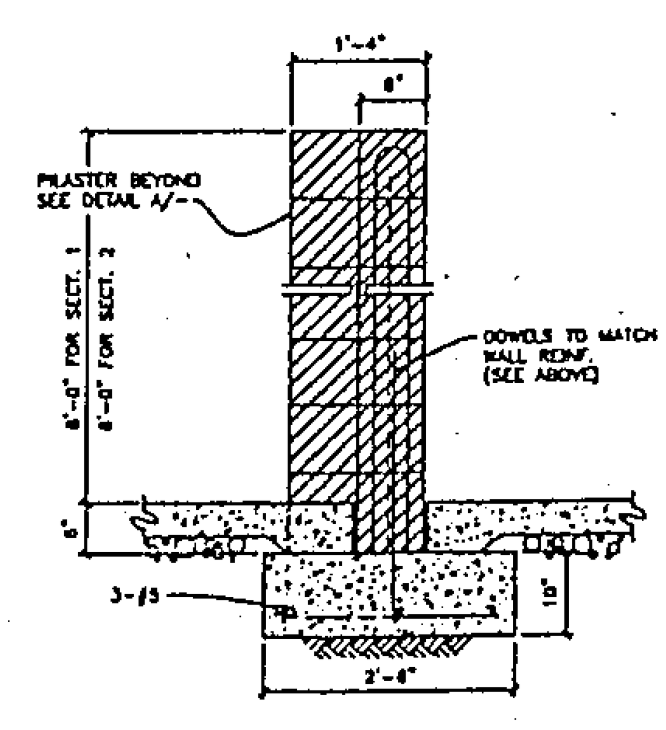
FOUNDATION PLAN
1/8"=1'-0"

FOOTING SCHEDULE

...TH)	BOTTOM REINFORCING EACH WAY (U.N.O.)	REMARKS
	[illegible]-#5	
	5-#5	
	6-#5	
	6-#5	
	7-#5	



DETAIL AT PILASTER A
3/4"=1'-0"



SECTION 1 2
3/4"=1'-0"

WESTGATE VILLAGE Date MAR 6, 02 Designed DT

Date Checked

SHEAR WALLS

SW1

ROOF PERP. WALLS

$W' = 25 \times 148 \times 102/2 + 102/2 \times 26/2 \times 2 \times 54 = 260304$ lbs

$E = 260304 \times 0.15 = 39046$ lbs

$L = 148$ ft

SW2

$W = 25 \times 148 \times 102/2 + 148/2 \times 26/2 \times 2 \times 54 = 292596$ lbs

$E = 292596 \times 0.15 = 43888$ lbs

$L = 102$ ft

SW3

CANOPY

$W = 292596 + 10 \times 120 \times 20 = 316596$ lbs

E $316596 \times 0.15 = 47488$ lbs

$L = 53$ ft

---

* WALLS WILL ALSO BE CHECKED AS SLENDER WALLS FOR WIND OF 16.2 psf AND SEISMIC (PART OF STRUCTURE)

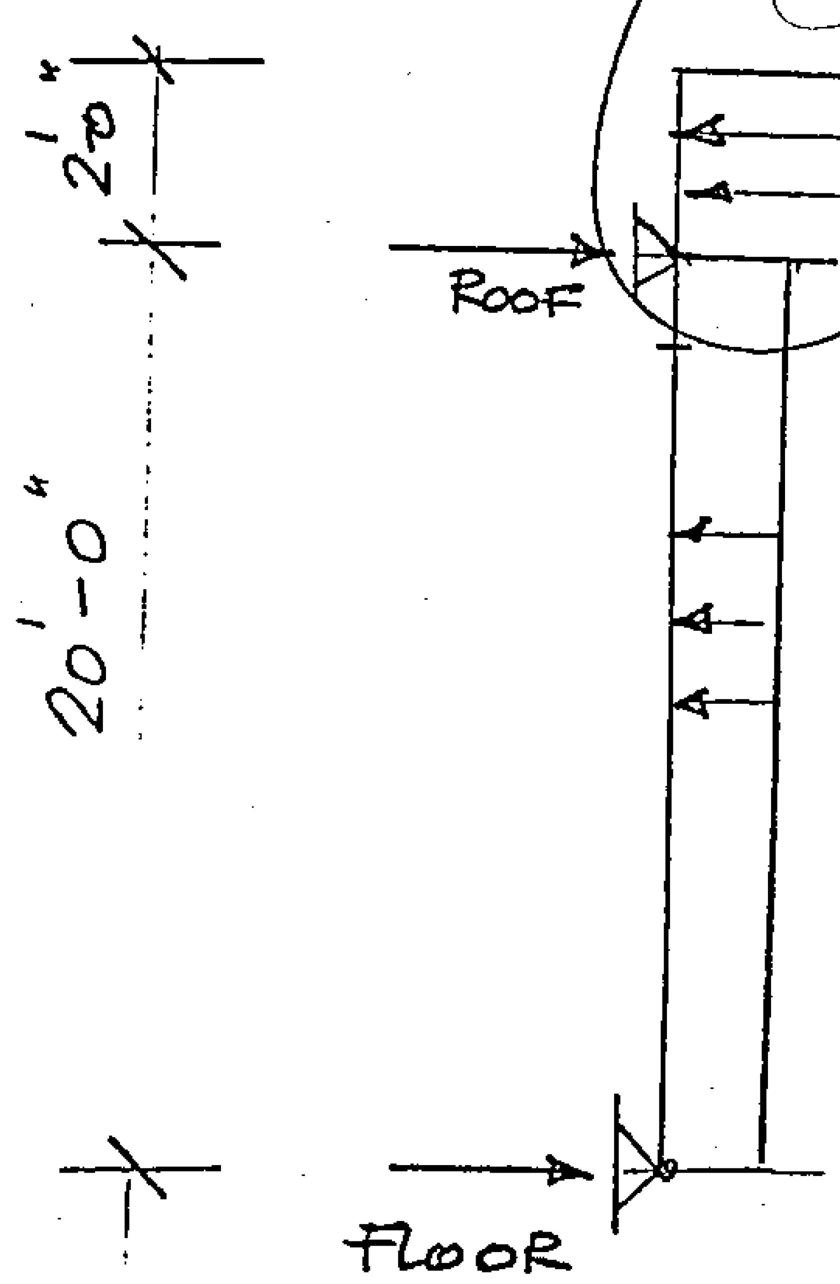
WESTGATE VILLAGE Date March 2002 Designed MP

Date Checked

SHEARWALL DESIGN

LOAD PERPENDICULAR TO WALL

A/ SEISMIC FORCE 4'?



$$F_p = \frac{a_p C_a I_p}{R_p}\left(1+3\frac{h_x}{h_r}\right) W_p$$

wall $a_p = 1.0$ parapet $a_p = 2.5$

$C_a = 0.33$ $h_r = 20 ft$

$R_p = 3.0$

$h_x = 20$ ft $W_p = 54$ psf

$$\Rightarrow F_{p1} = \frac{1.0\times0.33\times1.0}{3}\left(1+3\times\frac{20}{20}\right) W_p$$

