Case No. _____

In the

Supreme Court

of the

State of Nevada

Electronically Filed
Jul 15 2020 10:54 a.m.
Elizabeth A. Brown
Clerk of Supreme Court

DEKKER/PERICH/SABATINI LTD.,
NEVADA BY DESIGN, LLC d/b/a NEVADA BY DESIGN,
MELROY ENGINEERING, INC. d/b/a MSA ENGINEERING CONSULTANTS,
JW ZUNINO & ASSOCIATES, LLC, and
NINYO & MOORE, GEOTECHNICAL CONSULTANTS,

Petitioners,

vs.

THE EIGHTH JUDICIAL DISTRICT COURT,
STATE OF NEVADA,
CLARK COUNTY, and
THE HONORABLE TREVOR ATKIN,

Respondents,

CITY OF NORTH LAS VEGAS,

Real Party in Interest.

FROM DECISIONS OF THE EIGHTH JUDICIAL DISTRICT COURT,
CLARK COUNTY, NEVADA
CASE NO. A-19-798346-C
HONORABLE TREVOR ATKIN · DEPARTMENT 8 · PHONE: (702) 671-4338

PETITIONERS'APPENDIX TO
PETITION FOR WRIT OF MANDAMUS OR,
ALTERNATIVELY, PROHIBITION

VOLUME 9

Docket 81459 Document 2020-25977

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CHRONOLOGICAL INDEX - APPENDIX OF EXHIBITS

Exhibit:	Volume:	Bates: PET.APP.	Date:	Description:
8	9	001128 – 001340	11/02/2007	Exhibit 9 - Dekker/Perich/Sabatini's Structural Calculations

ALPHABETICAL INDEX - APPENDIX OF EXHIBITS

Exhibit:	Vol.:	Bates: PET.APP.	Date:	Description:
10	**11**	**001560 – 001562**	**08/20/2019 1:34 PM**	**City of North Las Vegas' Appendix of Exhibits to Opposition to Dekker/Perich/Sabatini, Ltd.'s Motion to Dismiss**
	11	001563 – 001580	07/11/2019	Exhibit 1 – City of North Las Vegas' Complaint
	11	001581 – 001614	02/07/2007	Exhibit 1 – Professional Architectural Services Agreement
	11	001615 – 001680	08/29/2007	Exhibit 2 – Ninyo & Moore's Geotechnical Evaluation
	11	001681 – 001694	01/30/2008	Exhibit 3 – City of North Las Vegas' Letter to Richardson Construction Inc re Construction Contract
	11	001695 – 001696	07/13/2009	Exhibit 4 – Notice of Completion
	12	001697 – 001832	12/11/2017	Exhibit 5 – American Geotechnical Inc's Geotechnical Investigation
	12	001833 – 001836	1988 - Present	Exhibit 6 – American Geotechnical Inc. Resume of Edred T. Marsh, Principal Geotechnical Engineer
	12	001837 – 001838	07/03/2019	Exhibit 7 – Declaration of Edred T. Marsh, P.E.
	12	001839 – 001840	10/17/2007	Exhibit 8 – Ninyo & Moore Letter to Dekker/Perich/Sabatini re Review of 95 Percent Bid Set Construction Documents
	13	001841 – 002053	11/02/2007	Exhibit 9 - Dekker/Perich/Sabatini's Structural Calculations
	14	002054 – 002131	11/02/2007	Exhibit 9 - Dekker/Perich/Sabatini's Structural Calculations
	14	002132 – 002210	11/10/2007	Exhibit 10 - Plans / Record Drawings
8	**7**	**000847 – 000849**	**08/20/2019 1:24 PM**	**City of North Las Vegas' Appendix of Exhibits to Opposition to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultant's Motion to Dismiss or in the Alternative, Motion for Summary Judgment**
	7	000850 – 000867	07/11/2019	Exhibit 1 – City of North Las Vegas' Complaint

	7	000868 – 000901	02/07/2007	Exhibit 1 – Professional Architectural Services Agreement
	7	000902 – 000967	08/29/2007	Exhibit 2 – Ninyo & Moore's Geotechnical Evaluation
	7	000968 – 000981	01/30/2008	Exhibit 3 – City of North Las Vegas' Letter to Richardson Construction Inc re Construction Contract
	7	000982 – 000983	07/13/2009	Exhibit 4 – Notice of Completion
	8	000984 – 001119	12/11/2017	Exhibit 5 – American Geotechnical Inc's Geotechnical Investigation
	8	001120 – 001123	1988 - Present	Exhibit 6 – American Geotechnical Inc's Resume of Edred T. Marsh, Principal Geotechnical Engineer
	8	001124 – 001125	07/03/2019	Exhibit 7 – Declaration of Edred T. Marsh, P.E.
	8	001126 – 001127	10/17/2007	Exhibit 8 – Ninyo & Moore Letter to Dekker/Perich/Sabatini re Review of 95 Percent Bid Set Construction Documents
	9	001128 – 001340	11/02/2007	Exhibit 9 - Dekker/Perich/Sabatini's Structural Calculations
	10	001341 – 001418	11/02/2007	Exhibit 9 - Dekker/Perich/Sabatini's Structural Calculations
	10	001419 – 001497	11/10/2007	Exhibit 10 - Plans / Record Drawings
	10	001498 – 001513	2019	Exhibit 2 – Assembly Bill 421 – 80th Session 2019
	10	001514 – 001546	05/15/2019	Exhibit 3 - Minutes of the Senate Committee on Judiciary, 80th Legislature
1	**1**	**000001 – 000017**	**07/11/2019 4:35 PM**	**City of North Las Vegas' Complaint Against Defendants – Exempt from Arbitration Under N.A.R. 3(A): Seeks Damages in Excess of $50,000**
	1	000018 – 000051	02/07/2007	Exhibit 1 – Professional Architectural Services Agreement
	1	000052 – 000117	08/29/2007	Exhibit 2 – Ninyo & Moore's Geotechnical Evaluation
	1	000118 – 000131	01/30/2008	Exhibit 3 – City of North Las Vegas' Letter to Richardson Construction Inc re Construction Contract
	1	000132 – 000133	07/13/2009	Exhibit 4 – Notice of Completion

	2	000134 – 000269	12/11/2017	Exhibit 5 – American Geotechnical Inc's Geotechnical Investigation
	2	000270 – 000273	1988 - Present	Exhibit 6 – American Geotechnical Inc. Resume of Edred T. Marsh, Principal Geotechnical Engineer
	2	000274 – 000275	07/03/2019	Exhibit 7 – Declaration of Edred T. Marsh, P.E.
	2	000276 – 000277	10/17/2007	Exhibit 8 – Ninyo & Moore Letter to Dekker/Perich/Sabatini re Review of 95 Percent Bid Set Construction Documents
	3	000278 – 000491	11/02/2007	Exhibit 9 - Dekker/Perich/Sabatini's Structural Calculations
	4	000492 – 000568	11/02/2007	Exhibit 9 - Dekker/Perich/Sabatini's Structural Calculations
	4	000569 – 000647	11/10/2007	Exhibit 10 - Plans / Record Drawings
18	**15**	**002307 – 002312**	**09/26/2019**	**City of North Las Vegas' Limited Opposition to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Change Date of Hearing on Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on Order Shortening Time**
	15	002313 – 002318	09/26/2019	Exhibit 1 – Register of Actions Case A-19-798346-C
	15	002319 – 002320	09/20/2019	Exhibit 2 – Weil & Drage, APC's Letter to All Counsel re Hearing of Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' on Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on September 27, 2019
25	**15**	**002407 – 002421**	**11/13/2019 11:58 AM**	**City of North Las Vegas' Motion to Alter Judgment**
	15	002422 – 002430	10/17/2019	Exhibit 1 - Notice of Entry of Order Granting Nevada by Design, LLC d/b/a Nevada By Design Engineering Consultants' Motion to Dismiss or, in the alternative, Motion for Summary Judgment and All Joinders to the Same
	15	002431 – 002448	07/11/2019	Exhibit 2 – City of North Las Vegas' Complaint

	15	002449 – 002455	09/30/2019	Exhibit 3 - Order Granting Nevada by Design, LLC d/b/a Nevada By Design Engineering Consultants' Motion to Change Date
	15	002456 – 002471	2019	Exhibit 4 - Assembly Bill 421 – 80th Session 2019
	16	002472 – 002504	05/15/2019	Exhibit 5 - Minutes of the Senate Committee on Judiciary – Eightieth Session
	16	002505 – 002510	09/30/2019	Exhibit 6 - Richardson Construction, Inc. and The Guarantee Company of North America USA's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
	16	002511 – 002514	09/30/2019	Exhibit 7 - JW Zunino & Associates LLC's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
6	**6**	**000821 – 000826**	**08/15/2019 5:02 PM**	**City of North Las Vegas' Motion to Strike and Opposition to Jackson Family Partnership LLC d/b/a Stargate Plumbing's Motion to Dismiss**
	6	000827 – 000828	08/06/2019	Exhibit 1 – Affidavit/Declaration of Service to Jackson Family Partnership LLC d/b/a Stargate Plumbing
62	**20**	**003467 – 003470**	**04/02/2020 4:21 PM**	**City of North Las Vegas' Notice of Entry of Decision and Order Denying Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss**
	20	003471 – 003480	04/02/2020	Exhibit 1 - Order Denying Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss
66	**21**	**003589 – 003592**	**05/05/2020 3:48 PM**	**City of North Las Vegas' Notice of Entry of Decision and Order Denying Richardson Construction, Inc. and The Guarantee Company of North America USA's Motion to Dismiss / Motion for Summary Judgment Based on Laches and All Joinders**
	21	003593 – 003597	05/05/2020	Exhibit 1 – Court's Decision and Order Denying Richardson Construction, Inc. and The Guarantee Company of North America USA's Motion to Dismiss / Motion for Summary Judgment Based on Laches and All Joinders

46	**18**	**003064 – 003067**	**01/24/2020 3:55 PM**	**City of North Las Vegas' Notice of Entry of Decision and Order Granting Its Motion to Alter Judgment**
	18	003068 – 003073	01/23/2020	Exhibit 1 – Court's Decision and Order
9	**11**	**001547 – 001559**	**08/20/2019 1:34 PM**	**City of North Las Vegas' Opposition to Dekker/Perich/Sabatini, Ltd.'s Motion to Dismiss**
52	**19**	**003255 – 003274**	**02/17/2020 4:39 PM**	**City of North Las Vegas' Opposition to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' and Joinders Motion to Dismiss on Order Shortening Time**
60	**20**	**003409 – 003413**	**03/16/2020 4:57 PM**	**City of North Las Vegas' Opposition to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion for Clarification Regarding Court's Minute Order Denying Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss Brought Pursuant to NRS 11.258, on Order Shortening Time**
	20	003414 – 003415	03/13/2020	Exhibit 1 – Email re Proposed Order Denying MSA's Motion to Dismiss on NRS 11.258
	20	003416 – 003425	Undated	Exhibit 2 – Order Denying Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss
	20	003426 – 003428	03/16/2020	Exhibit 3 – Email re Request to Withdraw Motion for Clarification on Order Shortening Time Without Prejudice
7	**6**	**000829 – 000846**	**08/20/2019 1:24 PM**	**City of North Las Vegas' Opposition to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultant's Motion to Dismiss or, in the Alternative, Motion for Summary Judgement**
45	**18**	**003047 – 003063**	**12/19/2019 4:59 PM**	**City of North Las Vegas' Reply in Support of Its Motion to Alter Judgment**

20	**15**	**002326 – 002330**	**09/27/2019 4:18 PM**	**City of North Las Vegas' Surreply to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Change Date of Hearing on Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on Order Shortening Time**
61	**20**	**003429 – 003466**	**03/30/2020 3:09 PM**	**Court Recorder's Transcript of Hearing re All Pending Motions, March 10, 2020**
63	**20**	**003481 – 003491**	**04/10/2020 3:04 PM**	**Court Recorder's Transcript of Hearing re All Pending Motions, March 17, 2020**
23	**15**	**002339 – 002398**	**10/10/2019 1:20 PM**	**Recorder's Transcript of Hearing Re: All Pending Motions, September 30, 2019**
65	**21**	**003541 – 003588**	**04/21/2020 8:19 AM**	**Court Recorder's Transcript of Proceedings re All Pending Motions, February 20, 2020**
64	**21**	**003492 – 003540**	**04/21/2020 8:19 AM**	**Court Recorder's Transcript of Proceedings re City of North Las Vegas' Motion to Alter Judgment, January 21, 2020**
29	**16**	**002678 – 002681**	**11/26/2019 12:35 PM**	**Dekker/Perich/Sabatini, Ltd.'s Joinder to JW Zunino & Associates LLC's Opposition to City of North Las Vegas' Motion to Alter**
49	**19**	**003147 – 003154**	**02/04/2020 3:11 PM**	**Dekker/Perich/Sabatini, Ltd.'s Joinder to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss on Order Shortening Time**
3	**5**	**000718 – 000720**	**08/06/2019 2:44 PM**	**Dekker/Perich/Sabatini, Ltd.'s Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, In the Alternative, Motion for Summary Judgment**

28	**16**	**002651 – 002660**	**11/26/2019 12:28 PM**	**Dekker/Perich/Sabatini, Ltd.'s Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Opposition to Motion to Alter Judgment; Opposition by Incorporation and Request to Reset Prior Motion to Dismiss**
	16	002659 – 002664	10/15/2019	Exhibit 1 – Order Granting Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment and all Joinders to Same
	16	002665 – 002677	08/06/2019	Exhibit 2 – Dekker/Perich/Sabatini, Ltd.'s Motion to Dismiss
4	**6**	**000721 – 000735**	**08/06/2019 2:44 PM**	**Dekker/Perich/Sabatini, Ltd.'s Motion to Dismiss**
	6	000734 – 000751	07/11/2019	Exhibit A – City of North Las Vegas' Complaint
	6	000752 – 000786	02/07/2007	Exhibit B – City of North Las Vegas' Complaint Exhibit 1 – Professional Architectural Services Agreement
	6	000787 – 000789	07/11/2019	Exhibit C – Affidavit of Aleema A. Dhalla, Esq.
	6	000790 – 000793	1988 – Present	Exhibit D – American Geotechnical, Inc.'s Resume of Edred T. Marsh, Principal Geotechnical Engineer
	6	000794 – 000801	03/23/2007	Exhibit E - Excerpts from Legislative History of N.R.S. 11.258
	6	000802 – 000803	07/03/2019	Exhibit F – Declaration of Edred T. Marsh, P.E.
	6	000804 – 000817	12/11/2017	Exhibit G - American Geotechnical, Inc's Geotechnical Investigation
13	**14**	**002219 – 002232**	**08/28/2019 8:48 AM**	**Dekker/Perich/Sabatini, Ltd.'s Reply to City of North Las Vegas' Opposition to Its Motion to Dismiss**
53	**19**	**003275 – 003285**	**02/18/2020 3:00 PM**	**Dekker/Perich/Sabatini, Ltd.'s Reply to City of North Las Vegas' Opposition to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' and Joinders to Motion to Dismiss on Order Shortening Time**
	19	003286 – 003287	07/03/2019	Exhibit A – Declaration of Edred T. Marsh, P.E.

	19	003288 – 003294	07/11/2019	Exhibit B – City of North Las Vegas' Complaint
12	**14**	**002214 – 002218**	**08/26/2019 4:15 PM**	**Jackson Family Partnership LLC d/b/a Stargate Plumbing's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, In the Alternative, Motion for Summary Judgment**
36	**18**	**002894 – 002900**	**12/02/2019 2:22 PM**	**Jackson Family Partnership LLC d/b/a Stargate Plumbing's Joinder to JW Zunino & Associates LLC's Opposition to Motion to Alter Judgment with Supplemental Points and Authorities**
7	**18**	**002901 – 002907**	**12/02/2019 2:22 PM**	**Jackson Family Partnership LLC d/b/a Stargate Plumbing's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Opposition to City of North Las Vegas' Motion to Alter Judgment with Supplemental Points and Authorities**
2	**18**	**003037 – 003039**	**12/03/2019 10:01 AM**	**JW Zunino & Associates LLC's Joinder to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Opposition to Motion to Alter Judgment**
50	**19**	**003155 – 003166**	**02/07/2020 3:04 PM**	**JW Zunino & Associates LLC's Joinder to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss on Order Shortening Time**
22	**15**	**002336 – 002338**	**09/30/2019 4:35 PM**	**JW Zunino & Associates LLC's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment**
31	**17**	**002686 – 002688**	**11/27/2019 10:43 AM**	**JW Zunino & Associates LLC's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Opposition to Motion to Alter Judgment**
38	**18**	**002908 – 002910**	**12/02/2019 2:34 PM**	**JW Zunino & Associates LLC's Joinder to Richardson Construction, Inc. and The Guarantee Company of North America USA's Opposition to Motion to Alter Judgment**

26	**16**	**002515 – 002527**	**11/25/2019 5:02 PM**	**JW Zunino & Associates LLC's Opposition to City of North Las Vegas' Motion to Alter Judgment**
	16	002528 – 002530	10/09/2019	Exhibit A – Affidavit of Rita Tuttle
57	**20**	**003385 – 003391**	**02/19/2020 11:29 AM**	**JW Zunino & Associates LLC's Reply to City of North Las Vegas' Opposition to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss on Order Shortening Time**
5	**6**	**000818 – 000820**	**08/08/2019 1:32 PM**	**Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Joinder to Nevada By Design, LLC d/b/a Nevada By Design Engineering Consultants' Motion to Dismiss or, In the Alternative, Motion for Summary Judgment**
40	**18**	**003029 – 003032**	**12/02/2019 3:19 PM**	**Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Joinder to JW Zunino & Associates, LLC's Opposition to City of North Las Vegas' Motion to Alter Judgment**
41	**18**	**003033 – 003036**	**12/02/2019 3:19 PM**	**Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Joinder to Nevada By Design, LLC d/b/a Nevada By Design Engineering Consultants' Opposition to City of North Las Vegas' Motion to Alter Judgment**
39	**18**	**002911 – 002936**	**12/02/2019 3:19 PM**	**Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Opposition to Motion to Alter Judgment**
	18	002937 – 002941	10/15/2019	Exhibit 1 – Order Granting Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment and all Joinders to Same
	18	002942 – 002960	08/20/2019	Exhibit 2 – City of North Las Vegas' Opposition to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
	18	002961 – 003021	10/10/2019	Exhibit 3 – Court Recorder's Transcript of Hearing: All Pending Motions

	18	003022 – 003024	10/15/2019	Exhibit 4 – Order Granting Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Change Date of Haring on Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on Order Shortening Time
	18	003025 – 003028	08/05/2019	Exhibit 5 – Cover Sheet Filings of: Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment; Dekker/Perich/Sabatini, Ltd.'s Motion to Dismiss; and Melroy Engineering, Inc. d/b/a MSA Engineering Consultants Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
7	**18**	**003074 – 003090**	**02/04/2020 12:14 PM**	**Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss on Order Shortening Time**
	19	003091 – 003108	07/11/2019	Exhibit A – City of North Las Vegas' Complaint
	19	003110 – 003111	07/11/019	Exhibit B – Affidavit of Aleema A. Dhalla, Esq.
	19	003112 – 003115	1988 - Present	Exhibit C – American Geotechnical Inc's Resume of Edred T. Marsh, Principal Geotechnical Engineer
	19	003116 – 003123	03/23/2007	Exhibit D – Legislative History of 11.258 Senate Bill 243
	19	003124 – 003137	12/11/2017	Exhibit E – American Geotechnical Inc's Geotechnical Investigation
	19	003138 – 003139	07/03/2019	Exhibit F – Declaration of Edred T. Marsh, P.E.
59	**20**	**003399 – 003408**	**03/16/2020 8:58 AM**	**Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion for Clarification Regarding Court's Minute Order Denying Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss Brought Pursuant to NRS 11.258, on Order Shortening Time**

55	**20**	**003308 – 003318**	**02/18/2020 5:02 PM**	**Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Reply to City of North Las Vegas' Opposition to Its Motion to Dismiss**
	20	003319 – 003325	02/12/2020	Exhibit 1 – Notice of Entry of Order Granting Kittrell Garlock and Associates, Architects, AIA, Ltd.'s Motion to Dismiss; Kittrell Garlock and Associates, Architects, AIA, Ltd.'s Motion to Dismiss City of North Las Vegas' Complaint
	20	003326 – 003340	11/22/2019	Kittrell Garlock and Associates, Architects, AIA, Ltd.'s Motion to Dismiss City of Las Vegas' Complaint
	20	003341 - 003347	11/06/2019	Exhibit A – City of North Las Vegas' Complaint
	20	003348 – 003353	N/A	Exhibit B – Michael Panish Expert Witness & Consultants Construction Systems Curriculum Vitae
	20	003354 – 003361	03/23/2007	Exhibit C - Legislative History of 11.258 Senate Bill 243
	20	003362 – 003366	12/09/2019	A-19-804979-C Kelli Nash' Opposition to Defendant's Motion to Dismiss its Complaint
	20	003367 – 003373	12/26/2019	A-19-804979 Kittrell Garlock and Associates, Architects, AIA, Ltd.'s Reply to Kelly Nash's Opposition to its Motion to Dismiss Kelly Nash's Complaint
	20	003374 – 003378	10/15/2019	Exhibit 1 – Stipulation and Order to Dismiss Kittrell Garlock and Associates, AIA, Ltd.
30	**16**	**002682 – 002685**	**11/26/2019 12:43 PM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Joinder to JW Zunino & Associates LLC's Opposition to City of North Las Vegas' Motion to Alter**
48	**19**	**003140 – 003146**	**02/04/2020 3:09 PM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Joinder to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss on Order Shortening Time**