$F_{p1} = 0.44 W_p$

$$F_{p2} = \frac{2.5\times0.33\times1.0}{3}(1+3) W_p = 1.10 W_p$$

$F_{p\,max} = 4.0 \; C_a I_p W_p = 4\times0.33\times1.0 \; W_p = 1.32 W_p$

$F_{p\,min} = 0.7 \; C_a I_p W_p = 0.7\times0.33\times1.0 W_p = 0.231 W_p$

$\Rightarrow$ WALL : $F_{p1} = 0.44\times54 \cong 24$ psf

$F_{p2} = 1.10\times54 \cong 59.4$ psf

96/168

===

M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Shear Wall

===

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Shear Wall SW1
Designer   : Dragan Todorovic
Date   : Apr 24, 02

Design Code : ACI 530-95/ASCE 5-95/TMS 402-95
Member Mark : SW1
Run No.   : 1
Remarks   : 0.9D + 1.0E

===

Loading and Overall Wall Geometry

=================================

Height	= 230 (in)			
Length	= 1776 (in)			
Gravity Load (Live)	=	0	(kip/ft)	Factor = 1
Gravity Load (Dead)	=	0	(kip/ft)	Factor = 0.9
Wall Weight	=	54	(psf)	Factor = 0.9
Moment	=	0	(kip-ft)	Factor = 1
Shear	=	39.046	(kip)	Factor = 1

Openings:

=========

None

Pier Design Check

=================

97/168

Pier No: 1

```
Geometry:
   left       = 0 (in)
   bottom     = 0 (in)
   width      = 1776 (in)
   height     = 230 (in)
   Pier stiffness ki      = 211825 (kip/in)
   Relative Stiffness Ri = 1

Loadings:
   Dead load                                = 153.195 (kip)
   Factored Axial Load (Compression) = 137.875 (kip)
   Factored Axial Load (Tension)       = 0 (kip)
   Factored Moment (in-plane)          = 748.382 (kip-ft)
   Factored Moment (out-of-plane)      = Not checked
   Factored Shear                      = 39.046 (kip)

Pier Classification and Properties:
   Masonry       : Concrete
   Construction : Hollow units, partially grouted
   Reinforcing  : Single layer
   Mortar        : Type S

   Wall thickness                            = 7.625 (in)
   Faceshell thickness                       = 1.25 (in)
   Masonry compressive strength f'm         = 2000 (psi)
   Masonry tensile strength ft              = 40 (psi)
   Grout spacing                             = 32 (in)
   Rebar Area, Ab                            = 0.31 (in^2)
   Rebar spacing                             = 32 (in)
   Steel yield strength fy                  = 60000 (psi)
   Rebar distribution                        = Uniformly distributed and
   Location of rebar from wall face, dc     = 3.8125 (in)
   Location of rebar from wall end, d_end = 4 (in)
   Total jamb steel area, As_jamb          = 0.31 (in^2)

Shear reinforcement:
```

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Joint reinforcement

Area, Av(wire) = 0.0326121 (in^2)
Spacing, s(wire) = 16 (in)
yield strength fy(wire) = 60000 (psi)

Bond beam steel

Area, Av(bond beam) = 0.620807 (in^2)
Spacing, s(bond beam) = 48 (in)
yield strength fy(bond beam) = 60000 (psi)

Factored Pier Capacity / Design Checks

Axial Load Resistance, Pr = 2205.43 (kip)
Moment Resistance (in-plane), Mr_y = 25259.4 (kip-ft)
Moment Resistance (out-of-plane), Mr_x = Not checked
Total Shear Resistance = 566.716 (kip)

Resistance of shear steel = 566.716 (kip)

Axial load - Moment Interaction -->OK
Shear resistance -->OK

99/168

M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Shear Wall

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Shear Wall SW1
Designer   : Dragan Todorovic
Date   : Apr 24, 02

Design Code : ACI 530-95/ASCE 5-95/TMS 402-95
Member Mark : SW1
Run No.   : 1
Remarks   : 1.2D + 1.0E

Loading and Overall Wall Geometry

Height	= 230 (in)			
Length	= 1776 (in)			
Gravity Load (Live)	=	0	(kip/ft)	Factor = 1
Gravity Load (Dead)	=	0	(kip/ft)	Factor = 1.2
Wall Weight	=	54	(psf)	Factor = 1.2
Moment	=	0	(kip-ft)	Factor = 1
Shear	=	39.046	(kip)	Factor = 1

Openings:

None

Pier Design Check

100/168

Pier No: 1

Geometry:

```
   left        = 0 (in)
   bottom      = 0 (in)
   width       = 1776 (in)
   height      = 230 (in)
   Pier stiffness ki       = 211825 (kip/in)
   Relative Stiffness Ri = 1
```

Loadings:

```
   Dead load                               = 153.195 (kip)
   Factored Axial Load (Compression) = 183.834 (kip)
   Factored Axial Load (Tension)      = 0 (kip)
   Factored Moment (in-plane)         = 748.382 (kip-ft)
   Factored Moment (out-of-plane)     = Not checked
   Factored Shear                     = 39.046 (kip)
```

Pier Classification and Properties:

```
   Masonry      : Concrete
   Construction : Hollow units, partially grouted
   Reinforcing  : Single layer
   Mortar       : Type S

   Wall thickness                              = 7.625 (in)
   Faceshell thickness                         = 1.25 (in)
   Masonry compressive strength f'm            = 2000 (psi)
   Masonry tensile strength ft                 = 40 (psi)
   Grout spacing                               = 32 (in)
   Rebar Area, Ab                              = 0.31 (in^2)
   Rebar spacing                               = 32 (in)
   Steel yield strength fy                     = 60000 (psi)
   Rebar distribution                          = Uniformly distributed and
   Location of rebar from wall face, dc        = 3.8125 (in)
   Location of rebar from wall end, d_end = 4 (in)
   Total jamb steel area, As_jamb              = 0.31 (in^2)
```

Shear reinforcement:

```
Joint reinforcement
-------------------
Area, Av(wire)             = 0.0326121 (in^2)
Spacing, s(wire)           = 16 (in)
yield strength fy(wire) = 60000 (psi)

Bond beam steel
---------------
Area, Av(bond beam)             = 0.620807 (in^2)
Spacing, s(bond beam)           = 48 (in)
yield strength fy(bond beam) = 60000 (psi)


Factored Pier Capacity / Design Checks
--------------------------------------

Axial Load Resistance, Pr                        = 2205.43 (kip)
Moment Resistance (in-plane), Mr_y               = 27814.4 (kip-ft)
Moment Resistance (out-of-plane), Mr_x           = Not checked
Total Shear Resistance                           = 566.716 (kip)

Resistance of shear steel                        = 566.716 (kip)

Axial load - Moment Interaction -->OK
Shear resistance -->OK
```

102/168

===
M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)
Fero Corporation

Masonry Slender Wall
===

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Slender Wall SLW1
Designer   : Dragan Todorovic
Date   : Mart 6,02

Design Code : Uniform Building Code
Member Mark : SLW1
Run No.   : 1
Remarks   : 0.9D + 1.0E
===

Overall Wall Geometry
=====================

Total height	= 288 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base
=================================

Floor/Support	Height (in)
1	230

Wall section
============

Section	From h (in)	To h (in)
1	0	230
2	230	288

103/168

Floor Loads

Floor No:1

Height above base =	230	(in)	
Live load =	0	(kip/ft)	Factor = 1.7
Dead load =	0	(kip/ft)	Factor = 1.4
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	230	-24	1
230	288	-59.4	1

Note:
1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	230	54	0.9
230	288	54.2969	0.9

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.193646	1.50055e-15
Floor No:1	0.553525	1.50055e-15

Wall Classification and Properties

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Wall Section No:1

========================

```
Masonry          : Concrete
Construction     : Hollow units, partially grouted
Reinforcing      : Single layer
Mortar           : Type S

Wall thickness                      = 7.625 (in)
Faceshell thickness                 = 1.25 (in)
Masonry compressive strength f'm    = 2000 (psi)
Masonry tensile strength ft         = 111.803 (psi)
Grout spacing                       = 32 (in)
Rebar spacing                       = 32 (in)
Steel yield strength fy             = 60000 (psi)
Bar area                            = 0.31 (in^2)
Number of bar layer                 = 1
Bar location (from wall face)       = 3.8125 (in)
Minimum horizontal steel for seismic zone 2 and higher:
                              Ah = 0.06675 (in^2/ft)
```

Wall Section No:2

========================

```
Masonry          : Concrete
Construction     : Hollow units, partially grouted
Reinforcing      : Single layer
Mortar           : Type S

Wall thickness                      = 7.625 (in)
Faceshell thickness                 = 1.25 (in)
Masonry compressive strength f'm    = 2000 (psi)
Masonry tensile strength ft         = 111.803 (psi)
Grout spacing                       = 32 (in)
Rebar spacing                       = 32 (in)
Steel yield strength fy             = 60000 (psi)
Bar area                            = 0.31 (in^2)
Number of bar layer                 = 1
Bar location (from wall face)       = 3.8125 (in)
Minimum horizontal steel for seismic zone 2 and higher:
                              Ah = 0.06675 (in^2/ft)
```

Wall Design Checks

Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf	= 0.764115 (kip/ft)
Design moment, Mf	= 0.786733 (kip-ft/ft)
Factored resistance, Mr	= 1.86895 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 62.5276 (kip/ft)

*** Section adequate ***

Wall Section 2

Factored load, Pf	= 0.236214 (kip/ft)
Design moment, Mf	= 0.693629 (kip-ft/ft)
Factored resistance, Mr	= 1.74758 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 68.0266 (kip/ft)

*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

106/168

Wall Deflection

===============

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	230	0.0744028
2	230	288	0.0245763

107/168

==

M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Shear Wall

==

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Shear Wall SW2
Designer   : Dragan Todorovic
Date   : Apr 24,02

Design Code : ACI 530-95/ASCE 5-95/TMS 402-95
Member Mark : SW2
Run No.   : 1
Remarks   : 0.9D + 1.0E

==

Loading and Overall Wall Geometry

================================

Height	= 204 (in)		
Length	= 1224 (in)		
Gravity Load (Live)	=	0 (kip/ft)	Factor = 0
Gravity Load (Dead)	=	0.435 (kip/ft)	Factor = 0.9
Wall Weight	=	54 (psf)	Factor = 0.9
Moment	=	0 (kip-ft)	Factor = 0
Shear	=	43.888 (kip)	Factor = 1

Openings:

=========

None

Pier Design Check

================

108/168

Pier No: 1

Geometry:

```
    left       = 0 (in)
    bottom     = 0 (in)
    width      = 1224 (in)
    height     = 204 (in)
    Pier stiffness ki       = 163996 (kip/in)
    Relative Stiffness Ri = 1
```

Loddings:

```
    Dead load                              = 138.015 (kip)
    Factored Axial Load (Compression)      = 124.213 (kip)
    Factored Axial Load (Tension)          = 0 (kip)
    Factored Moment (in-plane)             = 746.096 (kip-ft)
    Factored Moment (out-of-plane)         = Not checked
    Factored Shear                         = 43.888 (kip)
```

Pier Classification and Properties:

```
    Masonry      : Concrete
    Construction : Hollow units, partially grouted
    Reinforcing  : Single layer
    Mortar       : Type S

    Wall thickness                                 = 7.625 (in)
    Faceshell thickness                            = 1.25 (in)
    Masonry compressive strength f'm               = 2000 (psi)
    Masonry tensile strength ft                    = 40 (psi)
    Grout spacing                                  = 32 (in)
    Rebar Area, Ab                                 = 0.31 (in^2)
    Rebar spacing                                  = 32 (in)
    Steel yield strength fy                        = 60000 (psi)
    Rebar distribution                             = Uniformly distributed and
    Location of rebar from wall face, dc           = 3.8125 (in)
    Location of rebar from wall end, d_end         = 4 (in)
    Total jamb steel area, As_jamb                 = 0.31 (in^2)
```

Shear reinforcement:

```
Joint reinforcement
-------------------
Area, Av(wire)            = 0.0326121 (in^2)
Spacing, s(wire)          = 16 (in)
yield strength fy(wire) = 60000 (psi)

Bond beam steel
---------------
Area, Av(bond beam)           = 0.620807 (in^2)
Spacing, s(bond beam)         = 48 (in)
yield strength fy(bond beam) = 60000 (psi)


Factored Pier Capacity / Design Checks
--------------------------------------

Axial Load Resistance, Pr                        = 1735.48 (kip)
Moment Resistance (in-plane), Mr_y               = 13196.8 (kip-ft)
Moment Resistance (out-of-plane), Mr_x           = Not checked
Total Shear Resistance                           = 390.177 (kip)

Resistance of shear steel                        = 390.177 (kip)

Axial load - Moment Interaction -->Inadequate/Not checked
Shear resistance -->Inadequate/Not checked
```

110/168

===
M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)
Fero Corporation

Masonry Shear Wall
===

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Shear Wall SW2
Designer   : Dragan Todorovic
Date   : Apr 24,02

Design Code : ACI 530-95/ASCE 5-95/TMS 402-95
Member Mark : SW2
Run No.   : 1
Remarks   : 1.2D + 1.0E + 0.5S
===

Loading and Overall Wall Geometry
=================================

Height	= 204 (in)			
Length	= 1224 (in)			
Gravity Load (Live)	=	1.045	(kip/ft)	Factor = 0.5
Gravity Load (Dead)	=	0.435	(kip/ft)	Factor = 1.2
Wall Weight	=	54	(psf)	Factor = 1.2
Moment	=	0	(kip-ft)	Factor = 0
Shear	=	43.888	(kip)	Factor = 1