17	**15**	**002282 – 002292**	**09/18/2019 3:07 PM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Change Date of Hearing on Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on Order Shortening Time**
	15	002293 – 002294	08/06/2019	Exhibit A – Clerk of the Court's Notice of Hearing
	15	002295 – 002296	09/06/2019	Exhibit B – Court's Notice of Rescheduling Motions to Dismiss and Joinders
	15	002297 – 002202	09/09/2019	Exhibit C – Emails re Rescheduling of Hearing
	15	002203 – 002304	09/10/2019	Exhibit D – Emails re Rescheduling of Hearing
	15	002305 – 002306	N/A	Exhibit E – Las Vegas Law Offices of Snell & Wilmer
2	**5**	**000648 – 000663**	**08/05/2019 4:15 PM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment**
	5	000664 – 000681	07/11/2019	Exhibit A – City of North Las Vegas' Complaint
	5	000682 – 000684	07/13/2009	Exhibit B – City of North Las Vegas' Complaint Exhibit 4 Notice of Completion
	5	000685 – 000690	03/25/2019	Exhibit C - Nevada Legislature Website (80th Session) Concerning the "Effective Date" of the AB 421
	5	000691 – 000693	07/11/2019	Exhibit D – Aleem A. Dhalla, Esq.'s Affidavit of Merit Attached to City of North Las Vegas' Complaint
	5	000694 – 000707	12/11/2017	Exhibit E - American Geotechnical, Inc's Geotechnical Investigation
	5	000708 – 000709	07/03/2019	Exhibit F – Declaration of Edred T. Marsh, P.E.
	5	000710 – 000717	03/23/2007	Exhibit G – Excerpts from Legislative History of N.R.S. 11.258
24	**15**	**002399 – 002406**	**10/17/2019 10:08 AM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Notice of Entry of Order Granting Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment and All Joinders to Same**

27	**16**	**002531 – 002558**	**11/26/2019 11:17 PM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Opposition to Motion to Alter Judgment**
	16	002559 – 002563	10/15/2019	Exhibit 1 – Order Granting Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment and all Joinders to Same
	16	002564 – 002582	08/20/2019	Exhibit 2 – City of North Las Vegas' Opposition to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
	16	002583 – 002643	10/10/2019	Exhibit 3 – Court Recorder's Transcript of Hearing: All Pending Motions
	16	002644 – 002646	10/15/2019	Exhibit 4 – Order Granting Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Change Date of Hearing on Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on Order Shortening Time
	16	002647 – 002650	08/05/2019	Exhibit 5 - Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
			08/06/2019	Dekker/Perich/Sabatini, Ltd.'s Motion to Dismiss
			08/08/2019	Melroy Engineering, Inc. d/b/a MSA Engineering Consultants Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
19	**15**	**002321 – 002325**	**09/26/2019 5:16 PM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Reply to City of North Las Vegas' Limited Opposition to Motion to Change Date of Hearing**
54	**20**	**003295 – 003307**	**02/18/2020 3:57 PM**	**Nevada by Design, LLC d/b/a Nevada By Design Engineering Consultants' Reply to City of North Las Vegas' Opposition to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' and Joinders to Motion to Dismiss on Order Shortening Time**

14	**14**	**002233 – 002249**	**8/28/2019 9:02 AM**	**Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Rely to City of North Las Vegas' Opposition to Motion to Dismiss or, in the Alternative, Motion for Summary Judgement**
	14	002250 – 002255	07/01/019	Exhibit A – Assembly Bill No. 221 – Committee on Judiciary 80th Session (2019)
	14	002256 – 002257	2019	Exhibit B – 80th Session (2019)
	15	002258 – 002271	12/11/2017	Exhibit C – American Geotechnical Inc's Geotechnical Investigation
35	**17**	**002891 – 002893**	**12/02/2019 1:54PM**	**Ninyo & Moore, Geotechnical Consultants' Joinder to JW Zunino & Associates LLC's Opposition to City of North Las Vegas' Motion to Alter Judgment**
44	**18**	**003044 – 003046**	**12/06/2019 10:08 AM**	**Ninyo & Moore, Geotechnical Consultants' Joinder to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Opposition to Motion to Alter Judgment With Respect to Statute of Repose Arguments**
51	**19**	**003167 – 003174**	**02/07/2020 3:36 PM**	**Ninyo & Moore, Geotechnical Consultants' Joinder to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss on Order Shortening Time**
	19	003175 – 003240	08/29/2007	Exhibit A – Ninyo & Moore's Geotechnical Evaluation
	19	003241 – 003254	12/11/2017	Exhibit B – American Geotechnical Inc's Geotechnical Investigation
11	**14**	**002211 – 002213**	**08/23/2019 10:02 AM**	**Ninyo & Moore, Geotechnical Consultants' Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, In the Alternative, Motion for Summary Judgment**
15	**15**	**002272 – 002274**	**09/06/2019 12:14 PM**	**Ninyo & Moore, Geotechnical Consultants' Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, In the Alternative, Motion for Summary Judgment**

34	**17**	**002888 – 002890**	**12/02/2019 1:54 PM**	**Ninyo & Moore, Geotechnical Consultants' Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Opposition to City of North Las Vegas' Motion to Alter Judgment**
58	**20**	**003392 – 003398**	**02/19/2020 2:56 PM**	**Ninyo & Moore, Geotechnical Consultants' Reply to City of North Las Vegas Opposition to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' and Joinders to Motion to Dismiss on Order Shortening Time**
32	**17**	**002689 – 002693**	**11/27/2019 1:15 PM**	**Paffenbarger & Walden, LLC and P & W Bonds, LLC's Joinder in (1) Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Opposition to Motion to Alter Judgment; and (2) JW Zunino & Associates LLC Opposition to Motion to Alter Judgment**
43	**18**	**003040 – 003043**	**12/04/2019 8:35 AM**	**Paffenbarger & Walden, LLC and P & W Bonds, LLC's Joinder in (1) Richardson Construction, Inc. and The Guarantee Company of North America USA's Opposition to Motion to Alter Judgment; and (2) Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Opposition to Motion to Alter Judgment**
16	**15**	**002275 – 002281**	**09/13/2019 4:22 PM**	**Paffenbarger & Walden, LLC and P & W Bonds, LLC's Limited Joinder in Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment**
21	**15**	**002331 – 002335**	**09/30/2019 11:29 AM**	**Richardson Construction, Inc. and The Guarantee Company of North America USA's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment**

56	**20**	**003379 – 003384**	**02/18/2020 5:06 PM**	**Richardson Construction, Inc. and The Guarantee Company of North America USA's Limited Response to Melroy Engineering, Inc. d/b/a MSA Engineering Consultants' Motion to Dismiss on Order Shortening Times and All Joinder Thereto**
33	**17**	**002694 – 002887**	**11/27/2019 4:51 PM**	**Richardson Construction, Inc. and The Guarantee Company of North America USA's Opposition to Motion to Alter Judgment and Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Opposition to Motion to Alter Judgment**
	17	002706 – 002723	07/11/2019	Exhibit A – City of North Las Vegas' Complaint
	17	002724 – 002740	08/05/2019	Exhibit B - Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
	17	002741 – 002758	07/11/2019	Exhibit A – City of North Las Vegas' Complaint
	17	002759 – 002761	07/13/2009	Exhibit B – City of North Las Vegas' Complaint Exhibit 4 Notice of Completion
	17	002762 – 002767	03/25/2019	Exhibit C – AB421
	17	002768 – 002770	07/11/2019	Exhibit D – Affidavit of Aleema A. Dhalla, Esq.
	17	002771 – 002784	12/11/2017	Exhibit E – American Geotechnical Inc's Geotechnical Investigation
	17	002785 – 002786	07/03/2019	Exhibit F – Declaration of Edred T. Marsh, P.E.
	17	002787 – 002794	03/23/2007	Exhibit G – Senate Bill 243 - 11.258
	17	002795 – 002796	08/06/2019	Exhibit C – Clerk of the Court's Notice of Hearing
	17	002797 – 002815	08/20/2019	Exhibit D – City of North Las Vegas' Opposition to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
	17	002816 – 002822	09/04/2019	Exhibit E – Richardson Construction, Inc.'s and The Guarantee Company of North America USA's Motion to Dismiss

	17	002823 – 002824	09/06/2019	Exhibit F – Clerk of the Court's Notice of Hearing
	17	002825 – 002831	11/27/2019	Exhibit G – Register of Actions
	17	002832 – 002833	09/10/2019	Exhibit H – Emails re Rescheduling of Hearing
	17	002834 – 002846	09/18/2019	Exhibit I - Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Change Date of Hearing of Motion to Dismiss or, in the Alternative, Motion for Summary Judgment
	17	002847 – 002848	08/06/2019	Exhibit A – Clerk of the Court's Notice of Hearing
	17	002849 – 002850	09/06/2019	Exhibit B – Court's Notice of Rescheduling Motions to Dismiss and Joinders
	17	002851 – 002856	09/09/019	Exhibit C – Emails re Rescheduling of Hearing
	17	002857 – 002858	09/10/2019	Exhibit D – Emails re Rescheduling of Hearing
	17	002859 – 002860	N/A	Exhibit E – Las Vegas Law Offices of Snell & Wilmer
	17	002861 – 002862	09/20/2019	Exhibit J – Weil & Drage, APC Letter to All Counsel re Hearing of Nevada By Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on September 27, 2019
	17	002863 – 002868	09/26/2019	Exhibit K - Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Reply to City of North Las Vegas' Limited Opposition to Motion to Change Date of Hearing
	17	002869 – 002871	11/27/2019	Exhibit L – Register of Actions A-19-798346-C
	17	002872 – 002874	11/27/2019	Exhibit M – Register of Actions A-19-798346-C
	17	002875 – 002880	09/30/3019	Exhibit N – Richardson Construction, Inc. and The Guarantee Company of North America USA's Joinder to Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Dismiss or, in the Alternative, Motion for Summary Judgment

	17	002281 – 002887	10/17/2019	Exhibit O – Notice of Entry of Order Granting Nevada by Design, LLC d/b/a Nevada by Design Engineering Consultants' Motion to Change Date of Haring on Motion to Dismiss or, in the Alternative, Motion for Summary Judgment on Order Shortening Time

EXHIBIT 9

86004

OFFICE COPY

ENGINEERING CALCS HAVE BEEN REVIEWED AND ACCEPTED AS PART OF THE APPROVED PLANS

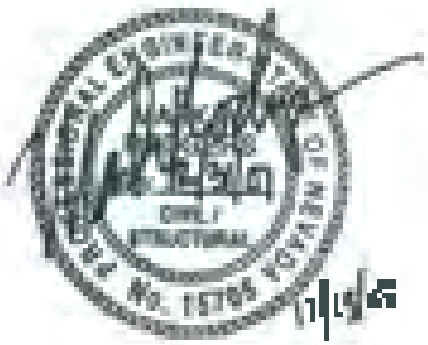
Dekker/Perich/Sabatini

civil structural mechanical electrical plumbing fire protection

STRUCTURAL CALCULATIONS

City Of North Las Vegas
Fire Station 53
North Las Vegas, NV

November 2, 2007



PET.APP.001279

[redacted]

GRAVITY DESIGN LOAD SUMMARY – ROOF LOADS

[redacted]

STRUCTURAL CALCULATIONS

City Of North Las Vegas
Fire Station 53
North Las Vegas, NV

Gravity Framing – Horizontal

Dekker/Perich/Sabatini

Name NLVFO Station #53
No. 07502-01
Subject Roof Deck Design
By AJB Date 03/26/02
Chkd Date
Sheet #

Roof Deck Design

Loads: DL = 20 PSF
LL = 20 PSF
TL = 40 PSF

Joist Spacing @ 6'-0" o.c.

USE Vulcraft 3B - 22 ga. Type Roof Deck

w/ Allow Total Load = 89 PSF >> 40 PSF : OK

* Refer to Diaphragm Calculation in the Lateral Design Section for Additional Deck Design Information.

1.5 B, BI, BA, BIA

Maximum Deck Length 45 ft
Extra Charge for Lengths Under 8 ft
Factory Mutual Approved (for 0C8A1 AM, 0D1A1 AM and 5T1AA Assy.)
ICBO Approved (No. 3415)





SECTION PROPERTIES

Deck Type	Galv. Thick.	Design Thick.	Area of Section	I	S	r	y

Type B deck provides excellent structural load carrying capacity per pound of steel utilized and its nestable design eliminates the need for dunnage

1" (or more) rigid insulation is required for Type B deck

Acoustical deck (Type BA, BIA) is particularly suitable in situations such as auditoriums, schools, and theatres where sound control is desirable. Acoustic perforations are located in the valleys where the rigid carrying properties are negligibly affected (see illustration)

Inert, non-organic glass fiber sound absorbing pads are placed in the rib openings to absorb up to 85% of the sound striking the deck

Deck are best installed and may require preparation

ACOUSTICAL INFORMATION

Deck Type	Absorption Coefficients						Noise Reduction Coefficient
	125	250	500	1000	2000	4000	

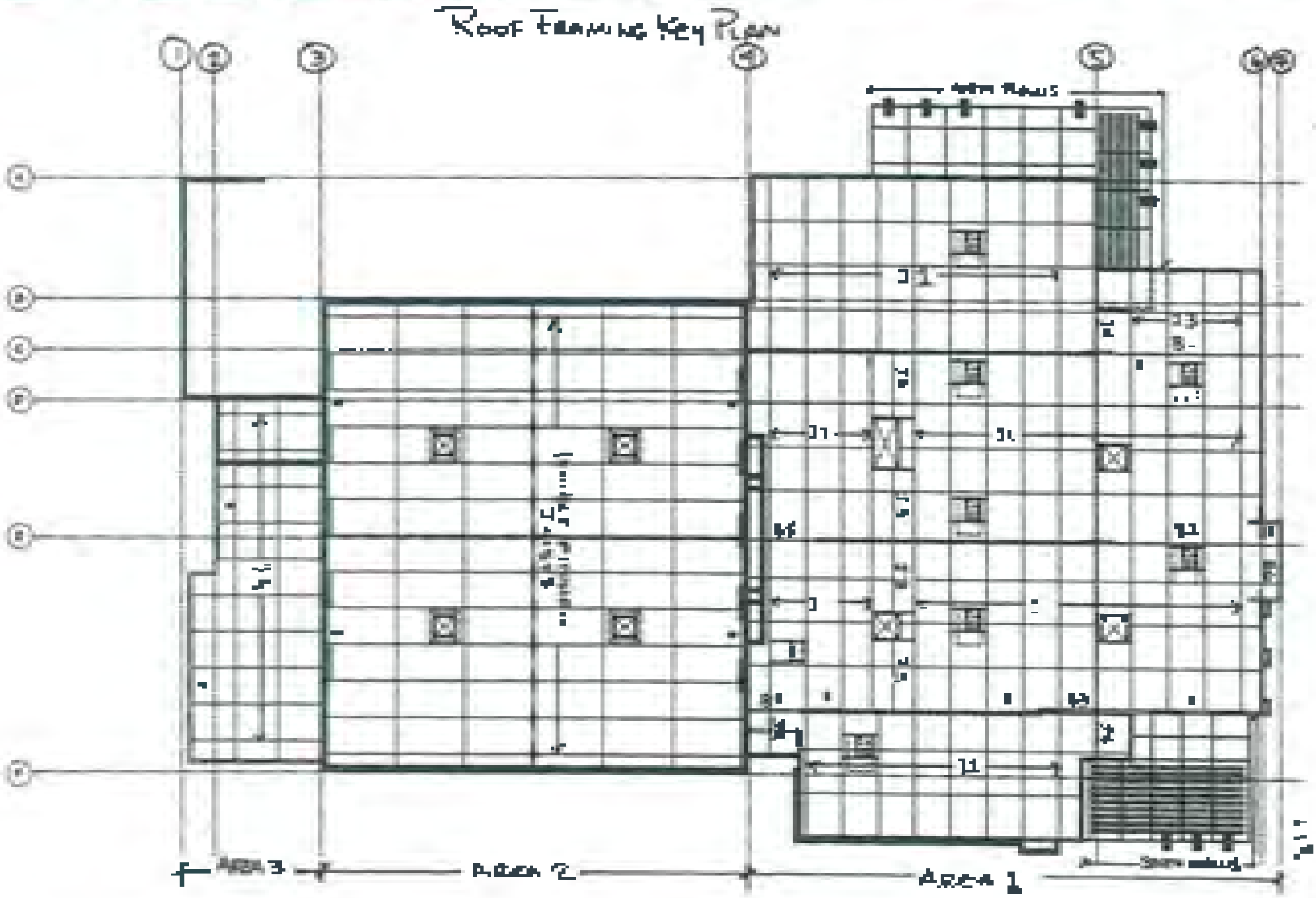
Source: Laboratory measurements conducted at Riverbank Acoustical Laboratories
Type BA deck with 1" thick rigid fiber glass insulation
Perforated area 23% of total surface area

VERTICAL LOADS FOR TYPE 1.5B

No. of Spans	Deck Type	Min. Bearing	Allowable Total Load (Including Deck Weight) in PSF for Various Spans in Feet										
			7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	15'-0"	16'-0"	17'-0"
1													
2													
3													

Notes:
1. Load tables are based on uniformly distributed loads and simple span conditions. Deflection limited to L/240 or 1" whichever is less. Consult Vulcraft for other design criteria.
2. Loads shown in the shaded areas are governed by shear and may be reduced by concentrated loads. Consult Vulcraft for more information.
3. Bearing shall be 4" min. for steel supports and 6" min. for concrete or masonry supports.

PET.APP.001233


Roof framing Key Plan
Area 3
Area 2
Area 1

Dokker/Perich/Sabatini

Project: NLCFD Station #53
Number: 07502-01
Subject: Roof Framing
By: AJB Date: 02/20/07
Checked: _____ Sheet: _____
Page: 44

1) Roof Joists - Area 1

- Joists - J1

Span, L = 31'-0"

Joist Spacing = 6'-0" o.c.

W_DL = 20 psf x 6'-0" = 120 plf
W_LL = = 120 plf

W_TL = 240 plf

USE 20K4 steel joists @ 6'-0" oc.

Allow. Total Load = 250 plf > 240 plf } OK
Allow. Live Load = 162 plf > 120 plf }

- Joists - J2

Span, L = 20'-11" ≈ 21'-0"

Joist Spacing = 6'-0" oc

W_DL = 120 plf
W_LL = 120 plf

W_TL = 240 plf

USE 14K3 steel joists @ 6'-0" oc

Allow. Total Load = 322 plf > 240 plf } OK
Allow. Live Load = 212 plf > 120 plf }

Dekker/Perich/Sabatini

[redacted]

Name: NLVFD Station #53
Job No.: 07302-01
Subject: ROOF FRAMING
By: AMB Date: 03/20/07
Check: Sheet No.:
Scale: HS

- JOISTS - J3

SPAN, L = 13'-9" ≈ 14'-0"

JOIST SPACING = 6'-0" oc.

DL = 120 PLF
LL = 120 PLF

WTL = 240 PLF

USE 10K1 STEEL JOISTS @ 6'-0" oc

ALLOW. TOTAL LOAD = 412 PLF > 240 PLF } OK
ALLOW. LIVE LOAD = 284 PLF > 120 PLF }

- JOISTS - J4

SPAN, L = 7'-5" ≈ 8'-0"

JOIST SPACING = 6'-0" oc

DL = 20 PLF
LL = 120 PLF

WTL = 240 PLF

USE 2.5K3 JOIST SUBSTITUTES @ 6'-0" oc

ALLOW. TOTAL LOAD = 408 PLF > 240 PLF } OK
ALLOW. LIVE LOAD = 193 PLF > 120 PLF }

2.5K SERIES

2.5K SERIES are steel bar joists with parallel chords and relatively light weight. They are designed for use in roofs where spans of 100' or less are adequate. These joists are commonly specified to span over hallways and short spans in stepped roofs.



NOTE: 2.5K SERIES NOT U.L. APPROVED

FABRICATION

- Depth: 2.5 in
- Maximum Length: 10 ft
- Minimum Length: 3 ft
- Contact your local Vulcraft dealer for shipping or packing information.

LOAD TABLES SPECIFICATIONS

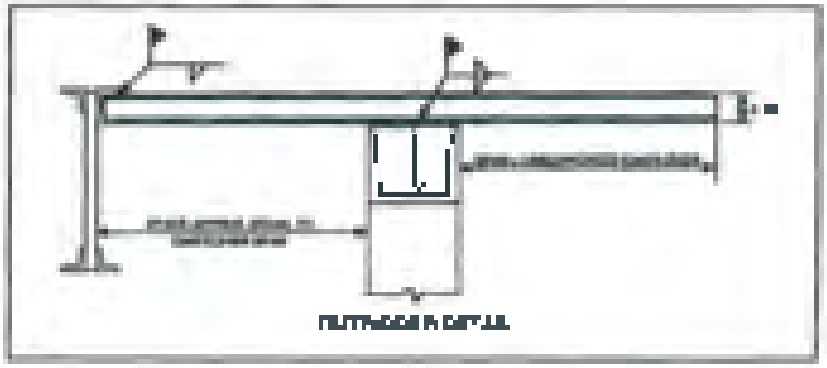
Joist substitutions are fabricated from material conforming to the SJI specifications. F_y = 50 ksi F_y = 36 ksi. Full lateral support to the compression flange is assumed as will be provided by roof deck.

2.5K SERIES SIMPLE SPAN LOAD TABLE

JOIST DESIGNATION			

ERECTION STABILITY

Caution must be exercised since joists successively exhibit some degree of instability. After erection and before laying of any construction are applied on the roof substitution the ends must be welded to the supports over SJI specs and the metal deck installed and welded to the top flange.



OUTRIGGER DETAIL

NOTE: 2.5K SERIES NOT U.L. APPROVED

LOAD TABLE FOR LOOSE OUTRIGGERS

Outrigger Type										

Dokker/Perich/Tabatini

[redacted]

Project: NLVFD Station #53
Job No.: 07502.051
Subject: Roof Framing
By: AJB Date: 08/15/07
Check: MA

Roof Joist - Area 2

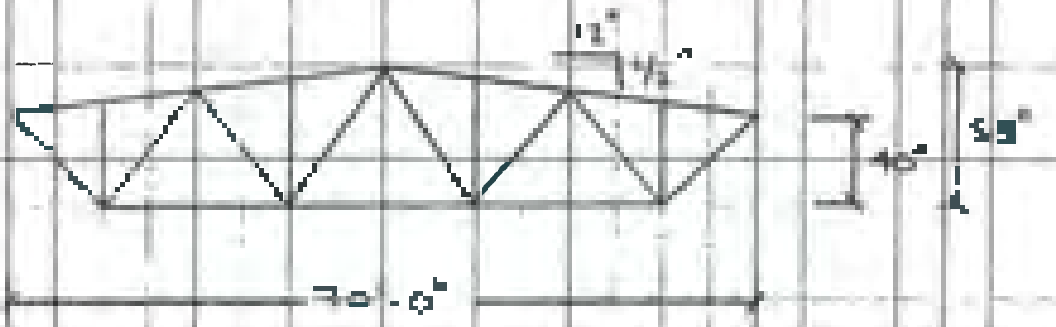
Span, L = 70'-0"

Joist Spacing = 6'-0" o.c.

W_{DL} = 20 psf × 6 ft = 120 PLF

W_{LL} = 120 PLF

$W_{T.L.}$ = 240 PLF



40-5B Double Pitched Top Chord Underslung

40" TSI T.L. Allow Load = 288 PLF > 240 PLF ∴ OK
L.L. Allow Load = 160 PLF > 120 PLF ∴ OK

NOTE: All rows of bridging shall be diagonal bridging with bolted connections at chords and intersection.

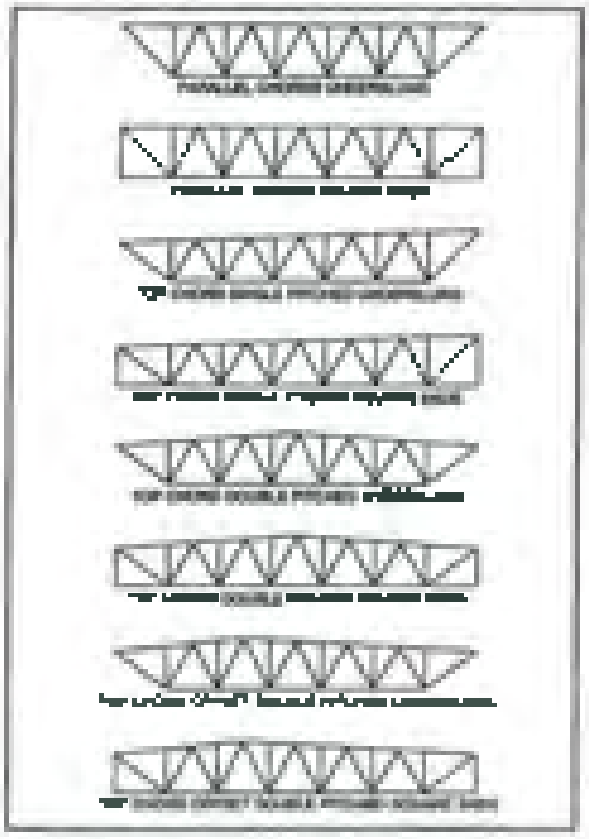
LH & DLH SERIES LONGSPAN STEEL JOISTS

STANDARD TYPES

Longspan steel joists can be furnished with either underslung or square ends, with parallel chords or with single or double pitched top chords to provide sufficient slope for roof drainage.

The Longspan joist designation is determined by the nominal depth of the center of the open triangle, for pitched chords pitched joists, where the depth should be given at the ridge. A joist of this designation should be used for the greater number of the cases design load over the design live load (TLL) given in psf.

All pitched ends will be cambered in addition to the pitch unless specified otherwise.



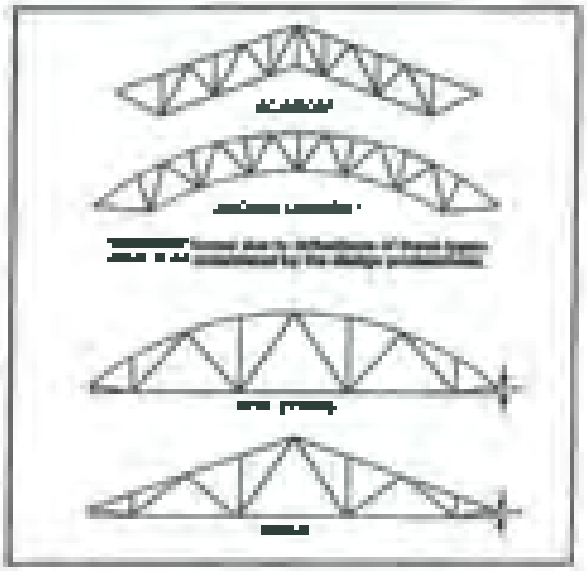
CAMBER

Non-Standard Types: The design professional shall provide on the structural drawings the amount of camber desired as a percent of the load and on a per-foot or span basis.

Standard Types: The camber listed in the table will be furnished and the joists unless the design professional specifies otherwise on the structural drawings.

NON-STANDARD TYPES

The following joists can also be supplied by Vulcraft, however, the particular building codes or local practices may require additional requirements or specifications. Refer to LONGSPAN JOISTS DEPTH OR LOADING.



**Contact Vulcraft for minimum depth of ends.

CAMBER FOR STANDARD TYPES

LH SERIES joists shall be cambered in accordance with the following table:

Top Chord Length	Approx. Camber
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

[illegible]



Hitchcock/Patrick/Sabatini

Job Name: NLVFD STATION #33
Job No.: 07502-01
Subject: ROOF FRAMING
By: AJB Date: 03/20/07
Checked: Date:
Sheet No.: H12

ROOF JOISTS - AREA 3 - TYPE J5

SPAN, L = 21'-2" ≈ 22'-0"

JOIST SPACING = 6'-0" OC.

W_{DL} = 120 PLF
W_{LL} = 120 PLF

W_{TL} = 240 PLF

USE 14K3 STEEL JOISTS @ 6'-0" OC

ALLOW. TOTAL LOAD = 293 PLF > 240 PLF ✓ OK
ALLOW. LIVE LOAD = 151 PLF > 120 PLF ✓ OK

Baker/Pereira/Schmidt

[redacted]

Job NAVFAC Station #52
Job No. 07501-01
Subject Roof Framing
By AJB Date 02/20/07
Check By Date
Sheet No. H13

- Beam - B1

Span, L = 27'-3"

Tributary Area = $\frac{13'-9" + 31'-0"}{2} \times 22.375$ SF

Loads:

W_{DL} = 20 PSF × 22.375 SF = 448 PLF

W_{LL} = 448 PLF

W_{TL} = .896 KIP

Unbraced Length = 6'-0" (Joist Spacing Max)

(See Enercalc for Design)

USE W16x26 Beam

- Beam - B2

Span, L = 27'-3"

Tributary Area = $\frac{31'-0" + 27'-9"}{2} \times 30.813$ SF

Loads:

W_{DL} = 20 PSF × 30.813 SF = 618 PLF

W_{LL} = 618 PLF

W_{TL} = 1.236 KIP

(See Enercalc for Design)

USE W18x35 Beam

Miller-Turick Solutions

Name: NLVFD STATION #53
No: 03502-01
Subject: ROOF FRAMING
By: A.L.B. Date: 03/10/21
Check: HJW

- BEAM - B3

SPAN, L = 13'-4"

TRIBUTARY AREA = $\frac{22'-11" + 25'-2"}{2} \times 24.5$ FT

LOADS:

W_{DL} = 20 PSF × 24.5 FT = 490 PLF

W_{LL} = 492 PLF

W_{TL} = 980 PLF

(SEE ENERCALC FOR DESIGN)

W12x19 BEAM

- BEAM - B4

SPAN, L = 8'-8"

TRIBUTARY AREA = $\frac{3'-0" + 28'-2"}{2} \times 15.58$ FT

LOADS:

W_{DL} = 20 PSF × 15.58 FT = 312 PLF

W_{LL} = 312 PLF

W_{TL} = 624 PLF

(SEE ENERCALC FOR DESIGN)

USE W8x10 BEAM

- BEAM - B5

SPAN, L = 11'-8"

TRIBUTARY AREA = 30.875 FT² (SAME AS B2)

LOADS:

W_DL = 618 PLF

W_LL = 618 PLF

W_TL = 1,236 PLF (SAME AS B2)

(SEE ENERCALC FOR DESIGN)

USE W12 x 14 BEAM

Becker/Perlich/Sabatini

[redacted]

Project: Santo Station #53
Job No.: 04502-01
Subject: Roof Framing
By: A-1R Date: 08/16/07
Checked: _____ Date: _____
Sheet: H16 of _____

Beam B6

Span = 9'-1"

Loads:

$W_{DL} = 20\ psf \times 6\ ft = 120\ plf$

$W_{LL} = \ "\ \ " \times 1 = 120\ plf$

$W_{TL} = 240\ plf$

(See Enercalc for Design)

USE HSS6x4x1/8 Beam

[redacted]

Steel Beam Design

Description: Roof Framing Beam - B1

General Information

[redacted]

Design Loads

[redacted]

Summary

[redacted]

Forces & Stresses Summary

[redacted]

[redacted]
[redacted]
Las Vegas, [redacted]
Tel: [redacted]
Fax: [redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Steel Beam Design

[redacted]

Description: Roof Framing Beam - B1

Section Properties: W10X33

Depth	[redacted]	Weight	[redacted]
Web Thick	[redacted]	I_x	[redacted]
S_x	[redacted]	r_x	[redacted]
Flange Thick	[redacted]	Z_x	[redacted]
Area	[redacted]	I_y	[redacted]
k	[redacted]	S_y	[redacted]
		r_y	[redacted]
Adjusted for LRFD Design		Z_y	[redacted]
J	[redacted]	C_w	[redacted]
Cw	[redacted]	rts	[redacted]
		ho	[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

H2.0

Steel Beam Design

[redacted]

Roof Framing Beam - B3

Section Properties W14X26

[redacted]			
J		Cw	
Ix		Zx	
		Sx	

[redacted]

Steel Beam Design

Description Roof Framing Beam - B3

General Information

Design Summary

Force & Stress Summary

[redacted]
[redacted]
[redacted]
[redacted]
[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Steel Beam Design

Description: Roof Framing Beam - B4

General Information

Steel Section : W8X18

Distributed Loads

Summary

Force & Stress Summary

[redacted]

Steel Beam Design

[redacted]

Roof Framing Beam - B4

Section Properties W8X10

[redacted]			

[redacted]
Las Vegas, NV 89118
Tel: 702-436-1464
Fax: 702-436-1966

Steel Beam Design

Page 1

Description Steel Framing Beam - B5

General Information

Steel Section: W12X14

Fy: [redacted]

Ultimate: [redacted]

Center Span: 11.00 ft
Left Cant.: 0.00 ft
Right Cant.: 0.00 ft
Lx - Unbraced Length: 11.00 ft

Load Duration Factor: 1.00
Framing: Primary

Distributed Loads

	LC 1	LC 2	LC 3	LC 4	LC 5	LC 6	LC 7
D	0.014						0.014
L	0.014						0.014
S							0.014
Start Location							0
End Location							11

Summary

Beam OK

Maximum Load Combination Governs

Moment - Actual: 21.000 k-ft
Moment - Allowable: 28.700 k-ft
Shear - Actual: 7.000 k
Shear - Allowable: 47.900 k
Max. Deflection: -0.203 in
Longitudinal Spacing: 1.000 ft
Length/Depth Ratio: 650.7

Force & Stress Summary

	Maximum	LC	Location	LC	Location	LC	Location
Mmax (+)	21.000 k-ft	1	5.500 ft				
Mmax (-)							
Mmax @ Left							
Mmax @ Right							
Vmax @ Left	7.000 k	1	0.000 ft				
Vmax @ Right	7.000 k	1	11.000 ft				
Deflection	-0.203 in	1	5.500 ft	1	0.000 ft	1	0.000 ft
Left Cant. Defl	0.000 in	1	0.000 ft	1	0.000 ft	1	0.000 ft
Right Cant. Defl	0.000 in	1	0.000 ft	1	0.000 ft	1	0.000 ft
Overall Defl Rgt	-0.203 in	1	5.500 ft	1	0.000 ft	1	0.000 ft
Reactions @ Left	7.000 k	1	0.000 ft	1	0.000 ft		
Reactions @ Rgt	7.000 k	1	11.000 ft	1	11.000 ft		

Footer: PET.APP.001896

[redacted]

[redacted]

[redacted]

[redacted]

Steel Beam Design

Description [redacted]

General Information

Design Criteria

Summary

Force & Stress Summary

Section Properties

Cochran Partners Incorporated
8680 Gormandy, Suite 130
Las Vegas, NV 89149
Tel. 702-436-4343
Fax. 702-436-1980

Title:
Project: ALB
Description:

Scope:

Job #:

Date:

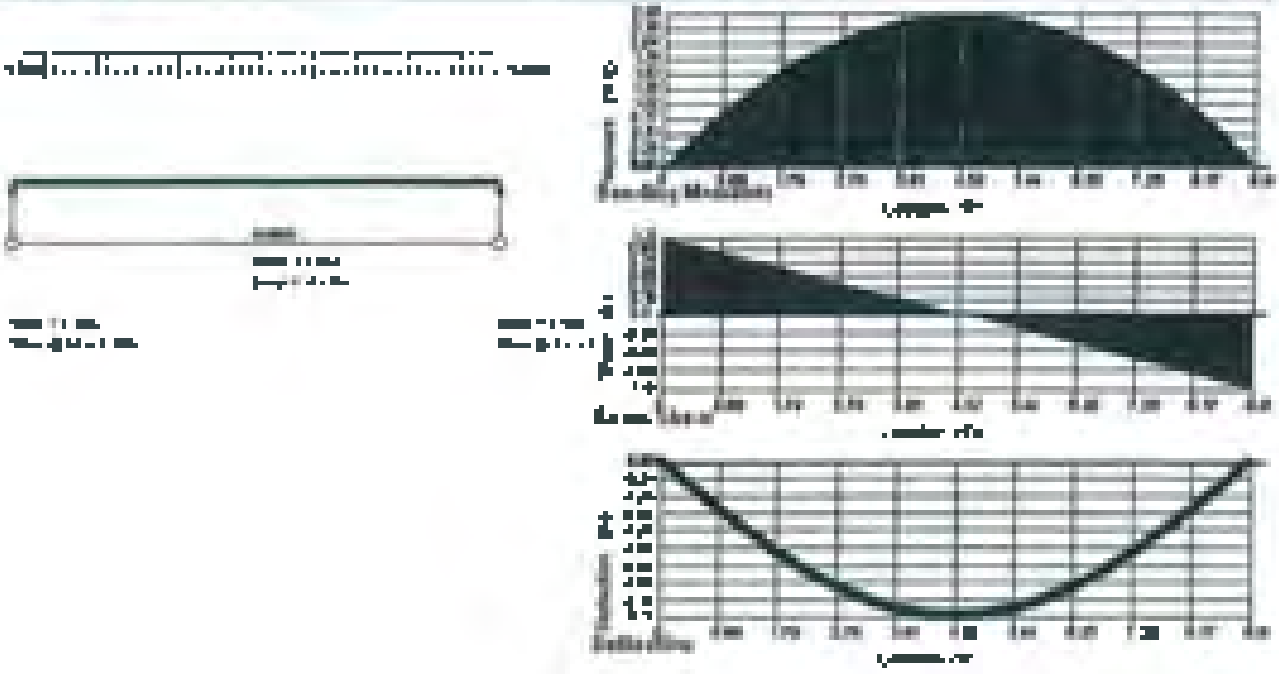
HDR

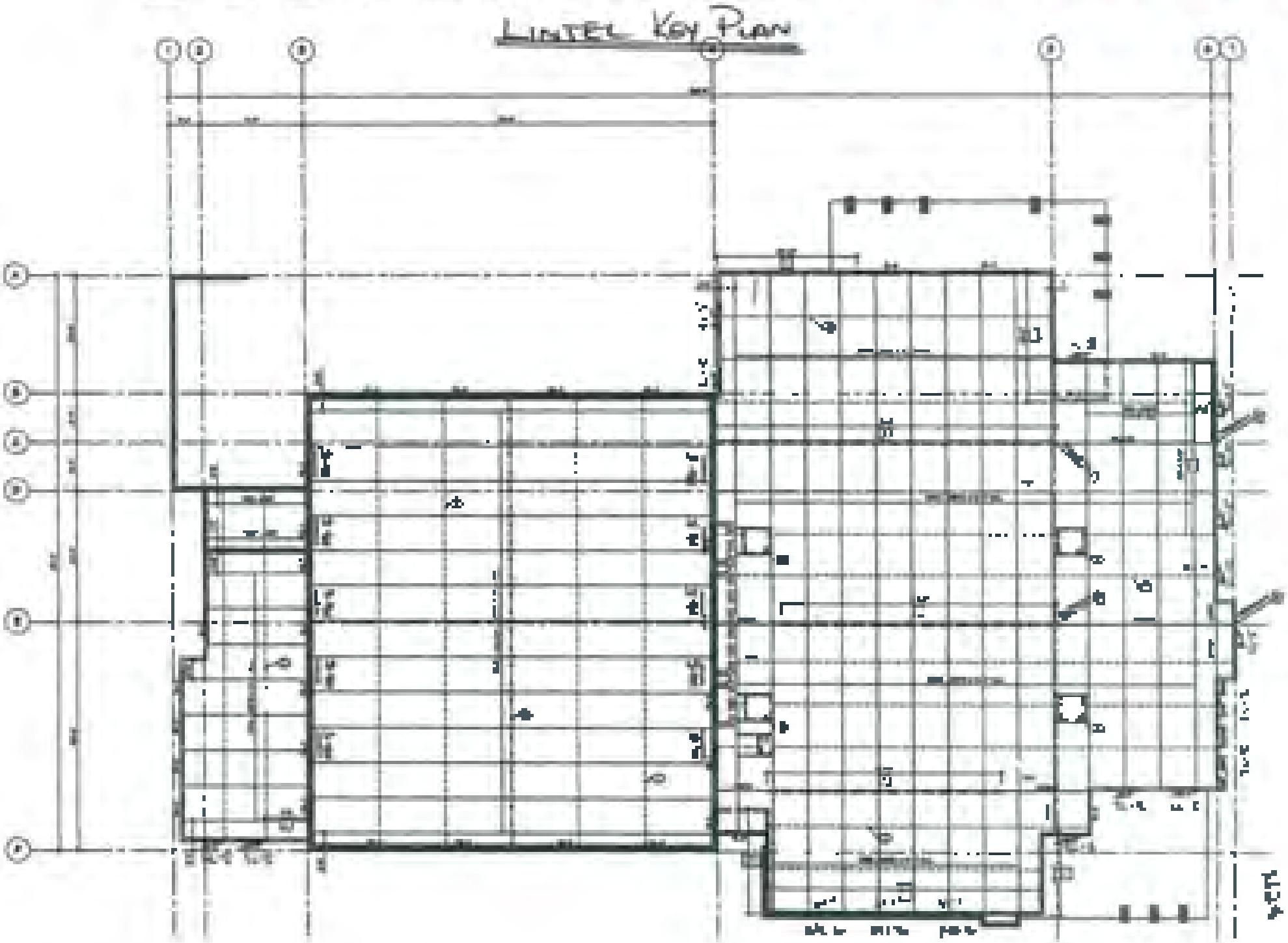
Steel Beam Design

Page 1

Description: Roof Framing Beam - B5

Beam & Diagram





PET.APP.001860

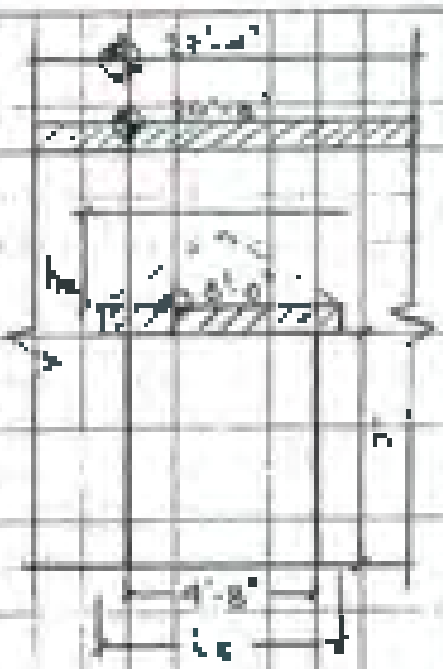
Bekker/Parish/Schlund

[redacted]

Project Name: NUFO Station #53
Project No.: 07502-01
Subject: MASONRY LINTELS
By: AJB Date: 09/21/07
Checked: Date:
Type: HSO

MASONRY LINTEL M1-1

- Span = 4'-8"
  L_c = 5'-4"
- Check Bar Arching:
  $h_a = \frac{L_c}{2} + 8" = 3'-4"$
  $2(4'-8") - 8"-8" + 12"-8" > h_a$
  ∴ Arching Occurs



- Loads:
  w_{beam} = 88 plf
  w_{CMU} = 52 plf
  w_{grout} (8 in, 8 psf) = 270 plf
  w_{rebar} = (5 in · 20 psf) = 700 plf
  w_{max} = 1,146 plf
- Flexure:
  $M = \frac{w L^2}{8} = \frac{1146\ plf\ (4.67\ ft)^2}{8} = 3,124\ ft\text{-}lb$
  $\Rightarrow M = \frac{f_m}{f_s} b_w\ jk\ d^2$
  $jk = \frac{6M}{f_m b_w d^2} = \frac{6(3,124\ lb\text{-}ft)(12"/ft)}{(1,500\ psi)(11.25")^2(6.4")} = 0.94$

Dekker/Perich/Sabatini

[redacted]

Project: NLVFD Station #53
Job No.: 07502-01
Subject: MASONRY LINTELS
By: A-DR Date: 02/01/07
Client: [redacted]
Ref: MSJC

(MSJC - 1 Comm)

$j = 1 - \frac{k}{3}$ .....