Openings:
=========

None

Pier Design Check
=================

111/168

```
------------
Pier No: 1
------------

    Geometry:
       left       = 0 (in)
       bottom     = 0 (in)
       width      = 1224 (in)
       height     = 204 (in)
       Pier stiffness ki      = 163996 (kip/in)
       Relative Stiffness Ri = 1

    Loadings:
       Dead load                                = 138.015 (kip)
       Factored Axial Load (Compression) = 218.913 (kip)
       Factored Axial Load (Tension)      = 0 (kip)
       Factored Moment (in-plane)          = 746.096 (kip-ft)
       Factored Moment (out-of-plane)      = Not checked
       Factored Shear                          = 43.888 (kip)

    Pier Classification and Properties:
       Masonry       : Concrete
       Construction : Hollow units, partially grouted
       Reinforcing   : Single layer
       Mortar        : Type S

       Wall thickness                              = 7.625 (in)
       Faceshell thickness                         = 1.25 (in)
       Masonry compressive strength f'm            = 2000 (psi)
       Masonry tensile strength ft                 = 40 (psi)
       Grout spacing                               = 32 (in)
       Rebar Area, Ab                              = 0.31 (in^2)
       Rebar spacing                               = 32 (in)
       Steel yield strength fy                     = 60000 (psi)
       Rebar distribution                          = Uniformly distributed and
       Location of rebar from wall face, dc        = 3.8125 (in)
       Location of rebar from wall end, d_end = 4 (in)
       Total jamb steel area, As_jamb              = 0.31 (in^2)

    Shear reinforcement:
```

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Joint reinforcement

```
Area, Av(wire)            = 0.0326121 (in^2)
Spacing, s(wire)          = 16 (in)
yield strength fy(wire)   = 60000 (psi)
```

Bond beam steel

```
Area, Av(bond beam)            = 0.620807 (in^2)
Spacing, s(bond beam)          = 48 (in)
yield strength fy(bond beam)   = 60000 (psi)
```

Factored Pier Capacity / Design Checks

```
Axial Load Resistance, Pr                              = 1735.48 (kip)
Moment Resistance (in-plane), Mr_y                     = 16726.2 (kip-ft)
Moment Resistance (out-of-plane), Mr_x                 = Not checked
Total Shear Resistance                                 = 390.177 (kip)

Resistance of shear steel                              = 390.177 (kip)

Axial load - Moment Interaction -->OK
Shear resistance -->OK
```

113/168

===

M A S O N R Y D E S I G N S O F T W A R E 2 . 0 (C)

Fero Corporation

Masonry Slender Wall

===

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Slender Wall SLW2
Designer : Dragan Todorovic
Date : Apr 25, 02

Design Code : Uniform Building Code
Member Mark : SLW2
Run No. : 1
Remarks : 1.4D

===

Overall Wall Geometry
=====================

Total height	= 264 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base
=================================

Floor/Support	Height (in)
1	204

Wall section
============

Section	From h (in)	To h (in)
1	0	204
2	204	264

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Floor Loads

Floor No:1

Height above base =	204	(in)	
Live load =	0	(kip/ft)	Factor = 1.7
Dead load =	0.435	(kip/ft)	Factor = 1.4
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	204	0	1.3
204	264	0	1.3

Note:

1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	204	54	1.4
204	264	54	1.4

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.00297818	1.48951e-16
Floor No:1	-0.00297818	1.48951e-16

Wall Classification and Properties

```
Wall Section No:1
========================
Masonry           : Concrete
Construction      : Hollow units, partially grouted
Reinforcing       : Single layer
Mortar            : Type S

Wall thickness                    = 7.625 (in)
Faceshell thickness               = 1.25 (in)
Masonry compressive strength f'm = 2000 (psi)
Masonry tensile strength ft       = 111.803 (psi)
Grout spacing                     = 32 (in)
Rebar spacing                     = 32 (in)
Steel yield strength fy           = 60000 (psi)
Bar area                          = 0.31 (in^2)
Number of bar layer               = 1
Bar location (from wall face)     = 3.8125 (in)


Wall Section No:2
========================
Masonry           : Concrete
Construction      : Hollow units, partially grouted
Reinforcing       : Single layer
Mortar            : Type S

Wall thickness                    = 7.625 (in)
Faceshell thickness               = 1.25 (in)
Masonry compressive strength f'm = 2000 (psi)
Masonry tensile strength ft       = 111.803 (psi)
Grout spacing                     = 32 (in)
Rebar spacing                     = 32 (in)
Steel yield strength fy           = 60000 (psi)
Bar area                          = 0.31 (in^2)
Number of bar layer               = 1
Bar location (from wall face)     = 3.8125 (in)


Wall Design Checks
==================
```

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Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf	= 0.987036 (kip/ft)
Design moment, Mf	= 0.0508138 (kip-ft/ft)
Factored resistance, Mr	= 1.92002 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 62.5276 (kip/ft)

*** Section adequate ***

Wall Section 2

Factored load, Pf	= 0.378036 (kip/ft)
Design moment, Mf	= 6.37753e-05 (kip-ft/ft)
Factored resistance, Mr	= 1.78016 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 68.0266 (kip/ft)

*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	204	0.00189279

2	204	264	0.00288554

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M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Slender Wall

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Slender Wall SLW2
Designer : Dragan Todorovic
Date : Apr 25, 02

Design Code : Uniform Building Code
Member Mark : SLW2
Run No. : 1
Remarks : 1.2D + 0.5S

Overall Wall Geometry

Total height	= 264 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

Floor/Support	Height (in)
1	204

Wall section

Section	From h (in)	To h (in)
1	0	204
2	204	264

Floor Loads

Floor No:1

Height above base =	204	(in)	
Live load =	1.045	(kip/ft)	Factor = 0.5
Dead load =	0.435	(kip/ft)	Factor = 1.2
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	204	0	1.3
204	264	0	1.3

Note:

1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	204	54	1.2
204	264	54	1.2

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.00510962	1.09796e-16
Floor No:1	-0.00510962	1.09796e-16

Wall Classification and Properties

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```
Wall Section No:1
========================
Masonry             : Concrete
Construction        : Hollow units, partially grouted
Reinforcing         : Single layer
Mortar              : Type S

Wall thickness                        = 7.625 (in)
Faceshell thickness                   = 1.25 (in)
Masonry compressive strength f'm      = 2000 (psi)
Masonry tensile strength ft           = 111.803 (psi)
Grout spacing                         = 32 (in)
Rebar spacing                         = 32 (in)
Steel yield strength fy               = 60000 (psi)
Bar area                              = 0.31 (in^2)
Number of bar layer                   = 1
Bar location (from wall face)         = 3.8125 (in)


Wall Section No:2
========================
Masonry             : Concrete
Construction        : Hollow units, partially grouted
Reinforcing         : Single layer
Mortar              : Type S

Wall thickness                        = 7.625 (in)
Faceshell thickness                   = 1.25 (in)
Masonry compressive strength f'm      = 2000 (psi)
Masonry tensile strength ft           = 111.803 (psi)
Grout spacing                         = 32 (in)
Rebar spacing                         = 32 (in)
Steel yield strength fy               = 60000 (psi)
Bar area                              = 0.31 (in^2)
Number of bar layer                   = 1
Bar location (from wall face)         = 3.8125 (in)


Wall Design Checks
==================
```

Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf = 1.36853 (kip/ft)
Design moment, Mf = 0.0871355 (kip-ft/ft)
Factored resistance, Mr = 2.00655 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 62.5276 (kip/ft)
*** Section adequate ***

Wall Section 2

Factored load, Pf = 0.324031 (kip/ft)
Design moment, Mf = 9.38662e-05 (kip-ft/ft)
Factored resistance, Mr = 1.76775 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 68.0266 (kip/ft)
*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	204	0.00648619

2	204	264	0.00986462

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==

M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Slender Wall

==

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Slender Wall SLW2
Designer : Dragan Todorovic
Date : Apr 25, 02

Design Code : Uniform Building Code
Member Mark : SLW2
Run No. : 1
Remarks : 1.2D + 1.6S + 0.8W

==

Overall Wall Geometry
=====================

Total height = 264 (in)
Number of wall sections (including parapet if applicable) = 2
Number of floors / lateral supports = 1
Boundary condition (base) = Pinned

Height of support above wall base
=================================

Floor/Support	Height (in)
1	204

Wall section
============

Section	From h (in)	To h (in)
1	0	204
2	204	264

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Floor Loads
===========

Floor No:1

Height above base =	204	(in)		
Live load =	1.045	(kip/ft)	Factor = 1.6	
Dead load =	0.435	(kip/ft)	Factor = 1.2	
Load eccentricity =	1	(in)		

Lateral Loads
=============

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	204	-16.2	0.8
204	264	-16.2	0.8

Note:
1. Negative (-ve) indicates loading directed away from wall

Wall Weight
===========

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	204	54	1.2
204	264	54	1.2

Support Reactions
=================

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.111282	3.46257e-16
Floor No:1	0.173865	3.46257e-16

Wall Classification and Properties
==================================

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Wall Section No:1
========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	=	7.625 (in)
Faceshell thickness	=	1.25 (in)
Masonry compressive strength f'm	=	2000 (psi)
Masonry tensile strength ft	=	111.803 (psi)
Grout spacing	=	32 (in)
Rebar spacing	=	32 (in)
Steel yield strength fy	=	60000 (psi)
Bar area	=	0.31 (in^2)
Number of bar layer	=	1
Bar location (from wall face)	=	3.8125 (in)

Wall Section No:2
========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	=	7.625 (in)
Faceshell thickness	=	1.25 (in)
Masonry compressive strength f'm	=	2000 (psi)
Masonry tensile strength ft	=	111.803 (psi)
Grout spacing	=	32 (in)
Rebar spacing	=	32 (in)
Steel yield strength fy	=	60000 (psi)
Bar area	=	0.31 (in^2)
Number of bar layer	=	1
Bar location (from wall face)	=	3.8125 (in)

Wall Design Checks
==================

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Moment-Axial Load Interaction
=============================

Wall Section 1

Factored load, Pf = 3.06888 (kip/ft)
Design moment, Mf = 0.489443 (kip-ft/ft)
Factored resistance, Mr = 2.38769 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 62.5276 (kip/ft)
*** Section adequate ***

Wall Section 2

Factored load, Pf = 0.324031 (kip/ft)
Design moment, Mf = 0.161512 (kip-ft/ft)
Factored resistance, Mr = 1.76775 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 68.0266 (kip/ft)
*** Section adequate ***

Wall Shear
==========

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection
===============

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	204	0.0459355

2	204	264	0.0372695

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M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Slender Wall

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Slender Wall SLW2
Designer   : Dragan Todorovic
Date   : Apr 25,02

Design Code : Uniform Building Code
Member Mark : SLW2
Run No.   : 1
Remarks   : 1.2D + 0.5S + 1.0E

Overall Wall Geometry

Total height	= 264 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

Floor/Support	Height (in)
1	204

Wall section

Section	From h (in)	To h (in)
1	0	204
2	204	264

Floor Loads

Floor No:1

Height above base =	204	(in)	
Live load =	1.045	(kip/ft)	Factor = 0.5
Dead load =	0.435	(kip/ft)	Factor = 1.2
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	204	-24	1
204	264	-59.4	1

Note:

1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	204	54.2969	1.2
204	264	54.2969	1.2

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.165329	4.8751e-15
Floor No:1	0.539738	4.8751e-15

Wall Classification and Properties

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Wall Section No:1
========================

```
Masonry            : Concrete
Construction       : Hollow units, partially grouted
Reinforcing        : Single layer
Mortar             : Type S

Wall thickness                     = 7.625 (in)
Faceshell thickness                = 1.25 (in)
Masonry compressive strength f'm = 2000 (psi)
Masonry tensile strength ft        = 111.803 (psi)
Grout spacing                      = 32 (in)
Rebar spacing                      = 32 (in)
Steel yield strength fy            = 60000 (psi)
Bar area                           = 0.31 (in^2)
Number of bar layer                = 1
Bar location (from wall face)      = 3.8125 (in)
```

Wall Section No:2
========================

```
Masonry            : Concrete
Construction       : Hollow units, partially grouted
Reinforcing        : Single layer
Mortar             : Type S

Wall thickness                     = 7.625 (in)
Faceshell thickness                = 1.25 (in)
Masonry compressive strength f'm = 2000 (psi)
Masonry tensile strength ft        = 111.803 (psi)
Grout spacing                      = 32 (in)
Rebar spacing                      = 32 (in)
Steel yield strength fy            = 60000 (psi)
Bar area                           = 0.31 (in^2)
Number of bar layer                = 1
Bar location (from wall face)      = 3.8125 (in)
```

Wall Design Checks
==================

Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf = 1.37031 (kip/ft)
Design moment, Mf = 0.655434 (kip-ft/ft)
Factored resistance, Mr = 2.00695 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 62.5276 (kip/ft)
*** Section adequate ***

Wall Section 2

Factored load, Pf = 0.325813 (kip/ft)
Design moment, Mf = 0.742476 (kip-ft/ft)
Factored resistance, Mr = 1.76816 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 68.0266 (kip/ft)
*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	204	0.0427792

2	204	264	0.00753061

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==

M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Slender Wall

==

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Slender Wall SLW2
Designer   : Dragan Todorovic
Date   : Apr 25,02

Design Code : Uniform Building Code
Member Mark : SLW2
Run No.   : 1
Remarks   : 0.9D + 1.0E

==

Overall Wall Geometry

=====================

Total height	= 264 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

=================================

Floor/Support	Height (in)
1	204

Wall section

============

Section	From h (in)	To h (in)
1	0	204
2	204	264

Floor Loads

Floor No:1

Height above base =	204	(in)	
Live load =	0	(kip/ft)	Factor = 0
Dead load =	0.435	(kip/ft)	Factor = 0.9
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	204	-24	1
204	264	-59.4	1

Note:

1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	204	54	0.9
204	264	54	0.9

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.162167	3.08892e-15
Floor No:1	0.542901	3.08892e-15

Wall Classification and Properties

```
Wall Section No:1
========================
Masonry          : Concrete
Construction     : Hollow units, partially grouted
Reinforcing      : Single layer
Mortar           : Type S

Wall thickness                      = 7.625 (in)
Faceshell thickness                 = 1.25 (in)
Masonry compressive strength f'm = 2000 (psi)
Masonry tensile strength ft         = 111.803 (psi)
Grout spacing                       = 32 (in)
Rebar spacing                       = 32 (in)
Steel yield strength fy             = 60000 (psi)
Bar area                            = 0.31 (in^2)
Number of bar layer                 = 1
Bar location (from wall face)       = 3.8125 (in)


Wall Section No:2
========================
Masonry          : Concrete
Construction     : Hollow units, partially grouted
Reinforcing      : Single layer
Mortar           : Type S