$k\left(1 - \frac{k}{3}\right) = 0.198$

$k = 0.40$

$\frac{k}{3} = \frac{0.40}{3} = 0.133$

$j = 1 - 0.133 = 0.867$

$A_s = \frac{M}{f_s j d} = \frac{3,129 \text{ lb-ft}(12 \text{ in/ft})}{(24{,}000 \text{ psi})(0.867)(6 \text{ in})}$

$A_s = 0.30$

USE 2 - #6 bars / $A_s = 0.88 \text{ in}^2 > 0.30 \text{ in}^2$ - OK

- Check Shear:

$V = w\left(\frac{L}{12} - \frac{d}{2}\right) = 1.46 \text{ psf}\left(\frac{8}{2} \text{ LF} - \frac{6 \text{ in}}{2(12)}\right)$

$V = 2,195 \text{ lbs}$

$f_v = \frac{V}{b_w d} = \frac{2,195 \text{ lbs}}{(11.25 \text{ in})(6 \text{ in})} = 31.5 \text{ psi}$

$F_v = \sqrt{f'_m} = \sqrt{1500 \text{ psi}} = 39 \text{ psi} > 31.5 \text{ psi}$

∴ No Shear Reinf. Req'd - OK

PET.APP.001862

Bakker/Portich/Sabatini

Project Name: NIYO STATION #3
Project No.: 23502-01
Subject: MASONRY LINTELS
By: AJB Date: 05/10/07
Check: Date:
Sheet: M32 of

MASONRY LINTEL M1-2

$L_c = 13' : 11"$

$L_x = 14' 0"$

- CHECK FOR ARCHING

$h_c = \frac{L_c}{2} = 8' - 7' 8"$

$20' - 8' - 15' - 4" = 5' 4" < h_c$

∴ ARCHING DOES NOT OCCUR



- LOADS:

LW BRICK : 88 PLF
LW WALL : 134 PLF
LW MASONRY (8" CMU) : 972 PLF
LW ROOF : 60 PLF
LW TOTAL = 1,296 PLF

- FLEXURAL:

$M = \frac{wL^2}{8} = \frac{(1.296\ k/ft)(13.92\ ft)^2}{8} = 28.786\ k\text{-}ft$

$jk = \frac{6M}{f'_m bd^2} = \frac{6\,[(28.786\ k\text{-}ft)(12"/ft)]}{(1500\ psi)(11.625")(14")^2}$

$jk = 0.606$

$k\left(1 - \frac{k}{3}\right) = 0.606 \Rightarrow k = 0.843$

Dekker/Perich/Sabatini

[redacted]

NLVFD Station #53
09502-01
MASONRY LINTELS
A-18 05/01/07
[redacted]
103

(ML 2 cont)

$$j = 1 - \frac{k}{3} = 1 - \frac{0.843}{3} = 0.719$$

$$A_s = \frac{M}{f_s\, j\, d} = \frac{28.786\ k\text{-}m\ (12''/ft)}{(24{,}000\ psi)(0.719)(14\text{-}in)}$$

$$A_s = 1.43\ in^2$$

⇒ TRY A DEEPER BEAM

24" deep Beam
$d = 21$ in

- assume:

$W_{beam\ self} = 88$ plf
$W_{slab} = 264$ plf
$W_{wall\ (8'\ high\ @\ 130\ pcf)} = 918$ plf
$W_{roof\ (psf \times 30')} = 120$ plf
$W_{TOTAL} = 1{,}390$ plf

- Moment

$$M = \frac{wL^2}{8} = \frac{(1.390\ kip/ft)(15.33')^2}{8} = 40.873\ k\text{-}ft$$

$$k = \frac{12M}{f_m\, b\, d^2} = \frac{12\,(40.873\ k\text{-}ft)(12''/ft)}{(500\ psi)(11.25\ in)(21\ in)^2}$$

$$k = 0.263$$

$$k\left(1 - \frac{k}{3}\right) = 0.263 \Rightarrow k = 0.291$$

PET.APP.001864

Bekker-Parish/Sabatini

[redacted]

Name: NAVY Storage B53
Job No.: 29502.01
Subject: MASONRY LINTELS
By: AJB Date: 05/01/07
Chkd: ____ Date: ____
Ref: NSW ____

(M-2 ex.)

$j = 1 - \frac{k}{3} = 1 - \frac{0.231}{3} = 0.903$

$A_s\,(req'd) = \frac{M}{F_s \cdot j \cdot d} = \frac{30{,}873\ \text{lb-ft}\ (12\ \text{in/ft})}{(24{,}000\ \text{psi})(0.903)(22\ \text{in})}$

$A_s\,(req'd) = 0.78\ \text{in}^2$

USE 2 - #6 @ 8" c/c $A_s = 0.88\ \text{in}^2 > 0.78\ \text{in}^2$

- Check Shear:

$V = w\left(\frac{\ell}{2} - \frac{d}{2}\right) = 1.390\ \text{k/ft}\left(\frac{13.42\ \text{ft}}{2} - \frac{2.2\ \text{ft}}{2\ (\text{ft/in})}\right)$

$V = 7{,}990\ \text{lbs}$

$f_v = \frac{V}{b \cdot d} = \frac{7{,}990\ \text{lbs}}{(6.625\ \text{in})(22\ \text{in})} = 31.2\ \text{psi}$

$F_v = \sqrt{f'_c} = \sqrt{1500} = 39\ \text{psi} > 31.2\ \text{psi} \therefore \text{OK}$

∴ No Shear Reinf. Req'd

Ketter/Reich/Schaldach

[redacted]

Job NUVO Station #53
Job No. 03502-01
Subject MASONRY LINTELS
By AJB Date 05/01/07
Check
Sheet KDS

MASONRY LINTEL M2-1

L = 16'-0"

L_c = 16'-8"

- Check True Arching:

$$M_c = \frac{L_c}{2} + 8" = 9'-0"$$

$$29'-8" - 14'-0" < 6'-8" < 9'-0"$$

∴ Arching Does Not Occur

- Loads:

(D) Superstructure = 88 PLF
(D) Lintel = 528 PLF
(D) Wall (8"x12'-8") = 1,026 PLF
(L) Snow (40 PSF x 35') = 1,400 PLF

w_{total} = 3,042 PLF

Moment:

$$M = \frac{w L^2}{8} = \frac{(3{,}042\ plf)(16'-0")^2}{8} = 97{,}344\ lb\text{-}ft$$

$$\rho = \frac{f_c}{f_y}\left[1 - \sqrt{1 - \frac{2M}{f_c b d^2}}\right] = \frac{4}{60}\left[1 - \sqrt{1 - \frac{2(97{,}344\ lb\text{-}ft)(12"/ft)}{(4{,}000\ psi)(11.625\ in)(48"\ in)^2}}\right]$$

ρ = 0.190

PET.APP.001866

(K = 3 cells)

$j = 1 - \frac{k}{3}$

$k\left(1 - \frac{k}{3}\right) = 0.190$

$\therefore k = 0.204$

$j = 1 - \frac{k}{3} = 1 - \frac{0.204}{3} = 0.932$

$A_s = \frac{M_u}{\phi f_y j d} = \frac{47,344 \text{ lb-ft } (12"/ft)}{(24,000 \text{ psi})(0.932)(46")}$

$A_s = 1.14 \text{ in}^2$

USE 4 - #5's w/ $A_s = 1.23 \text{ in}^2 > 1.14 \text{ in}^2$ ∴ OK

- CHECK SHEAR:

$V = w\left(\frac{l}{2} - \frac{d}{2}\right) = 3,092 \text{ plf}\left(\frac{20'}{2} - \frac{46"}{2(12"/ft)}\right)$

$V = 8,506 \text{ lbs}$

$f_v = \frac{V}{b \cdot d} = \frac{18,506 \text{ lbs}}{(11.625")(46")} = 34.6 \text{ psi}$

$F_v = 39 \text{ psi} > f_v$ ∴ OK

∴ NO SHEAR REINF. REQ'D

Dekker/Perich/Sabatini

[redacted]

Project: DAVEO Station #53
Job No.: 03502-01
Subject: MASONRY LINTELS
By: AJB Date: 05/01/07
Checked: Date:
Sheet No.: H37

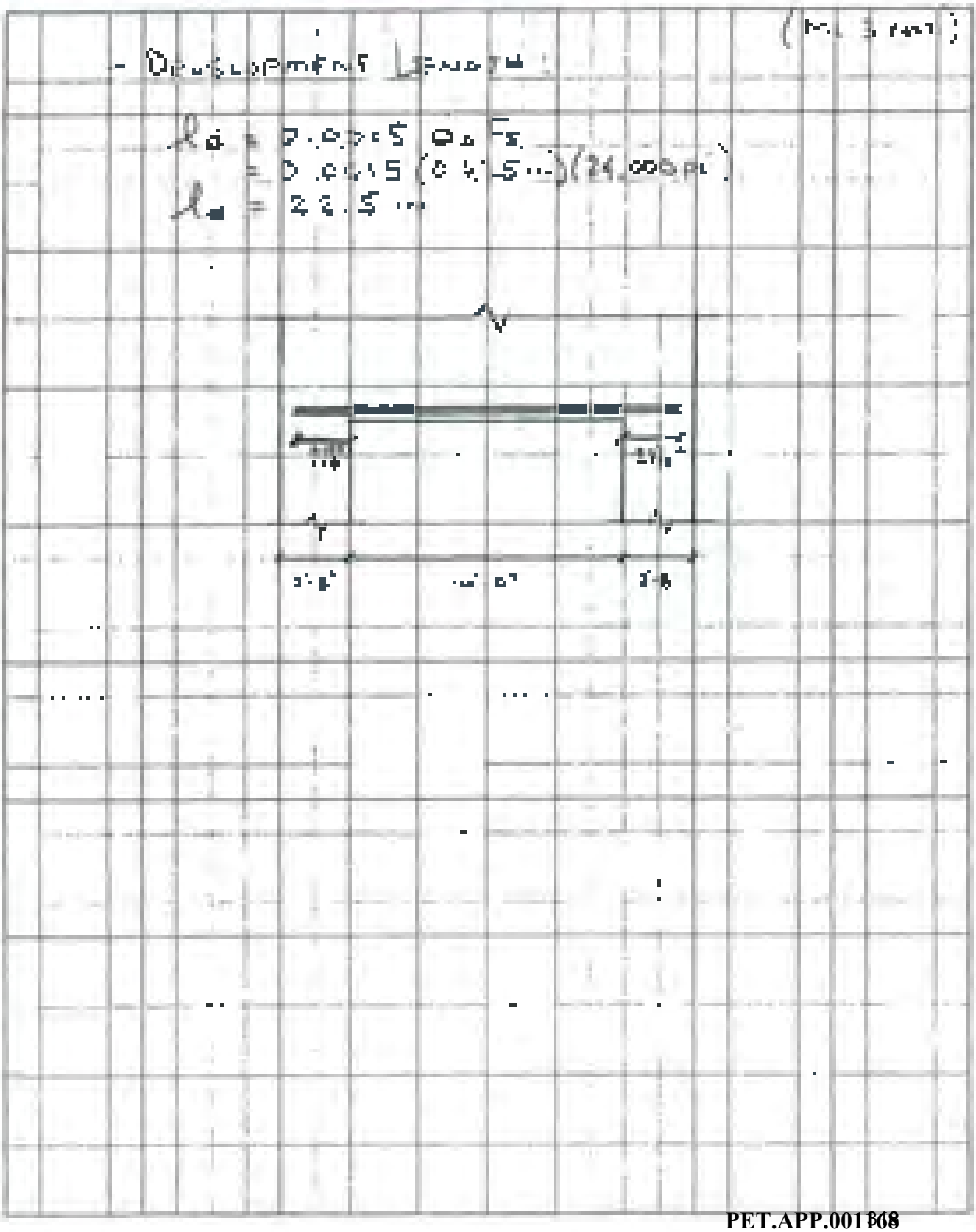
(M.I. 3 mm)

- Development Length:

$l_d = 0.075\, d_b F_s$

$= 0.075 (0.5\text{"})(24{,}000\text{ psi})$

$l_d = 22.5$ in



Bakker/Perrich/Bakalinsky

Project: NLVFD Station #5
Job No.: 07502-01
Subject: MASONRY LINTELS
By: AMB Date: 05/01/07
Checked:
Sheet: HSE

MASONRY LINTEL ML-1

L = 6'-0"

L.L. = 6' 8"

- Check for Arching:

$h_{req} = \frac{L_c}{2} + 8" = 4'-0"$

$12"\phi - 8" - 8" = 12" > 4"$

∴ Arching Occurs



Loads:

W_{Beam} = 85 PLF

W_{Lintel} = 89 PLF

W_{Wall} = (8)(12)(4) = 384 PLF

W_{Live} = (150)(10) = 1,400 PLF

W_{TOTAL} = 1,900 PLF

- Flexural:

$M = \frac{wl^2}{8} = \frac{1{,}900\ plf\ (6'-0")^2}{8} = 8{,}550\ lb\text{-}ft$

$\rho = \frac{6M}{f'm\ b\ d^2} = \frac{6\,(8{,}550\ lb\text{-}ft)(12\ in/ft)}{(1{,}500\ psi)(11.625\ in)(14\ in)^2}$

$\rho = 0.150$

Dokker/Perich/Sabatini

[redacted]

Name: NAVFAC STATION #53
Proj. No.: 09501-01
Subj: MASONRY LINTELS
By: AMB Date: 5/5/09
Checked: Date:
Sheet: 4 of

(MSJC Code):

$K\left(1 - \frac{K}{3}\right) = 0.180 \Rightarrow K = 0.192$

$j = 1 - \frac{K}{3} = 0.936$

$A_s = \frac{M}{f_s\, j\, d} = \frac{8,550 \text{ lb-ft} \times (12 \text{ in/ft})}{(24,000 \text{ psi}) \times 0.936 \times (14 \text{ in})}$

$A_s = 0.326 \text{ in}^2$

USE 2 - #6's w/ $A_s = 0.88 \text{ in}^2 > 0.326 \text{ in}^2$ ∴ OK

- Check Shear:

$V = w\left(\frac{L}{2} - \frac{d}{2}\right) = 1,000 \text{ plf}\left(\frac{18 \text{ ft}}{2} - \frac{14 \text{ in}}{2(12 \text{ in/ft})}\right)$

$V = 8,592 \text{ lbs}$

$f_v = \frac{V}{b\, d} = \frac{8592 \text{ lbs}}{(12 \text{ in})(14 \text{ in})} = 28 \text{ psi}$

$f_{v,m} = \sqrt{1500} = 39 \text{ psi} > 28 \text{ psi}$ ∴ OK

No Shear Reinf Reqd

Pfeiffer Partners Structural

[redacted]

Name: NUVO Section #53
No.: 07502 - 01
Subject: MASONRY LINTELS
By: A.L.G. Date: 08/17/07
Client:
Ref: Hand

Masonry Lintel ML-01

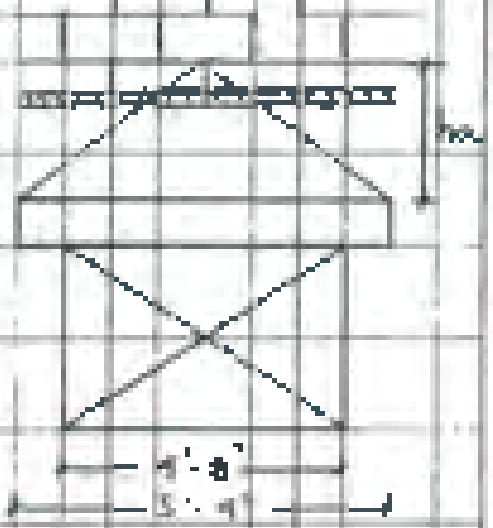
Span = 9'-8"

$L_{eff} = 5'-4" + 8" = 5'-4"$

$h_a = \frac{5'-4"}{2} + 8" = 3'-4"$

$\Rightarrow (20'-8") - (19'-4") = 1'-4"$

* Arching does NOT occur.



Loads:

$L_{wall} = 720$ #/ft

$L_{floor} = 91$ psf x 8 ft = 728 #/ft

Beam Reaction = 88 #/ft

$L_{DL} = 20$ psf x 3 ft = 60.0 #/ft

$L_{LL} = 20$ psf x 3 ft = 60.0 #/ft

$L_{Total} = 1,376$ #/ft

Flexural:

$M = \frac{wL^2}{8} = \frac{(1.376\ k/ft)(5.33\ ft)^2}{8} = 0.719\ k\text{-ft}$ (8.952 in-k)

$\rho_{req} = \frac{6M}{f'_m b d^2} = \frac{6(8.952\ k\text{-in})}{(1500\ psi)(7.625\ in)(11.5\ in)^2} = 0.142$

Dobkin Reich/Sabatini

[redacted]

Name: NLVFO Station #53
Project No.: 07502-01
Subject: MASONRY LINTELS
By: MB Date: 6/27/07
Checked: Date:
Scale: NTS Sheet #
(M = 5 ksi)

$K\left(1 - \frac{K}{3}\right) = 0.142$

$K = 0.149$

$j = 1 - \frac{0.149}{3} = 0.95$

$A_s\ (req'd) = \frac{M}{f_s \cdot j \cdot d} = \frac{80{,}352\ lb\text{-}in}{(24{,}000\ psi)(0.95)(14\ in)}$

$= 0.253\ in^2$

USE 2 - #6's $\rightarrow A_s = 0.88\ in^2 > 0.253\ in^2$ ∴ OK

- CHECK SHEAR

$V = w\left(\frac{L}{2} - \frac{d}{2}\right) = 2376\ plf\left(\frac{11.67'}{2} - \frac{1.167'}{2}\right)$

$= 9{,}162\ lbs$

$f_v = \frac{V}{b \cdot d} = \frac{9{,}162\ lbs}{(1.625\ in)(19\ in)} = 25.57\ psi$

$F_v = \sqrt{1500} = 38.7\ psi > f_v$ ∴ OK

∴ NO SHEAR REINF. REQ'D

Dekker/Perich/Sabatini

Project: NAVFAC STATION #53
Job No.: 07507.01
Subject: MASONRY LINTELS
By: AJB Date: 08/18/07
Checked: Date:
Sheet: NMS

[redacted]

MASONRY LINTEL ML-6

8" CMU WALL

SPAN = 4'-8"

L_{eff} = 5'-4"

P_D = 2'-0" + 8" x 5'-4"

(ONLY WALL LOADS APPLIED, NO ROOF AT THIS LOCATION)

--DEAD--

LINTEL = 58 PLF

WALL = 61 PSF x 5.33' = 325 PLF

W_{TOTAL} = 261 PLF

--FLEXURAL--

$$M = \frac{wL^2}{8} = \frac{(261\ plf)(4.67\ ft)^2}{8} = 713\ lb\text{-}ft\ (8538\ lb\text{-}in)$$

$$J_K = \frac{6M}{f'm\ b\ n\ d^2} = \frac{6(8538\ lb\text{-}in)}{(1500\ psi)(3.625\ in)(6\ in)^2} = 0.124$$

$$k\left(1 - \frac{k}{3}\right) = 0.124$$

$$k = 0.1296$$

$$J = 1 - \frac{0.1296}{3} = 0.957$$

Dekker/Perich/Sabatini

Name: NUMO STATION #53
Job No.: 07502-01
Subject: MASONRY LINTELS
By: AB Date: 01/18/07
Checked: Date:
Sheet: MB of
(Wall 5)

A_s (REQ'D) = $\frac{M}{\phi f_y jd} = \frac{8,538 \text{ lb-in}}{(24,000 \text{ psi})(0.95)(6 \text{ in})}$

$= 0.062 \text{ in}^2$

USE 2 - #5's w/ $A_s = 0.41 \text{ in}^2 > 0.062 \text{ in}^2$ ∴ OK

* 24 INSPECTION NO. 5 FOR PRE-INSPECTION PROD

Dekker/Perich/Sabatini

Name: NLVFD STATION #53
Project Number: 07502-01
Subject: STEEL HEADERS
By: AJB Date: 05/03/07
Checked: Date:
Sheet: HHF of

STEEL HEADER S-1

L = 3'-4" LONG BEARING HEADER

$w_{DL} = (20\ psf \times 20'-8"/2) + 20\ plf = 227\ plf$

$w_{LL} = 20\ psf \times 20'-8"/2 = 207\ plf$

$w_{TOTAL} = 414\ plf$

USE 2-600S162-43 BOXED HEADER

$\phi M_n = 1257\ ft\text{-}lbs > 414\ plf$ ∴ OK

(SEE ATTACHED USMP HEADER LOAD TABLE)

HEADER SUPPORT

- JACK STUDS:

$R_{DL} = \frac{wL}{2} = \frac{227\ plf \times (3.33')}{2}$

$R_{DL} = 378\ lbs$

$R_{LL} = 345\ lbs$

H = 10'-0"

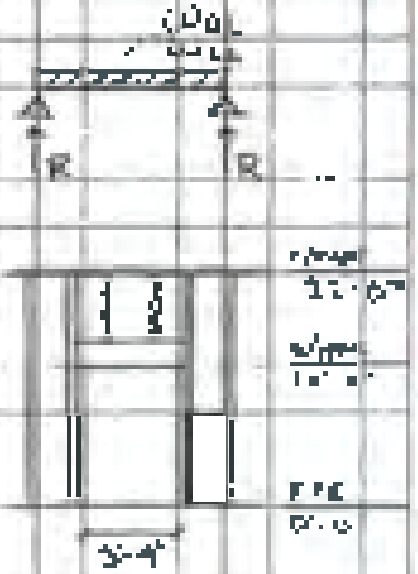
USE 4-600S162-43 STUDS

$I_{eff} = 2.316\ in^4$

$S_{eff} = 0.963\ in^3$

$r = 2.276\ in$

$A_g = 0.487\ in^2$



[redacted]

Project: HLRTO Storage #53
Project Number: 07502-01
Subject: STEEL HEADERS
By: AJB Date: 05/03/07
Checked: _____ Date: _____
Scale: NTS

SI - 1. (cont.)

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29{,}000\ ksi)}{\left(\frac{(1.0)(2.0\ in - 0.12\ in)}{0.276\ in}\right)^2} = 189\ ksi$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{33\ ksi}{189\ ksi}} = 0.564$$

$$F_{cr} = (0.658^{\lambda_c^2}) F_y = (0.658^{(0.564)^2}) 33\ ksi$$

$$= 28.86\ ksi$$

$$P_n = A_c F_{cr} = 0.344\ in^2 (28.86\ ksi) = 9{,}928\ lbs$$

$$P_a = \frac{P_n}{\Omega_c} = \frac{9{,}928\ lbs}{1.8} = 5{,}516\ lbs$$

$P = 723\ lbs < P_a \therefore$ OK

- FULL HEIGHT STUDS:

$$P_{studs} = \left[\left(\frac{3.33\ klf}{2}\right) + \left(\frac{1.33\ klf}{2}\right)\right](14.0\ ft) = 34.5\ kips$$

$$M = \frac{wL^2}{8} = \frac{(34.5\ kips)(22\ ft)^2}{8} = 2{,}087\ lb\text{-}ft$$

$$= 25\ k\text{-}in$$

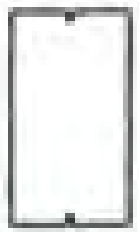
USE 2 - 600S162-43 FULL HEIGHT STUDS

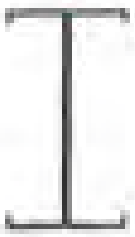
$$\phi_b M_n = (2)\ 16.68\ k\text{-}in = 33.36\ k\text{-}in \therefore$$ OK

Header Load Tables

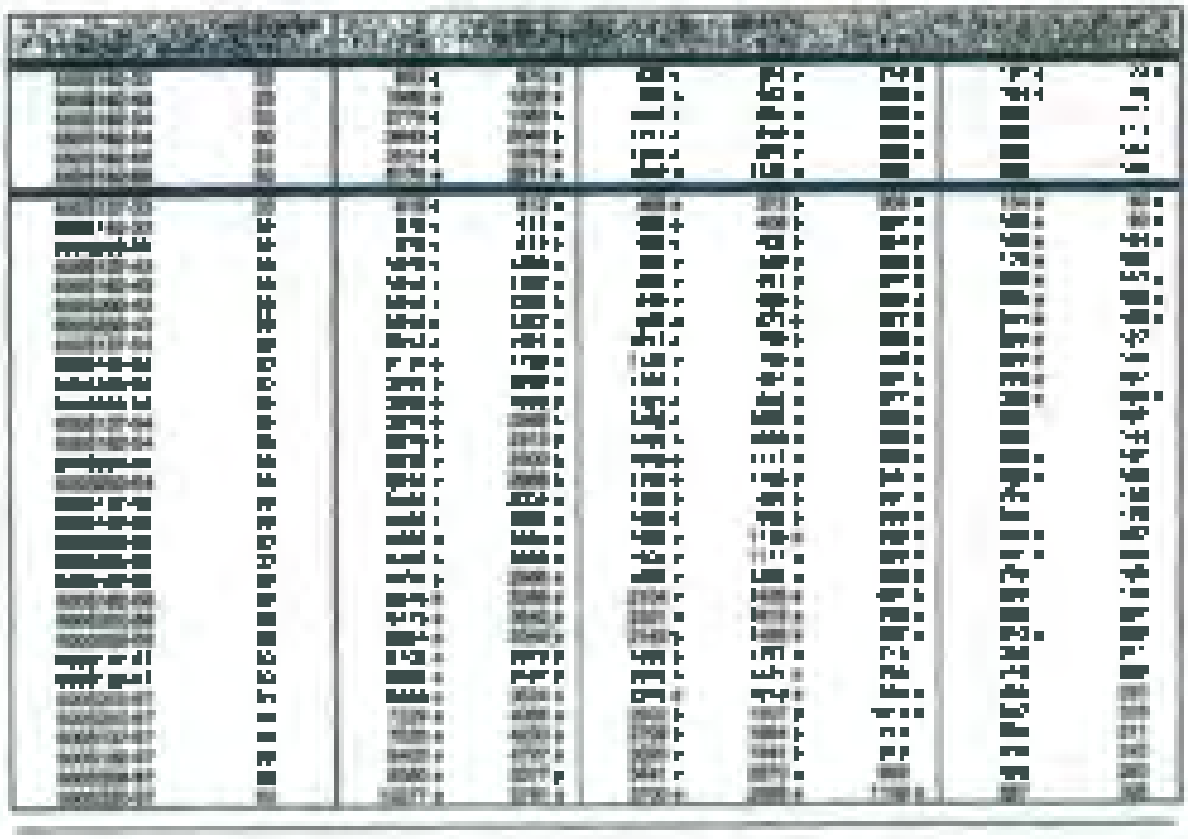
Header Load Table Notes

1. [illegible]
2. [illegible]
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4. [illegible]
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6. [illegible]
7. [illegible]
8. [illegible]



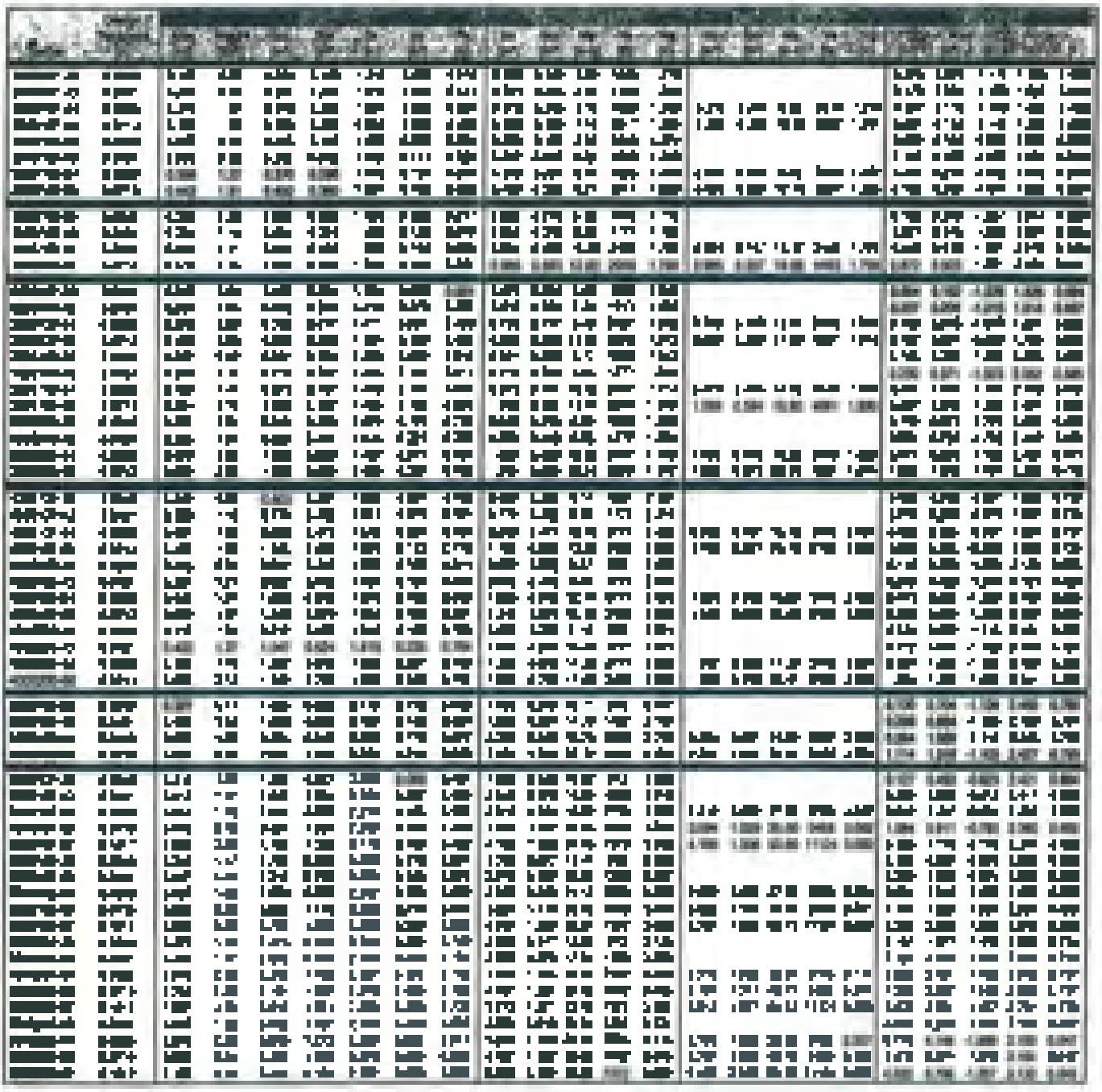


Header Allowable Uniform Loads (lb/ft)



[illegible]

Structural (S) Stud Section Properties



Dokker/Perich/Sabatini

[redacted]

Project: SAFD Station #53
Job No.: 07502-01
Subject: Steel Headers
By: A.L.B. Date: 6/09/11
Checked: Page:
Scale: NTS

Steel Header S1-2

L = 7'-6" (Load Bearing)

$W_{DL} = \left(\frac{27'-10'}{2} \times 20\ PSF\right) + 20\ PLF = 278\ PLF$

$W_{LL} = \left(\frac{27'-0'}{2} \times 20\ PSF\right) = 270\ PLF$

$= 278\ PLF$

$W_{TOTAL} = 548\ PLF$

(See Enercalc Sheet T-1 Design)

USE (1) 5-3.5" x 11 7/8" Header

- Jack Studs

$R_{DL} = 1150\ lbs$
$R_{LL} = 1040\ lbs.$
} From Enercalcs

$H = 10'-0"$

USE (2) 1-600S162-43 Stud

$P = 1,150 + 1040 = 2,190\ lbs$

From S1-1 Calculations

$P_a = 5,516\ lbs > P = 2,190$ OK

PET.APP.001829

FULL HEIGHT STUDS:

$P_{studs} = \left(\frac{1.5kf}{2} + \frac{133ft}{2}\right)(10.8)\,psf = 65.4\ plf$

$M = \frac{w\ell^2}{8} = \frac{(65.4\ plf)(12ft)^2}{8} = 3,956\ lb\text{-}ft$

$= 47.5\ k\text{-}in$

USE (3) 600S162-43 FULL HEIGHT STUDS

$M_n = (3)\,16.68\ k\text{-}in. = 50.04\ k\text{-}in > 47.5$

∴ OK



Steel Beam Design

General Information

Distributed Loads

Summary

Factored Applied Summary

Section Properties

Dekker/Perich/Sabatini

Name: NUFO STATION #53
Job No.: 07502.01
Subject: STEEL HEADERS
By: AJB Date: 02/09/07
Checked: Date:
Sheet: 1 of 1

STEEL HEADER SL-3

L = 11'-2" (LONG BEARING)

$W_{DL} = \left(\frac{28'-9"}{2} \times 20 \text{ PSF}\right) + 20 \text{ PLF} = 302 \text{ PLF}$

$W_{LL} = \left(\frac{28'-9"}{2} \times 20 \text{ PSF}\right) = 288 \text{ PLF}$

$W_{TOTAL} = 576 \text{ PLF}$

(SEE ENERCALC FOR DESIGN)

USE HSS6x4x5/16 HEADER

∴ JACK STUDS:

$R_{DL} = 1,800 \text{ lbs}$
$R_{LL} = 1,620 \text{ lbs}$ } FROM ENERCALC

$P = 3,420 \text{ lbs}$

$H = 10'-6"$

FROM SL-1 CALCULATIONS (1) 6003.62-.43

$P_a = 5,516 \text{ lbs} > 3,420 \text{ lbs}$ ∴ OK

∴ USE 2 - 6003.62-.43 STUD

(2 FOR BEARING)

- FULL HEIGHT STUDS: (S-3 UNIT)

$W_{HEAD} = \frac{11'}{2} \times 14.81 = 81.7$ PLF

$W_{STUDS} = \frac{1.32'}{2} \times 14.81 = 9.8$ PLF

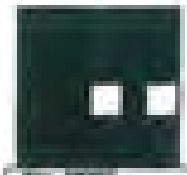
$M_L = 3.94$ K-FT $\times 12'' = 44.86$ K-IN

USE TRIBUTARY BEAM ANALYSIS

USE (3) 600S162-43 FULL HEIGHT STUDS

$M_L = (3)\ 16.68$ K-IN $= 50$ K-IN > 44.86

$\therefore$ OK



H53

Steel Beam Design

General Information

Distributed Loads

Summary

Force & Stress Summary

Section Properties

[redacted]

Job #
1454

Single Span Beam Analysis

Page 1

Description Roof Header Steel - SL-3

General Information

[redacted]		[redacted]	
[redacted]		[redacted]	
[redacted]		[redacted]	

Uniform Loads

On Center Span...		On Left Cantilever...		On Right Cantilever...	
[redacted]		[redacted]		[redacted]	

Trapezoidal Loads

Magnitude @ Left	[redacted]	[redacted]	[redacted]	[redacted]
Magnitude @ Right	[redacted]	[redacted]	[redacted]	[redacted]
Dist. To Left Side	[redacted]	[redacted]	[redacted]	[redacted]
Dist. To Right Side	[redacted]	[redacted]	[redacted]	[redacted]

Query Values

Center Location	[redacted]	Left Cant.	[redacted]	Right Cant.	[redacted]
[redacted]	[redacted]		[redacted]		[redacted]
[redacted]	[redacted]		[redacted]		[redacted]
[redacted]	[redacted]		[redacted]		[redacted]

Summary

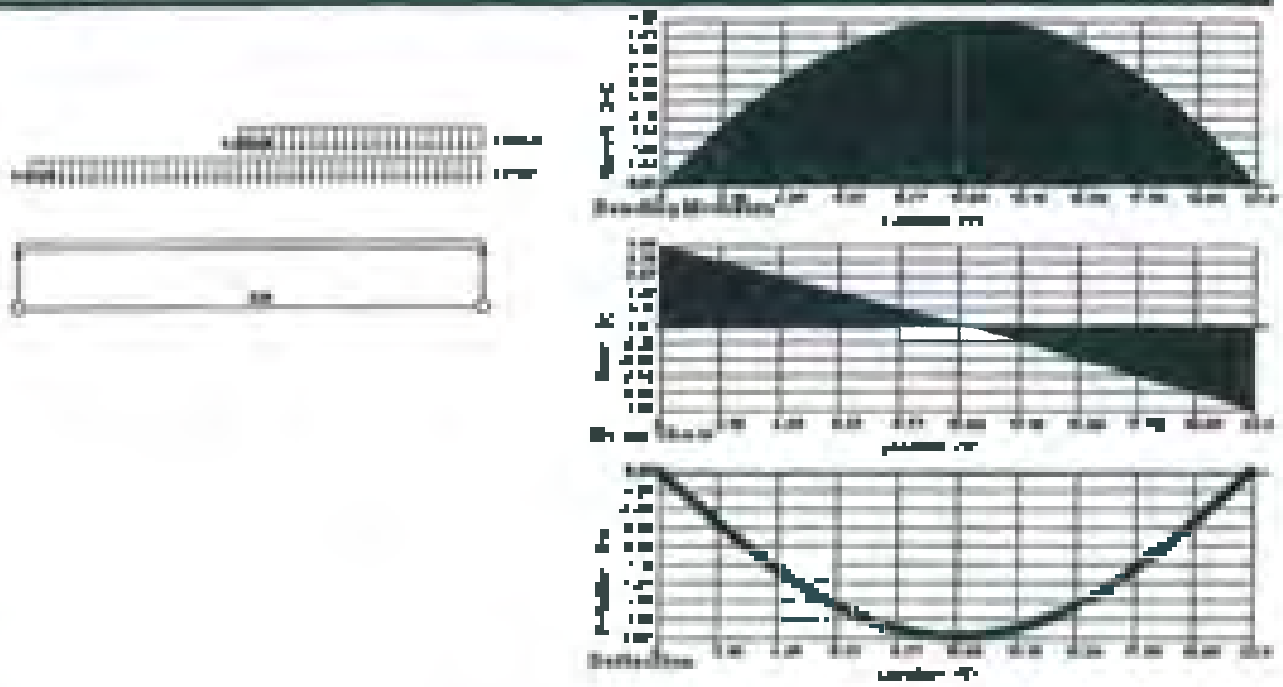
[redacted]				Reactions...		Reactions...	
[redacted]	[redacted]	at	[redacted]	@ Left	[redacted]	@ Left	[redacted]
[redacted]	[redacted]	at	[redacted]	@ Right	[redacted]	@ Right	[redacted]
@ Left Cant.	[redacted]			Moment	[redacted]		
@ Right Cant.	[redacted]			Deflections...			
[redacted]	[redacted]			@ Center	[redacted]	at	[redacted]
				@ Left Cant.	[redacted]	at	[redacted]
				@ Right Cant.	[redacted]	at	[redacted]

[redacted]

Single Span Beam Analysis

Description [redacted]

Sketch & Diagram



STEEL HEADER SL-4

L = 16'-0" (LOAD BEARING)

$W_{DL} = \left(\frac{28'-1"}{2} \times 20 psf\right) + 20 PLF = 308 PLF$

$W_{LL} = \left(\frac{28'-9"}{2} \times 20 psf\right) = 288 PLF$

$W_{TOTAL} = 596 PLF$

(SEE ENERCALC SHEET FOR D

USE HSS9x3x1/4 HEADER

- JACK STUDS:

$R_{DL} = 2,610$ lbs
$R_{LL} = 2,300$ lbs } FROM ENERCALC

$P_u = 4,710$ lbs

H = 10'-0"

FROM SL-1 CALCULATIONS: SOLID 2-ply

$P_a = 5,516$ lbs > 4,960 lbs

USE (2) 600S162-97 JACK STUDS

(2 FOR BEARING)

Keller+Gerich+Tokalnoi

[redacted]

NYPD Station #53
07502-01
Type: Hearings
Date: 9/30/09
Sheet
HSY

- Full Height Studs: (SL-4 stud)

Wind = 16 FT × 14 lb/ft = 118 #

10 studs × 1.33 FT × 14 lb/ft = 9.8 #

(See Energy Beam Analysis)

M = 5.09 k-ft × 12 in/ft = 60.84 k-in

S = 1(0.967 in³) + 3.068 in³
↳ Full Height Studs

φM_n = φF_y(0.95)(3.068 in³)(33 ksi)
= 69.8 k-in > 60.84 k-in OK

AE 9-60 05 60 43 Full Height Studs

[redacted]