Wall thickness                      = 7.625 (in)
Faceshell thickness                 = 1.25 (in)
Masonry compressive strength f'm = 2000 (psi)
Masonry tensile strength ft         = 111.803 (psi)
Grout spacing                       = 32 (in)
Rebar spacing                       = 32 (in)
Steel yield strength fy             = 60000 (psi)
Bar area                            = 0.31 (in^2)
Number of bar layer                 = 1
Bar location (from wall face)       = 3.8125 (in)


Wall Design Checks
==================
```

Moment-Axial Load Interaction
==============================

Wall Section 1

Factored load, Pf = 0.634523 (kip/ft)
Design moment, Mf = 0.709921 (kip-ft/ft)
Factored resistance, Mr = 1.83917 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 62.5276 (kip/ft)
*** Section adequate ***

Wall Section 2

Factored load, Pf = 0.243023 (kip/ft)
Design moment, Mf = 0.742546 (kip-ft/ft)
Factored resistance, Mr = 1.74915 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 68.0266 (kip/ft)
*** Section adequate ***

Wall Shear
==========

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection
===============

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	204	0.0381859

2	204	264	0.00211711

===

M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Shear Wall

===

Project   : WESTGATE VILLAGE
Job No.   : 9919
Description: Shear Wall SW3
Designer   : Dragan Todorovic
Date   : Apr 24, 02

Design Code : ACI 530-95/ASCE 5-95/TMS 402-95
Member Mark : SW3
Run No.   : 1
Remarks   : 0.9D + 1.0E

===

Loading and Overall Wall Geometry

Height	=	244	(in)	
Length	=	636	(in)	
Gravity Load (Live)	=	0	(kip/ft)	Factor = 0
Gravity Load (Dead)	=	0.435	(kip/ft)	Factor = 0.9
Wall Weight	=	54	(psf)	Factor = 0.9
Moment	=	0	(kip-ft)	Factor = 0
Shear	=	47.488	(kip)	Factor = 1

Openings:

None

Pier Design Check

```
------------
Pier No: 1
------------

    Geometry:
       left       = 0 (in)
       bottom     = 0 (in)
       width      = 636 (in)
       height     = 244 (in)
       Pier stiffness ki       = 68541 (kip/in)
       Relative Stiffness Ri = 1

    Loadings:
       Dead load                              = 81.2546 (kip)
       Factored Axial Load (Compression) = 73.1291 (kip)
       Factored Axial Load (Tension)        = 0 (kip)
       Factored Moment (in-plane)           = 965.589 (kip-ft)
       Factored Moment (out-of-plane)       = Not checked
       Factored Shear                       = 47.488 (kip)

    Pier Classification and Properties:
       Masonry        : Concrete
       Construction : Hollow units, partially grouted
       Reinforcing   : Single layer
       Mortar         : Type S

       Wall thickness                              = 7.625 (in)
       Faceshell thickness                         = 1.25 (in)
       Masonry compressive strength f'm            = 2000 (psi)
       Masonry tensile strength ft                 = 40 (psi)
       Grout spacing                               = 32 (in)
       Rebar Area, Ab                              = 0.31 (in^2)
       Rebar spacing                               = 32 (in)
       Steel yield strength fy                     = 60000 (psi)
       Rebar distribution                          = Uniformly distributed and
       Location of rebar from wall face, dc        = 3.8125 (in)
       Location of rebar from wall end, d_end = 4 (in)
       Total jamb steel area, As_jamb              = 0.31 (in^2)

    Shear reinforcement:
```

```
Joint reinforcement
-------------------
Area, Av(wire)           = 0.0326121 (in^2)
Spacing, s(wire)         = 16 (in)
yield strength fy(wire)  = 60000 (psi)

Bond beam steel
---------------
Area, Av(bond beam)            = 0.620807 (in^2)
Spacing, s(bond beam)          = 48 (in)
yield strength fy(bond beam)   = 60000 (psi)


Factored Pier Capacity / Design Checks
--------------------------------------

Axial Load Resistance, Pr                           = 725.197 (kip)
Moment Resistance (in-plane), Mr_y                  = 3810.2 (kip-ft)
Moment Resistance (out-of-plane), Mr_x              = Not checked
Total Shear Resistance                              = 202.125 (kip)

Resistance of shear steel                           = 202.125 (kip)

Axial load - Moment Interaction -->OK
Shear resistance -->OK
```

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M A S O N R Y D E S I G N S O F T W A R E 2 . 0 (C)
Fero Corporation

Masonry Shear Wall

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Shear Wall SW3
Designer : Dragan Todorovic
Date : Apr 24, 02

Design Code : ACI 530-95/ASCE 5-95/TMS 402-95
Member Mark : SW3
Run No. : 1
Remarks : 1.2D + 1.0E + 0.5S

Loading and Overall Wall Geometry

Height	=	244	(in)	
Length	=	636	(in)	
Gravity Load (Live)	=	0.634	(kip/ft)	Factor = 0.5
Gravity Load (Dead)	=	0.435	(kip/ft)	Factor = 1.2
Wall Weight	=	54	(psf)	Factor = 1.2
Moment	=	0	(kip-ft)	Factor = 0
Shear	=	47.488	(kip)	Factor = 1

Openings:

None

Pier Design Check

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Pier No: 1

Geometry:

```
left      = 0 (in)
bottom    = 0 (in)
width     = 636 (in)
height    = 244 (in)
Pier stiffness ki      = 68541 (kip/in)
Relative Stiffness Ri = 1
```

Loadings:

```
Dead load                              = 81.2546 (kip)
Factored Axial Load (Compression) = 114.306 (kip)
Factored Axial Load (Tension)      = 0 (kip)
Factored Moment (in-plane)         = 965.589 (kip-ft)
Factored Moment (out-of-plane)     = Not checked
Factored Shear                         = 47.488 (kip)
```

Pier Classification and Properties:

```
Masonry        : Concrete
Construction : Hollow units, partially grouted
Reinforcing  : Single layer
Mortar         : Type S

Wall thickness                               = 7.625 (in)
Faceshell thickness                          = 1.25 (in)
Masonry compressive strength f'm       = 2000 (psi)
Masonry tensile strength ft             = 40 (psi)
Grout spacing                                = 32 (in)
Rebar Area, Ab                               = 0.31 (in^2)
Rebar spacing                                = 32 (in)
Steel yield strength fy                  = 60000 (psi)
Rebar distribution                          = Uniformly distributed and
Location of rebar from wall face, dc    = 3.8125 (in)
Location of rebar from wall end, d_end = 4 (in)
Total jamb steel area, As_jamb          = 0.31 (in^2)
```

Shear reinforcement:

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Joint reinforcement

```
Area, Av(wire)             = 0.0326121 (in^2)
Spacing, s(wire)           = 16 (in)
yield strength fy(wire)    = 60000 (psi)
```

Bond beam steel

```
Area, Av(bond beam)             = 0.620807 (in^2)
Spacing, s(bond beam)           = 48 (in)
yield strength fy(bond beam)    = 60000 (psi)
```

Factored Pier Capacity / Design Checks

```
Axial Load Resistance, Pr                          = 725.197 (kip)
Moment Resistance (in-plane), Mr_y                 = 4607.39 (kip-ft)
Moment Resistance (out-of-plane), Mr_x             = Not checked
Total Shear Resistance                             = 202.125 (kip)