Steel Beam Design

Description: Headers - SL-4

Section Information

Steel Section : HSS8X4X1/4

Beam Span	10.00 ft	Material	Fy	46 ksi
Left Cant.	2.00 ft	Design Method	Load Duration Factor	1.00
Right Cant.	2.00 ft			
Unbraced Length	5.00 ft			

Distributed Loads

	D1	D2	D3	D4	D5	D6	D7
DL	0.200						
LL	0.240						
Start Location							
End Location							

Summary

Beam OK

	Actual	Allowable		
Maximum	10.400 k-ft	23.811 k-ft	Total Deflection	0.394 in
Max. Bending Stress	20.000 ksi	34.500 ksi	Live Load Defl.	0.227 in
Max. Shear	6.000 k	11.500 k	Live Load Defl. Crit.	240.0
Shear				
Max. Shear Stress	1.731 ksi	19.550 ksi		
Max. Pv	0.000 k			

Force & Stress Summary

	Maximum	Dist.	LL (D1 Center)	LL (D1 Center)	LL (D1 Cantilever)	LL (D1 Cantilever)	
Max. +M	10.400 k-ft	10.00 ft	10.00				1.00
Max. -M							1.00
Max. Bending Left Cantilever							1.00
Max. Bending Right Cantilever							1.00
Max. +V	6.000 k	2.00 ft	6.000				1.00
Max. -V	6.000 k	2.00 ft	6.000				1.00
Center Defl.	-0.394 in	5.00 ft	-0.227	-0.167	0.000	0.000	1.00
Left Cant. Defl.	0.000 in	0.000 ft	0.000	0.000	0.000	0.000	1.00
Right Cant. Defl.	0.000 in	0.000 ft	0.000	0.000	0.000	0.000	1.00
Camber Defl.	0.000 in	0.000 ft	0.000	0.000	0.000	0.000	1.00
Reactions @ Left	6.000 k	0.000 ft	6.000	4.800	0.000	0.000	1.00
Reactions @ Right	6.000 k	2.00 ft	6.000	4.800	0.000	0.000	1.00

As used per Eq. E7-1, RTU = 0.6

Section Properties

HSS8X4X1/4

Depth	8.000 in	Area	7.420 in²
Wall Thick	0.233 in	Ix	64.200 in⁴
Width	3.953 in	Sx	16.050 in³
Flange Thick	0.233 in	rx	2.940 in
Area	7.420 in²	Iy	21.300 in⁴
Ix	64.200 in⁴	Sy	10.800 in³
		ry	1.690 in
Values for LRFD Design:			
J	84.200 in⁴	Zx	20.000 in³
Cw	11.200 in⁶	Zy	13.200 in³

[redacted]
[redacted]
Las Vegas, NV [redacted]
Fax: 702-[redacted]

Title: [redacted]
Engineer: [redacted]
Description:

Job #: [redacted]
Date: [redacted]

#57

Single Span Beam Analysis

Description: Full Height Stud - SC-4

General Information

Center Span	[redacted]	Material Properties	[redacted]
Left Cantilever	[redacted]	Beam Material	[redacted]
Right Cantilever	[redacted]	Support Conditions	Pin-Pin

Uniform Loads

On Center Span...		On Left Cantilever...		On Right Cantilever...	
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

Trapezoidal Loads

Magnitude @ Left	[redacted]	[redacted]	[redacted]	[redacted]
Magnitude @ Right	[redacted]	[redacted]	[redacted]	[redacted]
Dist. To Left Edge	[redacted]	[redacted]	[redacted]	[redacted]
Load Length	[redacted]	[redacted]	[redacted]	[redacted]

Query Values

Center Location	[redacted]	Left Cant.	[redacted]	Right Cant.	[redacted]
Moment	[redacted]		[redacted]		[redacted]
Shear	[redacted]		[redacted]		[redacted]
Deflection	[redacted]		[redacted]		[redacted]

Summary

Maximum...				Moments		Reactions	
Moment @ Center	[redacted]	at	[redacted]	@ Left	[redacted]	@ Left	[redacted]
Shear @ Center	[redacted]	at	[redacted]	@ Right	[redacted]	@ Right	[redacted]
@ Left End	[redacted]			Maximum	[redacted]		
@ Right End	[redacted]			Deflections			
Maximum =	[redacted]			@ Center	[redacted]	at	[redacted]
				@ Left End	[redacted]	at	[redacted]
				@ Right End	[redacted]	at	[redacted]

Dekker/Perich/Sabatini

Project Name: NLVPD STATION #53
Project Number: 09502-01
Subject: STEEL LINTELS
By: AJB Date: 05/09/07
Checked: Date:
Sheet: H60

STEEL HEADER SL-5

L = 7'-6" (NON BEARING)

W_{ROOF} (3m x 20 PSF) + 10 PLF = 80 PLF
W_{WALL} (3m x 20 PSF) = 60 PLF

W_{TOTAL} = 140 PLF

(SEE SSMA HEADER LOAD TABLE)

USE (2) 600S162-43 BACK TO BACK HEADER

W_{ALLOW} = 343 PLF > 140 PLF ∴ OK

- JACK STUDS:

$R_{SL5} = \frac{(80\,PLF)(7.5\,ft)}{2} = 300\,\#$

$R_{LW} = \frac{(60\,PLF)(7.5\,ft)}{2} = 225\,\#$

525 lbs

H_s = 10'-0"

From SL-1 JACK STUD CALCULATIONS FOR (1) 600S162-43:

P_a = 5,516 lbs > 525 lbs ∴ OK

USE 1-600S162-43 JACK STUD

Bekker/Perich/Sabatini

Name: NUFO STATION #53
Job No: 01502.01
Subject: STEEL LINTELS
By: AJB Date: 05/04/17
Note:
Sheet:

FULL HEIGHT STUDS: (SL 5 PORT.)

SAME AS SL:2

USE (2) C600S162-43 FULL HEIGHT STUDS

Second Floor Framing Key Plan

Bekker/Perich/Sabatini

Name: ALVEO STATION #53
Job No.: 07502.01
Subject: TRELLIS FRAMING
By: GLB Date: 06/01/07
Sheet: _____
Chkd: BLS

SOUTH TRELLIS

- BEAM B1:

LOADS:

w_{DL} = 5 psf × 5.5 ft = 28 plf

w_{LL} = 20 psf × 5.5 ft = 111 plf

L = 7'-6"

(SEE ENERCALC FOR DESIGN)

USE HSS5×3×¼

- BEAM B2

w_{DL} = 5 psf × 5.5 ft = 28 plf

w_{LL} = 20 psf × 5.5 ft = 111 plf

L = 13'-8½"

(SEE ENERCALC FOR DESIGN)

USE HSS5×3×¼

Dekker/Perich/Sabatini

Project Name: NAVFD STATION #53
Project Number: 01202-01
Subject: CANOPY FRAMING
By: AJB Date: 06/07/07
Checked: Date:
Sheet: Wet of

- Beam B3

P_{DL} = 140 lbs + 250 lbs = 390 lbs

P_{LL} = 200 lbs + 760 lbs = 1170 lbs

L = 19'

L_n = 6' - 1 1/2"

(See Envelope for Design)

USE HSS 10 x 3 x 3/16

- Beam B4

W_{DL} = 5 psf x 3.1 ft = 15.5 plf

W_{LL} = 20 psf x 3.1 ft = 62 plf

P_{DL} = 640 lbs } from B3 reactions

P_{LL} = 1,510 lbs }

L = 2' - 3"

L_n = 1' - 9"

(See Envelope for Design)

USE HSS 10 x 3 x 1/4

[redacted]

Steel Beam Design

Description: South Trellis - B7

Design Information

Material Properties

Section Properties

Summary

Forces & Stress Summary

[redacted]

Steel Beam Design

Description: South Trellis - AC

General Information

Steel Section : [redacted]

Distributed Loads

Summary

Force & Stress Summary

Section Properties [redacted]

Steel Beam Design

Page 1

South Trestle - B3

General Information

Steel Section : W18X50

Center Span	18.00 ft	Fy	50.0 ksi
Left Cant.	0.00 ft	Load Duration Factor	1.00
Right Cant.	0.00 ft	Elastic Modulus	29,000 ksi
Lu : Unbraced Length	6.00 ft		

Point Loads

	B1	B2	B3	B4	B5	B6	B7	
Dead Load	3.000	3.000	0.000					k
Live Load	1.170	1.170	1.170					k
Snow Load								k
Location	3.000	9.000	15.000					ft

Summary

Beam OK

Load Combination Governs

	Actual	Design		
Moment	72.900 k-ft	207.900 k-ft	Max. Deflection	0.000 in
fb : Bending Stress	18.000 ksi	45.000 ksi	Allowable Dead	1.080 in
M / Ma	0.350 : 1		Allowable LL Defl.	0.540 in
Shear	7.110 k	46.000 k		
fv : Shear Stress	0.810 ksi	19.800 ksi		
fv / Fv	0.041 : 1			

Forces & Stress Summary

	Maximum	Dx	DL	LL	LL Unbalanced	Load Combination
Axial, P	11.700 k	0.000	11.700			1.4D
Major M						1.4D
Major M at Left						1.2D+1.6L+0.5S
Major M at Right						1.2D+1.6L+0.5S
Shear @ Left	7.110 k	0.000	3.420			1.2D+1.6L+0.5S
Shear @ Right	7.110 k	0.000	3.420			1.2D+1.6L+0.5S
Center Defl	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in 1.2D+1.6L+0.5S
Left Cant Defl	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in 1.2D+1.6L+0.5S
Right Cant Defl	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in 1.2D+1.6L+0.5S
Clear Span Defl	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in 1.2D+1.6L+0.5S
Reaction @ Left	7.110 k	0.000	3.420	3.690		1.2D+1.6L+0.5S
Reaction @ Right	7.110 k	0.000	3.420	3.690		1.2D+1.6L+0.5S

Fy used per Fy ID=2, ETJ = 0%

Section Properties W18X50 (ASTM A992)

Depth	18.000 in	Area	14.700 in²
Web Thick	0.355 in	Ix	800.000 in⁴
Width	7.500 in	Sx	88.900 in³
Flange Thick	0.570 in	Rx	7.380 in
Area	14.700 in²	Iy	40.100 in⁴
Wt	50.000 lb/ft	Sy	10.700 in³
Weight for Unbraced Length		Ry	1.650 in
J	1.900 in⁴	Zx	101.000 in³
Cw	4.940 in⁶	Zy	16.300 in³

[redacted]

Steel Beam Design

Description South Trellis

General Information

Load Combination : [redacted]

Center Span
Left Cant.
Right Cant
Le : Unbraced Length

Distributed Loads

Point Loads

Summary

Force & Stress Summary

[redacted]

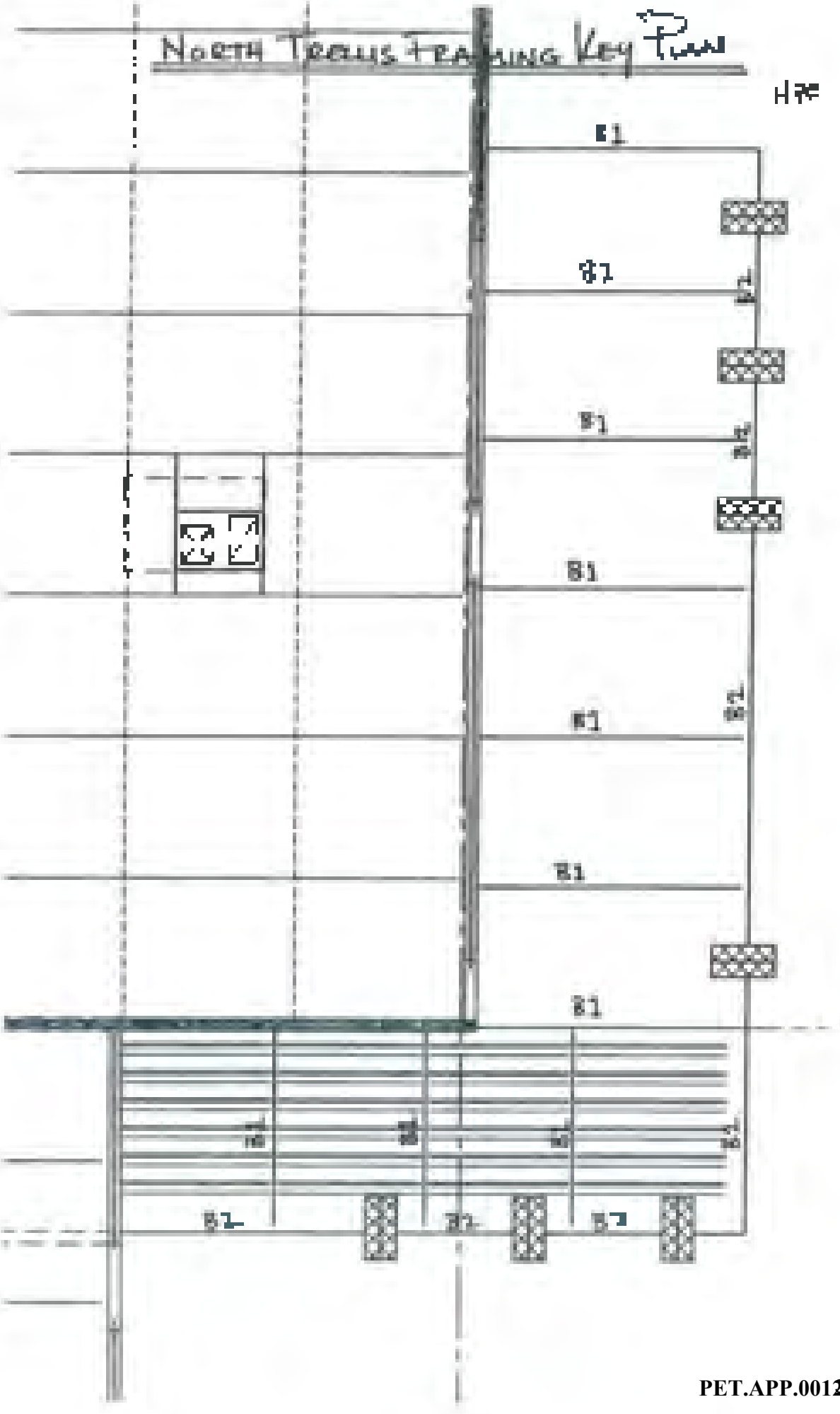
Steel Beam Design

Page 1

South Trellis - B4

Section Properties

Depth		Weight	
Values for Load Design			


North Trellis Framing Key Plan
B1
B1
B1
B1
B1
B1
B1
B2
B2
B2

Baker/Perich/Sakahini

[redacted]

Job No. NAVFAC Station #53
No. .09501.01
Subject TRELLIS FRAMING
By AGB Date 06/01/03
Chkd. By _____ Date _____
Sheet No. A71 of _____

NORTH TRELLIS

- BEAM B1:

W_{DL} = 5 psf × 7.25 ft = 36.25 plf
W_{LL} = 20 psf × 7.25 ft = 145 plf

L = 11'-9"

(See Enercalc For Design)

USE HSS 5x3x3/16

- BEAM B2:

P_{DL} = 260 lbs } From B1 Reactions
P_{LL} = 850 lbs }

L = 18 ft

L = 3'-9"

(See Enercalc For Design)

USE HSS10x3x3/16

[redacted]

Steel Beam Design

Description: North Trellis B1

General Information

Distributed Loads

Summary

Force & Stress Summary

Section Properties

Title:
Job #:
Date:
Designer:
Description:
Scope:

Steel Beam Design

Description North Trellis - 6'

General Information

Steel Section : W8X31

Center Span		Fy		
Left Cant		Load Combination		
Right Cant		Deflection Method		
Unbraced Length				

Point Loads

	#1	#2	#3	#4	#5	#6	#7
Dead Load							
Live Load							
Wind Load							
Location							

Summary

Steel Section

	Actual	Allowable		
Moment			Max Deflection	
Shear			Length/Defl	
Max Deflection			Length/Defl Limit	

Force & Stress Summary

	Moment	Shear	Axial	Deflection	Length/Defl	Length/Defl Limit
Left Cant						
Left Cant @ Left						
Left Cant @ Right						
Center Span						
Center @ Left						
Center @ Right						
Right Cant						
Right Cant @ Left						
Right Cant @ Right						
Overall Max						
Overall Min						

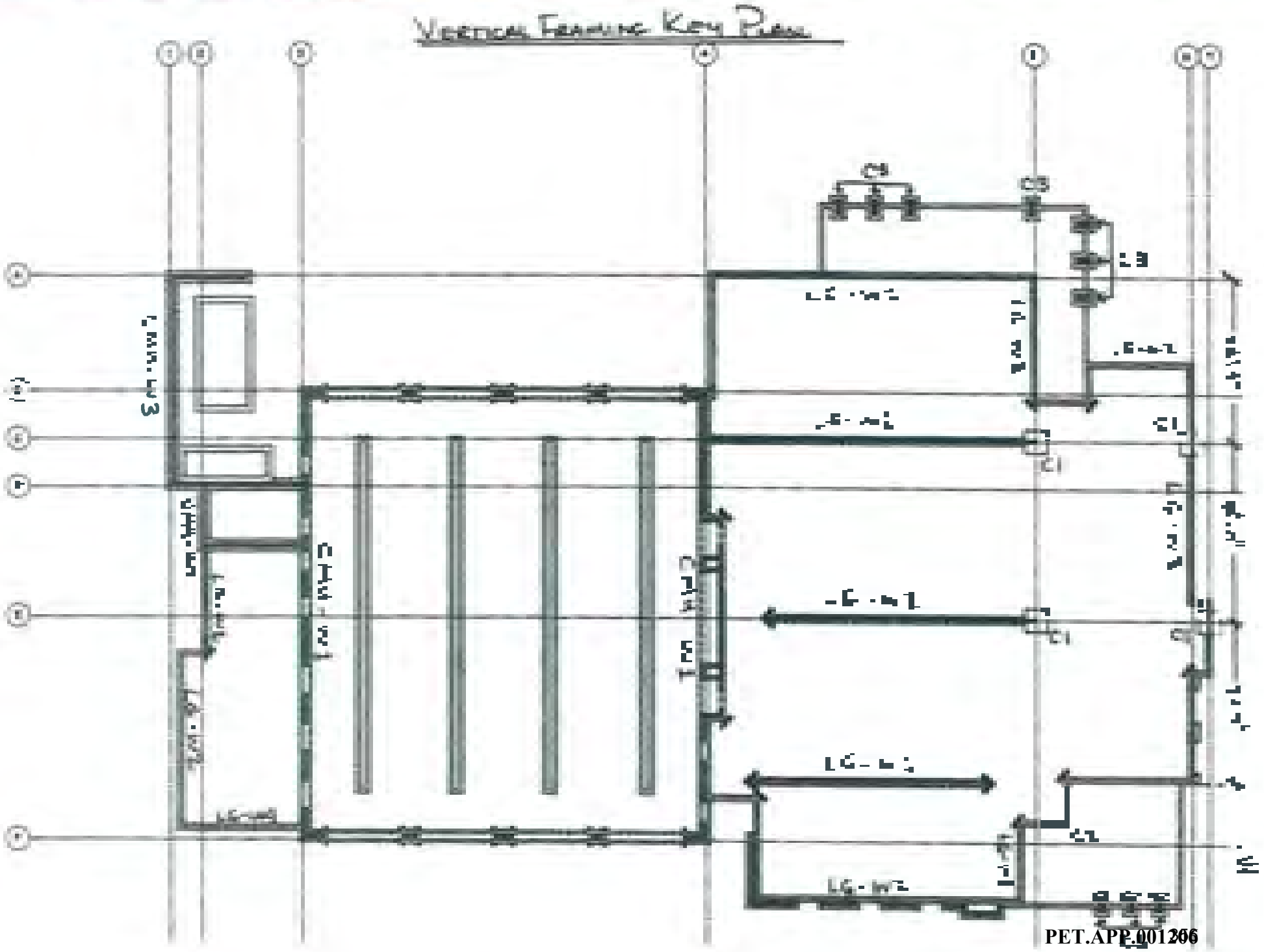
Section Properties

Depth		Weight	
Web Thick		Area	
Flange		Ix	
Flange Thick		Sx	
Area		Rx	
Iy		Sy	
Yielding Stress		Ry	
E		Zx	
Zy		J	

STRUCTURAL CALCULATIONS

City Of North Las Vegas
Fire Station 53
North Las Vegas, NV

Gravity Design – Vertical


VERTICAL FRAMING KEY PLAN
C3
C3
C1
C1
C1
C1
C2
C2
C2

Dekker/Perich/Sabatini

Project Name: MIXED STATION #53
Project Number: 07502.01
Subject: COLUMN DESIGN
By: JLB Date: 09/30/09
Checked: Date:
Sheet: 10 of

COLUMN DESIGN - COLUMN C1

LOADS: $P_{DL} = 8.9^k$, $P_{LL} = 8.4^k$ } FROM BEAM B2 REACTIONS

LENGTH = 15'-0"

(SEE ENERCALC DESIGN)

USE HSS 4x4x1/4 COLUMN

BEAM - C2

SL-2 REACTIONS:

$P_{DL} = 1150$ lbs
$P_{LL} = 1090$ lbs

SL-B REACTIONS:

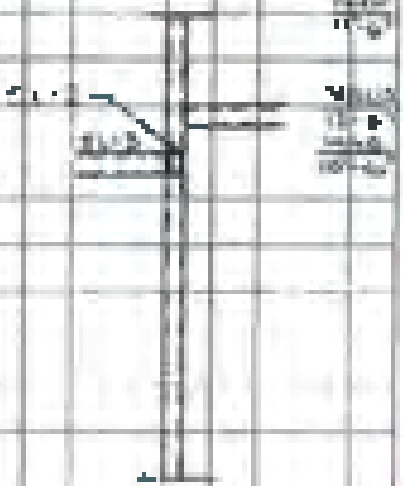
$P_{DL} = 1800$ lbs
$P_{LL} = 1610$ lbs

CANOPY BEAM REACTIONS:

$P_{DL} = 810$ lbs
$P_{LL} = 2,910$ lbs

(SEE ENERCALC FOR DESIGN)

USE HSS 4x4x1/4 COLUMN



Becker·Parich/Sabatini

Name: NWPD STATION #53
Number: 03502.01
Title: COLUMN DESIGN
By: AJB Date: 06/01/12
Checked: _____ Date: _____
Sheet: V3 of _____

CMU COLUMN C3

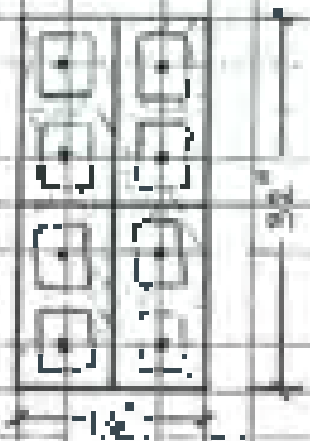
Loads:

P_{DL} = 500 lbs
P_{LL} = 1,300 lbs
} FROM NORTH TOWER B2

Wind = 20 psf

(SEE ENDECAVE FOR DESIGN)

USE 6" x 32" CMU COLUMN W/ #5's @ EA CELL



[redacted]

Steel Column

Description: Interior Column - C1

General Information

Code Ref: AISC 360, IBC, ASCE 7, ACI 318

Steel Section	[redacted]	Fy	[redacted]	Add Stiffeners	Reinforced
		Curtailed Fy	[redacted]	Yes	Appearance
Column Height	[redacted]	Beam Moment	[redacted]		
End Fully	Pinned	X-X Unbraced	[redacted]	Yes	[redacted]
Use 0.85 Short Term Loads Combined		Y-Y Unbraced	[redacted]	Yes	[redacted]

Loads

Axial Load:

Dead Load	[redacted]	Live Load Reduction	[redacted]
Live Load	[redacted]	Roof Live Load Reduction	[redacted]
Short Term Load	[redacted]		

Summary

Column Design OK

Section: [redacted], Height = [redacted], Axial Load = [redacted], Mx = [redacted], My = [redacted], Ecc = [redacted]

Unbraced Lengths: X-X = [redacted], Y-Y = [redacted]

Governing Load Case	Load	Axial	DL + LL	DL + LL + Eccentricity
1.4DC (Governing Eq. 1)	[redacted]	[redacted]	[redacted]	[redacted]
1.2DC + 1.6LC (Governing Eq. 2)	[redacted]	[redacted]	[redacted]	[redacted]
1.2DC + 1.6LC (Governing Eq. 3)				

LRFD Axial: Factored per Eq. E3-1, E7-1 + Ok

Y-Y Axial: Factored per Eq. E3-2, E7-2 + Ok

Summary

Allowable & Actual Stresses	Load	Axial	DL + LL	DL + LL + Ecc
Fa - Allowable	[redacted]	[redacted]	[redacted]	[redacted]
fa - Actual	[redacted]	[redacted]	[redacted]	[redacted]
Fbx - Allowable (F5-1)	[redacted]	[redacted]	[redacted]	[redacted]
fbx - Actual	[redacted]	[redacted]	[redacted]	[redacted]
Fby - Allowable (F5-1)	[redacted]	[redacted]	[redacted]	[redacted]
fby - Actual	[redacted]	[redacted]	[redacted]	[redacted]

Analysis Values

Pnx - Strength	[redacted]	Cmx - Coeff.	[redacted]	Gov't Eq. No.	[redacted]
Pny - Strength	[redacted]	Lby - Unbrd.	[redacted]	Gov't Eq. No.	[redacted]
Mnx - Strength	[redacted]	Cmx - Coeff.	[redacted]	Gov't Eq. No.	[redacted]
Mny - Strength	[redacted]	Cmy - Coeff.	[redacted]	Gov't Eq. No.	[redacted]
Mnx - Strength	[redacted]	Cmy - Coeff.	[redacted]	Gov't Eq. No.	[redacted]

[redacted]

Steel Solution

Description: Interior Column - G1

Section Properties

Depth	[redacted]	Weight	[redacted]		
Wall Thick.	[redacted]	A	[redacted]	J	[redacted]
Design Thick.	[redacted]	Ix	[redacted]	Cw	[redacted]
Flange Thick	[redacted]	Sx	[redacted]	Zx	[redacted]
Area	[redacted]	rx	[redacted]	Iy	[redacted]
Ix	[redacted]	Ro	[redacted]		[redacted]
		Ry	[redacted]		

Section Type = HSS-Square

Sketch & Diagram



[redacted]

Steel Column

Design Section: Interior Column - C2

General Information

[redacted]

Loading

[redacted]

Summary

[redacted]

Stresses

[redacted]

Analysis Values

[redacted]

[redacted]

Steel Column

Description: Interior Column - C3

Section Properties: HSS4X4X1/4

Depth	[redacted]	Weight	[redacted]	Values for LRFD Design	
Web Thick	[redacted]	Ag	[redacted]	J	[redacted]
Width	[redacted]	Ipy	[redacted]	Cw	[redacted]
Flange Thick	[redacted]	Ix	[redacted]	Zx	[redacted]
Area	[redacted]	Sy	[redacted]	Zy	[redacted]
rg	[redacted]	Rx	[redacted]		[redacted]
		Ry	[redacted]		

Section Type = HSS-Square

Sketch & Diagram

[redacted]



[redacted]

Masonry Column Design

Description: [redacted]

General Information

Code Ref: ACI 530-05

Column Height	[redacted] ft	Fm	[redacted] psi	Seismic Factor	[redacted]
Width	[redacted] in	Fy	[redacted] ksi	Reduction Factor	[redacted]
Depth	[redacted] in	Area of Steel	[redacted]	Wind Direction	[redacted]
Column Area	[redacted] sq in			Load Duration Factor	[redacted]
Pedestal	[redacted]			Service Temperature	
Bar Count Along Width	[redacted]				
Bar Size	[redacted]				
Pedestal Depth	[redacted] in				

Applied Loads

Service Load		Lateral Load	
Dead Load	[redacted] lb	Wind Load in Direction	[redacted] lb
Live Load	[redacted] lb	Design Shear	[redacted] lb
Eccentricity	[redacted] in	Load Case	[redacted]
Load Case	[redacted]	Wind Load	[redacted] psf

Design Values

Allowable Stress Formula	[redacted] psi	Fm	[redacted]	[redacted]	[redacted]
Fa [redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]		[redacted]	
[redacted]	[redacted]	[redacted]		[redacted]	
Max Allow Stress with Axial Load	[redacted] psi				
Max Allow Axial Load with Moment	[redacted] lb				

Load Combinations & Stress Details Summary

	Combination	Axial	Moment	Bending Stresses	Axial Stresses
	No.	lb	psi	Fb	Fa
Dead + Live Load	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Dead + Live + Wind	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Dead + Live + Seismic	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

Summary

Column Design OK

[redacted] Height = [redacted] Width = [redacted] Depth = [redacted] Using [redacted] Reinforcing Steel = [redacted]

Max Bending + Axial Comp. Stress	[redacted]	psi
Allowable	[redacted]	psi
Max. Axial Comp. Compressive Stress	[redacted]	psi
Allowable	[redacted]	psi
Max Shear Bending Stress	[redacted]	psi
Allowable	[redacted]	psi

Description: Cold Column - C3

Summary of Loads & Reactions

Vertical Loads...

Lateral Loads...

Controlled Reactions:

Becker·Perich/Sabatini

[redacted]

Job: MSFD STATION #43
Job No: 07501-01
Subject: BASE PLATES
By: A.LB Date: 06/01/07
Checked: VD Sheet:

- Base Plate A

Axial Loads

P = 8.9 + 8.42 = 17.32 k (From Column C3)

f'_c = 4,500 psi
F_y = 36,000 psi

(See Excel for Design)

Use 8"x10"x¾" Base Plate

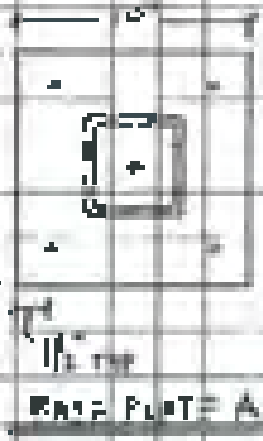
- Base Plate B

P = 17.32 k (Same as BP.A)

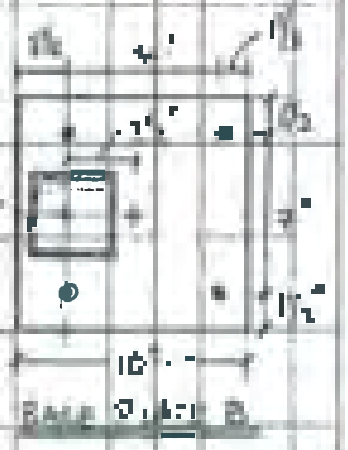
f'_c = 4,500 psi
F_y = 36,000 psi

(Same as A except Geometry)

Use 8"x10"x¾" Base Plate



Base Plate A



Base Plate B

Dekker/Perich/Sabatini

[redacted]

Name: HVAC Support #53
Job No.: 07502-01
Subject: BASE PLATES
By: AMB Date: 4/8/08
Checked: [redacted] Date: [redacted]
Sheet: 46

BASE PLATE C

ANALYSIS:

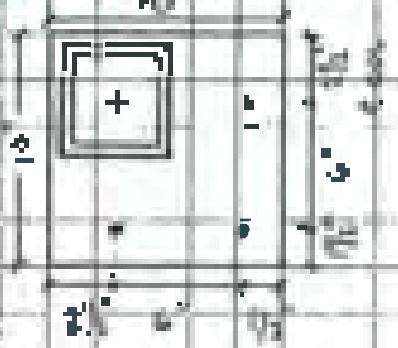
$T = 3.76k + 4.67k = 8.43k$ (FROM COLUMN C2)

$F_y = 1,500$ psi

$E_s = 30,000$ ksi

(SAME AS A DIFFERENT GEOMETRY)

USE 10"x10"x 1" BASE PLATE



[redacted]

Steel Column Base Plate

Description: Base Plate A

General Information

Loads		Steel Section	[redacted]
Axial Load	[redacted]	Section Length	[redacted]
Y-Y Axis Moment	[redacted]	Section Width	[redacted]
Plate Dimensions		Flange Thickness	[redacted]
Plate Length	[redacted]	Web Thickness	[redacted]
Plate Width	[redacted]	Strength Reduction	
Plate Thickness	[redacted]	Compression	[redacted]
Support Pier Size		Base Plate	[redacted]
Pier Length	[redacted]	Load Duration Factor	[redacted]
Pier Width	[redacted]	Section Bolt Data	
		Distance from Plate Edge	[redacted]
		Bolt Centers per Side	[redacted]
		Tensioning Capacity	[redacted]
		Bolt Area	[redacted]

Summary

Concrete Bearing Stress	Bearing Stress OK	
Actual Bearing Stress	[redacted] psi	
Allow per ACI 318-08 Sec. 10.14.1		Bolt Bearing - No Shear Transfer
Allow per ACI 318-08 Sec. 10.14.1 LDF	[redacted] psi	
Allow per AISC J8	[redacted] psi	
Plate Bending Stress	Reinforcement OK	
Actual fb	[redacted] ksi	
Allow Allow Plate fb	[redacted] ksi	
Required Bolt Area	Bolt Tension OK	
Actual Tension	[redacted] k	
Allowable	[redacted] k	

Bechtel/Parsch/Saboline

[redacted]

Project: NWD Station #33
Job No.: 07302-001
Subject: CMU Wall Design
By: AJB Date: 03/19/07
Chkd: ____ Sht: ____
Rev: WS

Wall Design - W1

$H_{wall} = 22.5$ Ft
$H_{parapet} = 5$ Ft

Tributary Area:

A2 = 35 Sf

A3 = 10.75 Sf

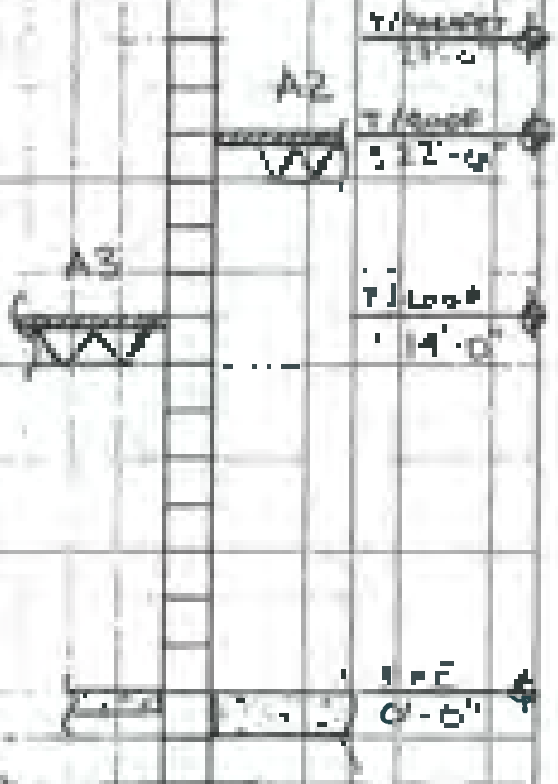
$W_{DL} = 20_{psf}(35 + 10.75)$
$= 915$ PLF

$W_{LL} = 915$ PLF

Seismic Factor = 0.409
(See Lateral Loads for Walls)

(See Excalc For Gravity Design)

12" CMU Wall w/ #6 @ 24" oc



Dekker/Perich/Sabatini

Project: NUFO_Signage #53
Project #: 07502.01
Subject: CMU Wall Design
By: AJB Date: 03/30/07
Check: Date:
Scale: NTS

CMU Wall - W2

W_{CMU} = 3.33 psf

H_{CMU} = 5 ft

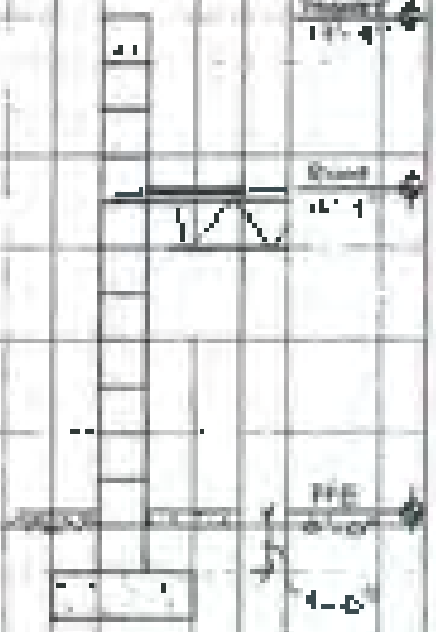
Loads:

W_{DL} = 20 psf x 3.33 ft = 200 plf

W_{LL} = 20 psf x 3.33 ft = 200 plf

(See Envelope for Design)

5' CMU Wall = 25 k @ 24" oc



Bakker/Parrish/Sabatini

[redacted]

NLVFD Station #53
07502.01
CMU Wall Design
AJB
05/30/07
[redacted]
[redacted]

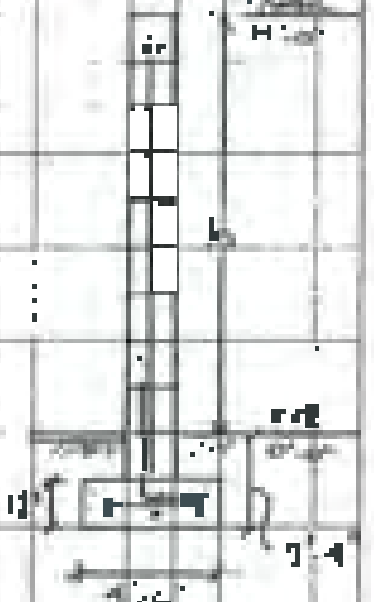
CMU Site Wall - W3

Soil Bearing Pressure = 2,000 psf
(1.2 P = 1,000 psf increased to 2,000 psf
Min. Code Reqd. Allow. Increase per depth & width of footing)

P_a = 42 psf
P_p = 275 psf

Surcharge:
At Top = 200 psf (Uniform Pressure)
at -4.0' = 20 psf (L)

f'_c = 1,500 psi
f'_m = 1,500 psi
F_y = 60,000 psi



- LATERAL LOADS:

* PER ASCE 7-05, Sec. 6.5.14

$F = q_z G C_f A_s$ (lb) [Eq. 6-27]

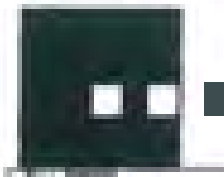
q_z = 18 psf
G = 0.85
C_f = 1.90 (Per Fig. 6-20)

Return Wall Reduction Factor

$\therefore P = q_z G C_f$ = 18 psf (0.85)(1.90)(0.95) = 31 psf

(See Engineering for Design)

* Use 8" CMU Wall w/#5 @ 8" o.c. (Vert.) Fully Grouted - (Std. No. 9 Dur-O-Wall Truss @ 16" o.c. Horiz.)



[redacted]

Masonry Wall Design

Description CMU Wall - W1

Material Information

Loads

Design Values

Load Combinations & Stress Details Summary

Summary

PET.APP.001221

[redacted]

[redacted]

[redacted]

[redacted]

Masonry Wall Design

Page 1

General Information

Wall Height	10.33 ft	Seismic Factor	0.4000	Fm	1,500.0 psi
Panel Height	8.33 ft	Calculated Axial Pu	800.00	Fs	24,000.0 psi
		Axial Load Applied	1,148	Special Inspection	
Thickness	8 in	Wall Self Wt	1.000	Grout @ Rebar Only	
Bond Beam	8			Normal Weight Block	
Rebar Spacing	24 in			Equivalent Solid Thickness	6.000 in
Grade of Rebar	60 ksi	@ Center			

Loads

Uniform Load		Concentrated Axial Load		Area Load	30.000 psf
Dead Load	300.000 plf	Dead Load	1,000 lbs		
Live Load	200.000 plf	Live Load	0.000 lbs		
Load Eccentricity	4.000 in	Axial Load			
Roof Load					

Design Values

f'm	1,500.000 psi	Wall Area	0.000 in2	n=	20.000	fr	0.000 psi
Es / Em	20 ksi	Modulus of Elasticity	2,000 ksi	r	2.309 in	h/r	53.70
Allow Compression	0.300 f'm	Moment of Inertia	237.333 in4				

Max Allow Axial Load = 0.20 f'm [1 - (h/140r)^3] Ag = 240.00 psi

Allow Masonry Bending Stress = 0.33 f'm = 495.00 psi

Allow Steel Bending Stress = 24,000.00 psi

Load Combination 1 Stress Details Summary

	Moments	Axial Load	Bending Stresses		Axial Compressions
Stress Types	in-k	lbs	Rebar psi	Masonry psi	psi
DL + LL	7.400.3	3000.0	4,354.0	197.5	13.89
DL + LL + Wind	8.128.8	3000.0	5,284.4	1,284.4	13.89
DL + LL + Seismic	5,284.4	3000.0	3,440.7	748.1	13.89
Reduced Base & Footing					
DL + LL	7,400.3	3000.0	4,811.4	1,811.4	Modes
DL + LL + Wind	8,128.8	3000.0	5,741.8	2,741.8	1945
DL + LL + Seismic	5,284.4	3000.0	3,898.1	898.1	1945

Summary

10.333 high wall with a 0.00 parapet Normal Block at 8.00in wall on 00.00in x 24.00in x 0.00in deep

Max Bending Compressive Stress	293.28	psi
Allowable	495.00	OK
Max Axial Only Compressive Stress	13.89	psi
Allowable	202.50	OK
Max Steel Bending Stress	5,741.80	psi
Allowable	21,600.00	OK

[redacted]

[redacted]

[redacted]

Dobler-Jerich/Bakalinni

Project Name: NLNEO Station #038
Project Number: 09502-01
Subject: CMH Wing Design
By: AJB Date: 10/27/08
Checked: Date:
Sheet: Wall of

CMH Site Wall - W3

Soil Bearing Pressure (SBP) = 2,000 psf

↓ SBP = 1,100 psf → Increased per soil report allowable increase per depth & width of footing.