Resistance of shear steel                          = 202.125 (kip)

Axial load - Moment Interaction -->OK
Shear resistance -->OK
```

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M A S O N R Y D E S I G N S O F T W A R E 2 . 0 (C)
Fero Corporation

Masonry Slender Wall

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Slender Wall SLW3
Designer : Dragan Todorovic
Date : Apr 25,02

Design Code : Uniform Building Code
Member Mark : SLW3
Run No. : 1
Remarks : 1.4D

Overall Wall Geometry

Total height	= 288 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

Floor/Support	Height (in)
1	244

Wall section

Section	From h (in)	To h (in)
1	0	244
2	244	288

Floor Loads

Floor No:1

Height above base =	244	(in)	
Live load =	0	(kip/ft)	Factor = 0
Dead load =	0.435	(kip/ft)	Factor = 1.4
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	244	0	0
244	288	0	0

Note:
1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	244	54	1.4
244	288	54	1.4

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.00248253	5.23424e-17
Floor No:1	-0.00248253	5.23424e-17

Wall Classification and Properties

Wall Section No:1

========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	=	7.625 (in)
Faceshell thickness	=	1.25 (in)
Masonry compressive strength f'm	=	2000 (psi)
Masonry tensile strength ft	=	111.803 (psi)
Grout spacing	=	32 (in)
Rebar spacing	=	32 (in)
Steel yield strength fy	=	60000 (psi)
Bar area	=	0.31 (in^2)
Number of bar layer	=	1
Bar location (from wall face)	=	3.8125 (in)

Wall Section No:2

========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	=	7.625 (in)
Faceshell thickness	=	1.25 (in)
Masonry compressive strength f'm	=	2000 (psi)
Masonry tensile strength ft	=	111.803 (psi)
Grout spacing	=	32 (in)
Rebar spacing	=	32 (in)
Steel yield strength fy	=	60000 (psi)
Bar area	=	0.31 (in^2)
Number of bar layer	=	1
Bar location (from wall face)	=	3.8125 (in)

Wall Design Checks

==================

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Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf = 0.886227 (kip/ft)
Design moment, Mf = 0.0507907 (kip-ft/ft)
Factored resistance, Mr = 1.89702 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 62.5276 (kip/ft)
*** Section adequate ***

Wall Section 2

Factored load, Pf = 0.277227 (kip/ft)
Design moment, Mf = 4.06945e-05 (kip-ft/ft)
Factored resistance, Mr = 1.757 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 68.0266 (kip/ft)
*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	244	0.00271546

2	244	288	0.00253491

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M A S O N R Y D E S I G N S O F T W A R E 2 . 0 (C)

Fero Corporation

Masonry Slender Wall

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Slender Wall SLW3
Designer : Dragan Todorovic
Date : Apr 25.02

Design Code : Uniform Building Code
Member Mark : SLW3
Run No. : 1
Remarks : 1.2D + 0.5S

Overall Wall Geometry

Total height	= 288 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

Floor/Support	Height (in)
1	244

Wall section

Section	From h (in)	To h (in)
1	0	244
2	244	288

Floor Loads

Floor No:1

Height above base =	244	(in)	
Live load =	0.634	(kip/ft)	Factor = 0.5
Dead load =	0.435	(kip/ft)	Factor = 1.2
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	244	0	0
244	288	0	0

Note:
1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	244	54	1.2
244	288	54	1.2

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.00342246	-1.47826e-16
Floor No:1	-0.00342246	-1.47826e-16

Wall Classification and Properties

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```
Wall Section No:1
========================
Masonry           : Concrete
Construction      : Hollow units, partially grouted
Reinforcing       : Single layer
Mortar            : Type S

Wall thickness                        = 7.625 (in)
Faceshell thickness                   = 1.25 (in)
Masonry compressive strength f'm      = 2000 (psi)
Masonry tensile strength ft           = 111.803 (psi)
Grout spacing                         = 32 (in)
Rebar spacing                         = 32 (in)
Steel yield strength fy               = 60000 (psi)
Bar area                              = 0.31 (in^2)
Number of bar layer                   = 1
Bar location (from wall face)         = 3.8125 (in)


Wall Section No:2
========================
Masonry           : Concrete
Construction      : Hollow units, partially grouted
Reinforcing       : Single layer
Mortar            : Type S

Wall thickness                        = 7.625 (in)
Faceshell thickness                   = 1.25 (in)
Masonry compressive strength f'm      = 2000 (psi)
Masonry tensile strength ft           = 111.803 (psi)
Grout spacing                         = 32 (in)
Rebar spacing                         = 32 (in)
Steel yield strength fy               = 60000 (psi)
Bar area                              = 0.31 (in^2)
Number of bar layer                   = 1
Bar location (from wall face)         = 3.8125 (in)


Wall Design Checks
==================
```

Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf	= 1.07662 (kip/ft)
Design moment, Mf	= 0.0699652 (kip-ft/ft)
Factored resistance, Mr	= 1.94034 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 62.5276 (kip/ft)

*** Section adequate ***

Wall Section 2

Factored load, Pf	= 0.237623 (kip/ft)
Design moment, Mf	= 4.85749e-05 (kip-ft/ft)
Factored resistance, Mr	= 1.7479 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 68.0266 (kip/ft)

*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	244	0.00671534

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2	244	288	0.00625579

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M A S O N R Y D E S I G N S O F T W A R E 2 . 0 (C)
Fero Corporation

Masonry Slender Wall

Project	:	WESTGATE VILLAGE
Job No.	:	9919
Description	:	Slender Wall SLW3
Designer	:	Dragan Todorovic
Date	:	Apr 25,02

Design Code	:	Uniform Building Code
Member Mark	:	SLW3
Run No.	:	1
Remarks	:	1.2D + 1.6S + 0.8W

Overall Wall Geometry

Total height	= 288 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

Floor/Support	Height (in)
1	244

Wall section

Section	From h (in)	To h (in)
1	0	244
2	244	288

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Floor Loads

Floor No:1

Height above base =	244	(in)	
Live load =	0.634	(kip/ft)	Factor = 1.6
Dead load =	0.435	(kip/ft)	Factor = 1.2
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	244	-16.2	0.8
244	288	-16.2	0.8

Note:

1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	244	54	1.2
244	288	54	1.2

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.133513	-1.43406e-14
Floor No:1	0.177557	-1.43406e-14

Wall Classification and Properties

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Wall Section No:1
=========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	= 7.625 (in)
Faceshell thickness	= 1.25 (in)
Masonry compressive strength f'm	= 2000 (psi)
Masonry tensile strength ft	= 111.803 (psi)
Grout spacing	= 32 (in)
Rebar spacing	= 32 (in)
Steel yield strength fy	= 60000 (psi)
Bar area	= 0.31 (in^2)
Number of bar layer	= 1
Bar location (from wall face)	= 3.8125 (in)

Wall Section No:2
=========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	= 7.625 (in)
Faceshell thickness	= 1.25 (in)
Masonry compressive strength f'm	= 2000 (psi)
Masonry tensile strength ft	= 111.803 (psi)
Grout spacing	= 32 (in)
Rebar spacing	= 32 (in)
Steel yield strength fy	= 60000 (psi)
Bar area	= 0.31 (in^2)
Number of bar layer	= 1
Bar location (from wall face)	= 3.8125 (in)

Wall Design Checks
==================

Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf = 2.41163 (kip/ft)
Design moment, Mf = 0.707734 (kip-ft/ft)
Factored resistance, Mr = 2.24146 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 62.5276 (kip/ft)
*** Section adequate ***

Wall Section 2

Factored load, Pf = 0.237623 (kip/ft)
Design moment, Mf = 0.086635 (kip-ft/ft)
Factored resistance, Mr = 1.7479 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 68.0266 (kip/ft)
*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	244	0.101319

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2	244	288	0.0568834

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M A S O N R Y D E S I G N S O F T W A R E 2 . 0 (C)

Fero Corporation

Masonry Slender Wall

==

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Slender Wall SLW3
Designer : Dragan Todorovic
Date : Apr 25,02

Design Code : Uniform Building Code
Member Mark : SLW3
Run No. : 1
Remarks : 1.2D + 0.5S + 1.0E

==

Overall Wall Geometry

=====================

Total height	= 288 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

=================================

Floor/Support	Height (in)
1	244

Wall section

============

Section	From h (in)	To h (in)
1	0	244
2	244	288

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Floor Loads

Floor No:1

Height above base =	244	(in)	
Live load =	0.634	(kip/ft)	Factor = 0.5
Dead load =	0.435	(kip/ft)	Factor = 1.2
Load eccentricity =	1	(in)	

Lateral Loads

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	244	-24	1
244	288	-59.4	1

Note:

1. Negative (-ve) indicates loading directed away from wall

Wall Weight

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	244	54	1.2
244	288	54	1.2

Support Reactions

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.226435	-8.23295e-15
Floor No:1	0.479433	-8.23295e-15

Wall Classification and Properties

161/168

Wall Section No:1
========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	= 7.625 (in)
Faceshell thickness	= 1.25 (in)
Masonry compressive strength f'm	= 2000 (psi)
Masonry tensile strength ft	= 111.803 (psi)
Grout spacing	= 32 (in)
Rebar spacing	= 32 (in)
Steel yield strength fy	= 60000 (psi)
Bar area	= 0.31 (in^2)
Number of bar layer	= 1
Bar location (from wall face)	= 3.8125 (in)

Wall Section No:2
========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	= 7.625 (in)
Faceshell thickness	= 1.25 (in)
Masonry compressive strength f'm	= 2000 (psi)
Masonry tensile strength ft	= 111.803 (psi)
Grout spacing	= 32 (in)
Rebar spacing	= 32 (in)
Steel yield strength fy	= 60000 (psi)
Bar area	= 0.31 (in^2)
Number of bar layer	= 1
Bar location (from wall face)	= 3.8125 (in)

Wall Design Checks
==================

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Moment-Axial Load Interaction
=============================

Wall Section 1

Factored load, Pf = 1.79925 (kip/ft)
Design moment, Mf = 1.14842 (kip-ft/ft)
Factored resistance, Mr = 2.10424 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 62.5276 (kip/ft)
*** Section adequate ***

Wall Section 2

Factored load, Pf = 0.237623 (kip/ft)
Design moment, Mf = 0.397346 (kip-ft/ft)
Factored resistance, Mr = 1.7479 (kip-ft/ft)
Maximum axial load resistance, Pr(max) = 68.0266 (kip/ft)
*** Section adequate ***

Wall Shear
==========

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection
===============

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	244	0.509198

2	244	288	0.230334

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M A S O N R Y   D E S I G N   S O F T W A R E   2 . 0 (C)

Fero Corporation

Masonry Slender Wall

==

Project : WESTGATE VILLAGE
Job No. : 9919
Description: Slender Wall SLW3
Designer : Dragan Todorovic
Date : Apr 25,02

Design Code : Uniform Building Code
Member Mark : SLW3
Run No. : 1
Remarks : 0.9D + 1.0E

==

Overall Wall Geometry

=====================

Total height	= 288 (in)
Number of wall sections (including parapet if applicable)	= 2
Number of floors / lateral supports	= 1
Boundary condition (base)	= Pinned

Height of support above wall base

=================================

Floor/Support	Height (in)
1	244

Wall section

============

Section	From h (in)	To h (in)
1	0	244
2	244	288

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Floor Loads
===========

Floor No:1

Height above base	=	244	(in)	
Live load	=	0	(kip/ft)	Factor = 0
Dead load	=	0.435	(kip/ft)	Factor = 0.9
Load eccentricity	=	1	(in)	

Lateral Loads
=============

From height (in)	To height (in)	Lateral load, q (psf)	Load Factor
0	244	-24	1
244	288	-59.4	1

Note:
1. Negative (-ve) indicates loading directed away from wall

Wall Weight
===========

From height (in)	To height (in)	Wall dead weight, w (psf)	Load Factor
0	244	54	0.9
244	288	54	0.9

Support Reactions
=================

Floor/Support	Lateral Reaction (kip/ft)	Moment (kip-ft/ft)
Base	0.225026	6.54143e-15
Floor No:1	0.480842	6.54143e-15

Wall Classification and Properties
==================================

166/168

Wall Section No:1
========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	= 7.625 (in)
Faceshell thickness	= 1.25 (in)
Masonry compressive strength f'm	= 2000 (psi)
Masonry tensile strength ft	= 111.803 (psi)
Grout spacing	= 32 (in)
Rebar spacing	= 32 (in)
Steel yield strength fy	= 60000 (psi)
Bar area	= 0.31 (in^2)
Number of bar layer	= 1
Bar location (from wall face)	= 3.8125 (in)

Wall Section No:2
========================

Masonry	:	Concrete
Construction	:	Hollow units, partially grouted
Reinforcing	:	Single layer
Mortar	:	Type S

Wall thickness	= 7.625 (in)
Faceshell thickness	= 1.25 (in)
Masonry compressive strength f'm	= 2000 (psi)
Masonry tensile strength ft	= 111.803 (psi)
Grout spacing	= 32 (in)
Rebar spacing	= 32 (in)
Steel yield strength fy	= 60000 (psi)
Bar area	= 0.31 (in^2)
Number of bar layer	= 1
Bar location (from wall face)	= 3.8125 (in)

Wall Design Checks
==================

Moment-Axial Load Interaction

Wall Section 1

Factored load, Pf	= 1.11169 (kip/ft)
Design moment, Mf	= 1.10235 (kip-ft/ft)
Factored resistance, Mr	= 1.94829 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 62.5276 (kip/ft)

*** Section adequate ***

Wall Section 2

Factored load, Pf	= 0.178217 (kip/ft)
Design moment, Mf	= 0.397967 (kip-ft/ft)
Factored resistance, Mr	= 1.73426 (kip-ft/ft)
Maximum axial load resistance, Pr(max)	= 68.0266 (kip/ft)

*** Section adequate ***

Wall Shear

Wall Section 1

Shear capacity exceeds factored shear along full height of section

Wall Section 2

Shear capacity exceeds factored shear along full height of section

Wall Deflection

Sec.	From height (in)	To height (in)	Deflection, d (in)
1	0	244	0.433097

2	244	288	0.180452

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