P_A = 42 pcf

P_P = 245 pcf

- Surcharge:
  - At toe = 200 psf (Guardrail P_{LO})
  - At heel = 20 psf (LL)

f'_c = 4,500 psi

f_{cm} = 1,500 psi

F_y = 60,000 psi

- Lateral Loads:

* ASCE 7-05, Sect. 6.5.14

$F = q_h G C_a A_s$ [Eq. 6-27]

B = 56.67 ft

h = 14 ft

B/h = 2.6

s/n = 1.0



Dekker/Perich/Sabatini

Project: NAVEO Station #53
Project No.: 99302-01
Title: CMU Wall Design
By: AJE Date: 10/29/01
Checked: Date:
Page: WH

* Per Table 6.7.2D

For Case A & B: $C_f = 1.9$

For Case C:

Region 0-3	$C_f = 2.39$	$\Rightarrow$	Reduction (0.8 - V_h) 1.911 CONTROLS
3-23	$C_f = 1.58$	$\Rightarrow$	1.26

* Per Note 1, for case C w/ $V_h \geq 0.8$, C_f from table must be multiplied by the reduction factor [(1.3 - V_h) = 1.3 - 1.0 · 0.8]

$q_h = 10.9$ psf (windward) / -12.1 psf (leeward) (Eq. 6-15) Const. & Cladding

$G = 0.85$ (Sect. 6.5.8)

$P = 12.1 \text{ psf } (0.85)(1.911) = 19.69$ psf

(Use Envelope for Design)

— Shear Reinforcement:

$V = 19.90 \text{ psf} \times (36.67 \text{ ft} \times 19 \text{ ft}) = 9,087 \#$

$f_v = \frac{V}{bd} = \frac{9,087 \text{ lbs}}{(7.625 \times 428 \text{ in})} = 2.78$ psi

C_M Inspection $\Rightarrow$ Shear Reinforcement is adequate

$A_v \text{ (min)} = 0.0007 (7.625)(12"/ft) = .07 \text{ in}^2/\text{ft}$

376

PET.APP.001226

Netter·Petrich/Tabulini

MUNI SYSTEMS #53
07502-01
CMU WALL DESIGN
AAB 10/19/07
1/22

$A_{(DUR-O-WALL)} = 0.049 \frac{in^2}{PAIR} \times \frac{12"}{16" oc} = 0.0735 \frac{in^2}{ft}$

$A_{(BOND\ BEAMS)} = 0.61 in^2 \times \frac{12"}{48" oc} = 0.1525 \frac{in^2}{ft}$

$A_{TOTAL} = 0.204 \frac{in^2}{ft} > 0.064 \frac{in^2}{ft}$ OK

USE 8" CMU WALL w/ #5 @ 8" oc. (VERT.)
FULLY GROUTED AND HORIZONTAL REINF.
OF 3½" HEAVY DUR-O-WAL TRUSS @ 16" oc.
w/ 2 #5's @ 48" oc BOND BEAMS.

Cantilevered Retaining Wall Design

Description: CMU Wall - W3 (Generator Pad)

Criteria

Soil Data

Footing Strengths & Dimensions

Surcharge Loads

Design Summary

Stem Construction

Footing Design Results

[redacted]

[redacted]

Cantilevered Retaining Wall Design

Page 3

Description: [redacted] (Generator Pad)

Summary of Overturning & Resisting Forces & Moments

		Overturning Forces			Resisting Forces		
		Force lbs	Distance ft	Moment ft-lb	Force lbs	Distance ft	Moment ft-lb
[redacted]							
Total							

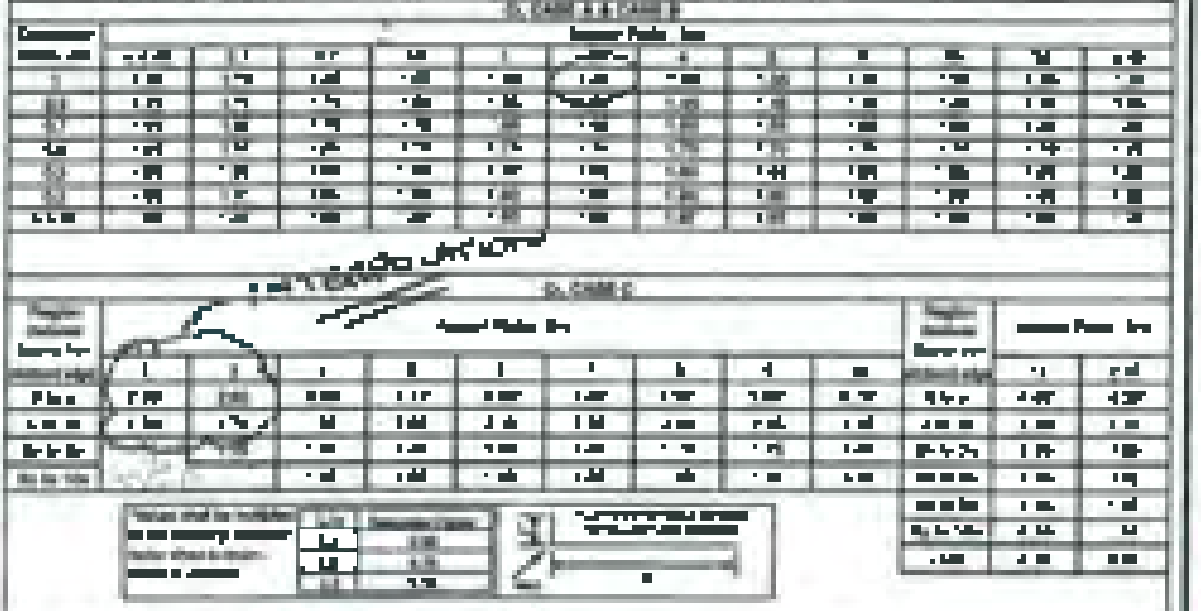
Overturning/Overturning Ratio = [redacted]

Vertical Loads used for Soil Pressure = [redacted] lbs

Factored Moments of Vertical Loads used for Soil Pressure

[illegible] – Method 2		All Heights
Figure 6-28	Force Coefficients, C_f	Solid Freestanding Walls & Solid Signs





Notes:

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]
6. [illegible]
7. [illegible]

Figure 6-3 (cont'd)	Net Design Wind Pressures	Walls & Roofs
Enclosed Buildings		

Net Design Wind Pressures, p_{net} (psf) [illegible]

Zone	Effective Area sq ft	Basic Wind Speed V (mph)											
		85		90		100		105		110		120	
Roof Overhangs	1	[illegible]	7.1	-11.8	7.7	-14.0	7.9	-13.9	8.1	-15.1	8.4	-16.3	9.1	-18.8
	1	[illegible]	7.1	-10.7	7.7	-11.7	8.5	-12.1	8.8	-13.1	9.1	-14.2	9.8	-16.5
	1	[illegible]	7.1	-10.7	7.7	-11.7	8.5	-12.1	8.8	-13.1	9.1	-14.2	9.8	-16.5
	1	[illegible]	7.1	-10.7	7.7	-11.7	8.5	-12.1	8.8	-13.1	9.1	-14.2	9.8	-16.5
	2	[illegible]	7.1	-16.1	7.7	-17.5	8.5	-18.2	8.8	-19.7	9.1	-21.3	9.8	-24.8
	2	[illegible]	7.1	-16.1	7.7	-17.5	8.5	-18.2	8.8	-19.7	9.1	-21.3	9.8	-24.8
	2	[illegible]	7.1	-16.1	7.7	-17.5	8.5	-18.2	8.8	-19.7	9.1	-21.3	9.8	-24.8
	3	[illegible]	7.1	-21.4	7.7	-23.3	8.5	-24.2	8.8	-26.2	9.1	-28.3	9.8	-33.0
	3	[illegible]	7.1	-21.4	7.7	-23.3	8.5	-24.2	8.8	-26.2	9.1	-28.3	9.8	-33.0
	3	[illegible]	7.1	-21.4	7.7	-23.3	8.5	-24.2	8.8	-26.2	9.1	-28.3	9.8	-33.0
Roof Overhangs	1	[illegible]	7.1	-11.8	7.7	-12.9	8.5	-13.4	8.8	-14.5	9.1	-15.7	9.8	-18.3
	1	[illegible]	7.1	-11.8	7.7	-12.9	8.5	-13.4	8.8	-14.5	9.1	-15.7	9.8	-18.3
	1	[illegible]	7.1	-11.8	7.7	-12.9	8.5	-13.4	8.8	-14.5	9.1	-15.7	9.8	-18.3
	1	[illegible]	7.1	-11.8	7.7	-12.9	8.5	-13.4	8.8	-14.5	9.1	-15.7	9.8	-18.3
	2	[illegible]	7.1	-11.8	7.7	-12.9	8.5	-13.4	8.8	-14.5	9.1	-15.7	9.8	-18.3
	2	[illegible]	7.1	-11.8	7.7	-12.9	8.5	-13.4	8.8	-14.5	9.1	-15.7	9.8	-18.3
	2	[illegible]	7.1	-11.8	7.7	-12.9	8.5	-13.4	8.8	-14.5	9.1	-15.7	9.8	-18.3
	3	[illegible]	7.1	-18.1	8.1	-34.3	8.5	-20.6	8.8	-22.3	9.1	-24.0	9.8	-27.8
	3	[illegible]	7.1	-18.1	7.7	-19.7	8.5	-20.6	8.8	-22.3	9.1	-24.0	9.8	-27.8
	3	[illegible]	7.1	-18.1	7.7	-19.7	8.5	-20.6	8.8	-22.3	9.1	-24.0	9.8	-27.8
	3	[illegible]	7.1	-18.1	7.7	-19.7	8.5	-20.6	8.8	-22.3	9.1	-24.0	9.8	-27.8
Roof	1	[illegible]	10.4	-14.8	11.3	-16.0	11.7	-16.6	12.0	-17.9	12.4	-19.3	13.3	-22.3
	1	[illegible]	10.4	-16.5	11.3	-17.9	11.7	-18.5	12.0	-20.0	12.4	-21.5	13.3	-24.8
	1	[illegible]	10.4	-21.5	11.3	-23.3	11.7	-24.2	12.0	-26.1	12.4	-28.1	13.3	-32.4
	1	[illegible]	10.4	-36.1	11.3	-39.1	11.7	-40.6	12.0	-43.8	12.4	-47.1	13.3	-54.3
	2	[illegible]	10.4	-14.8	11.3	-16.0	11.7	-16.6	12.0	-17.9	12.4	-19.3	13.3	-22.3
	2	[illegible]	10.4	-16.5	11.3	-17.9	11.7	-18.5	12.0	-20.0	12.4	-21.5	13.3	-24.8
	2	[illegible]	10.4	-21.5	11.3	-23.3	11.7	-24.2	12.0	-26.1	12.4	-28.1	13.3	-32.4
	2	[illegible]	10.4	-36.1	11.3	-39.1	11.7	-40.6	12.0	-43.8	12.4	-47.1	13.3	-54.3
	3	[illegible]	10.4	-14.8	11.3	-16.0	11.7	-16.6	12.0	-17.9	12.4	-19.3	13.3	-22.3
	3	[illegible]	10.4	-16.5	11.3	-17.9	11.7	-18.5	12.0	-20.0	12.4	-21.5	13.3	-24.8
	3	[illegible]	10.4	-21.5	11.3	-23.3	11.7	-24.2	12.0	-26.1	12.4	-28.1	13.3	-32.4
	3	[illegible]	10.4	-36.1	11.3	-39.1	11.7	-40.6	12.0	-43.8	12.4	-47.1	13.3	-54.3

Note: [illegible]

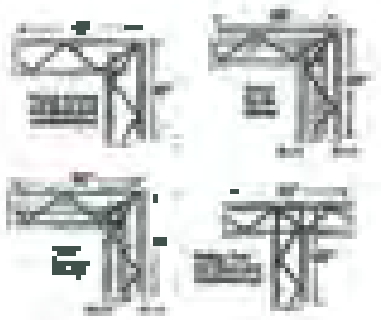
Bolts, washers and nuts required where no other attachment is specified.

Where one of DUR-O-WAL Truss Tie or Dovetail Anchor slots are also accepted by the Block, additional or staggered block courses may be required. Consult the DUR-O-WAL Technical Department for assistance.

See DUR-O-WAL publications for more detailed information: "Reinforcing Masonry Walls with Joint Reinforcement, A Design Aid" Bulletin 83-7; "Building Reinforced Masonry Walls with Joint Reinforcement" Bulletin 83-12R and for information for the use of DUR-O-WAL to resist horizontal bending see "DUR-O-WAL Masonry Reinforcement in High-Rise, Shearing Wall Buildings" Bulletin 74-5.

Prefabricated Corners and Tees

A complete line of prefabricated corner and tee units are available in all design types and finishes. If a requirement is designated corners are made or available other than Truss or Double Rectangle types.



Calculation Method

By supplying the amount of steel supplied by DUR-O-WAL, I have used the following method.

Area of Steel = As Required x LF

Spacing

Example:

Area supplied by

3.040 x .12 = 0.365 in²/ft

12

This is less than that required, (when face bolts shown on page 3&4)

Add 1 each small bolt in each course should supply

0.365 - 0.25 = 0.115 in²/ft

1

which exceeds the minimum requirements.

Warranty

Seller makes no warranty of any kind, express or implied, except that the goods sold shall be of merchantable quality, of seller's standard manufacture and free from defects resulting from the use of the goods, whether used singly or in conjunction with other goods. Seller neither assumes nor authorizes any person to assume for Seller any other liability in connection with the sale or use of the goods sold and there is no oral agreement or warranty collateral to or affecting this transaction.

DUR-O-WAL

Bekker/Perich/Sabatini

Project: MIXED SERVICES #53
Project No.: 03502-01
Subject: STUD WALL DESIGN
By: LMPS Date: 05/04/10
Chkd: VLF

STUD WALL - W1

- JOIST BEARING STUDS

P_{DL} = 5 psf

$P_{DL} = \left(\frac{28'-9" + 33'-0"}{2}\right) \times 6'-0" \times 20\ psf = 3,585\ lbs$

$P_{LL} = 3,585\ lbs$

H = 14'-7½" (175.5 in)

$P_{allow} = 3,300\ lbs/stud$ (INTERPOLATED BETWEEN 55MA TABLES)

USE (3) 600S162-43 @ JOIST BEARING

$w/P_{allow} = 3 \times 3300 = 7,900\ lbs > 7,170\ lbs$ ∴ OK

- FIELD STUDS:

H = 14'-7½"

S = 16" o.c.

Limiting Wall Height = 28'-9" (2 spans) > 14'-7½" ∴ OK

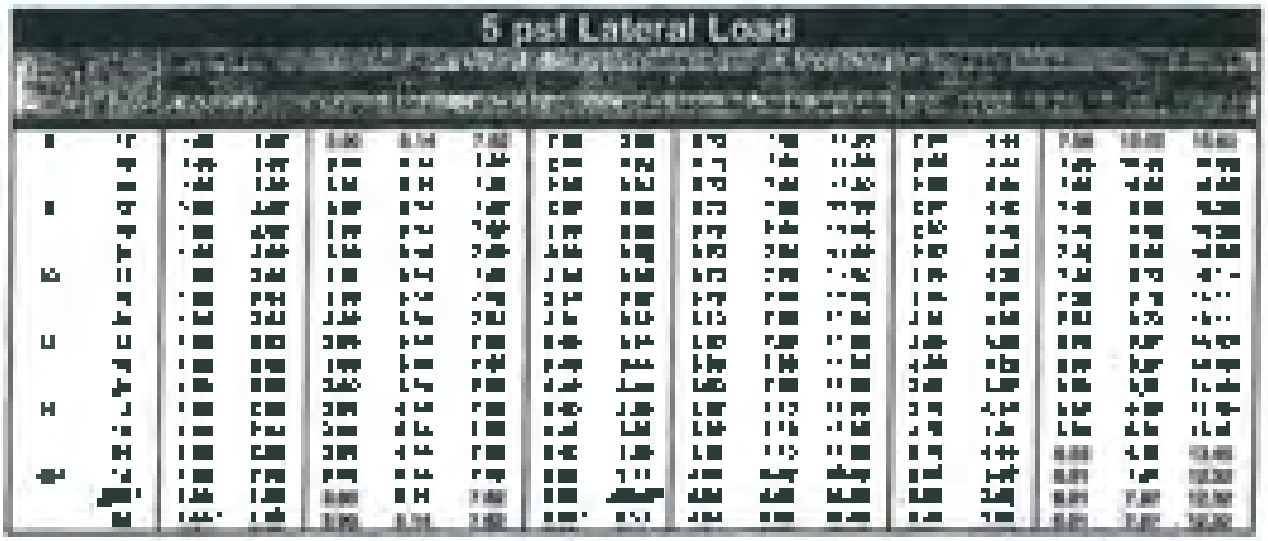
$P_{allow} = 3,300\ lbs$

$P = (29.875\ sf)(10\ psf)(14.625\ ft) = 1,513\ lbs < P_{allow}$ ∴ OK

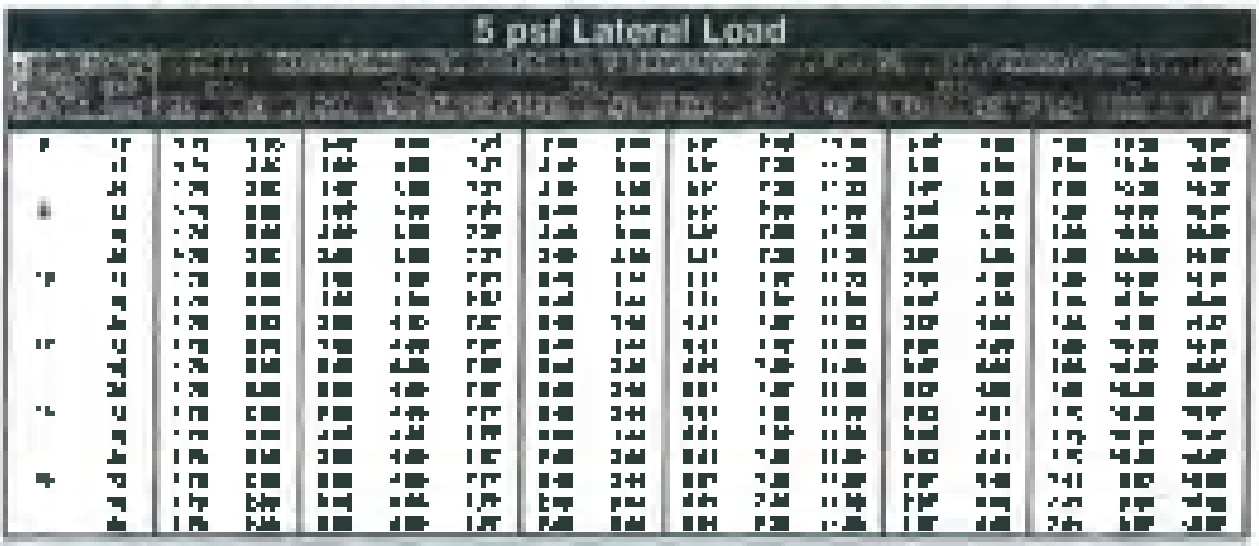
USE 600S162-43 @ 16" o.c. STUD WALL

Combined Axial and Lateral Load Tables

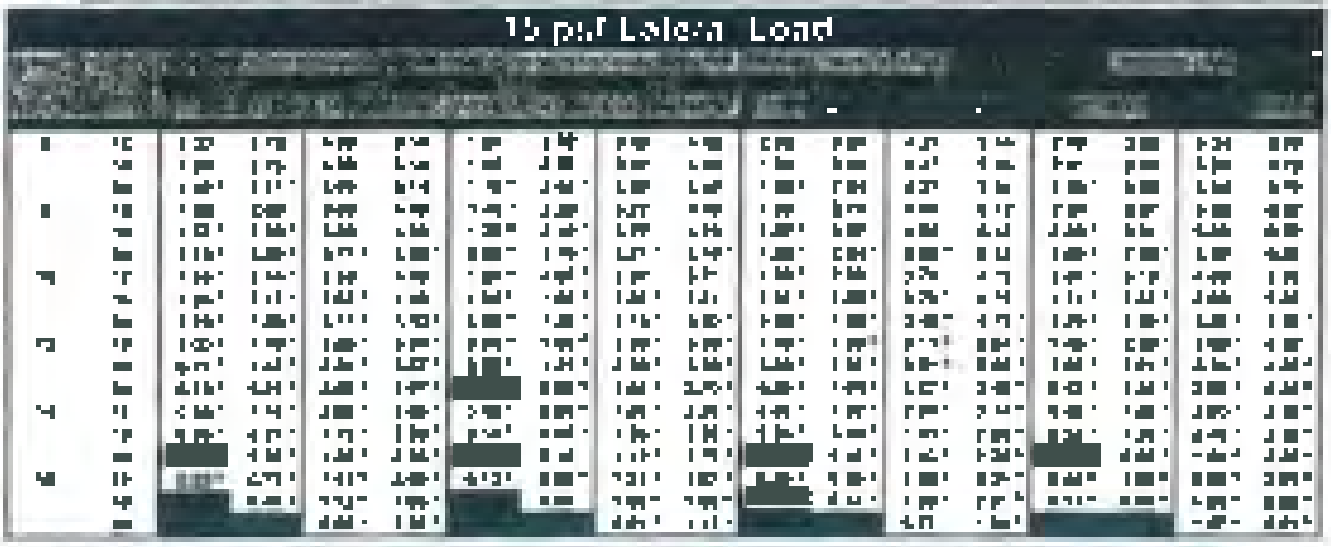
5 psf Lateral Load



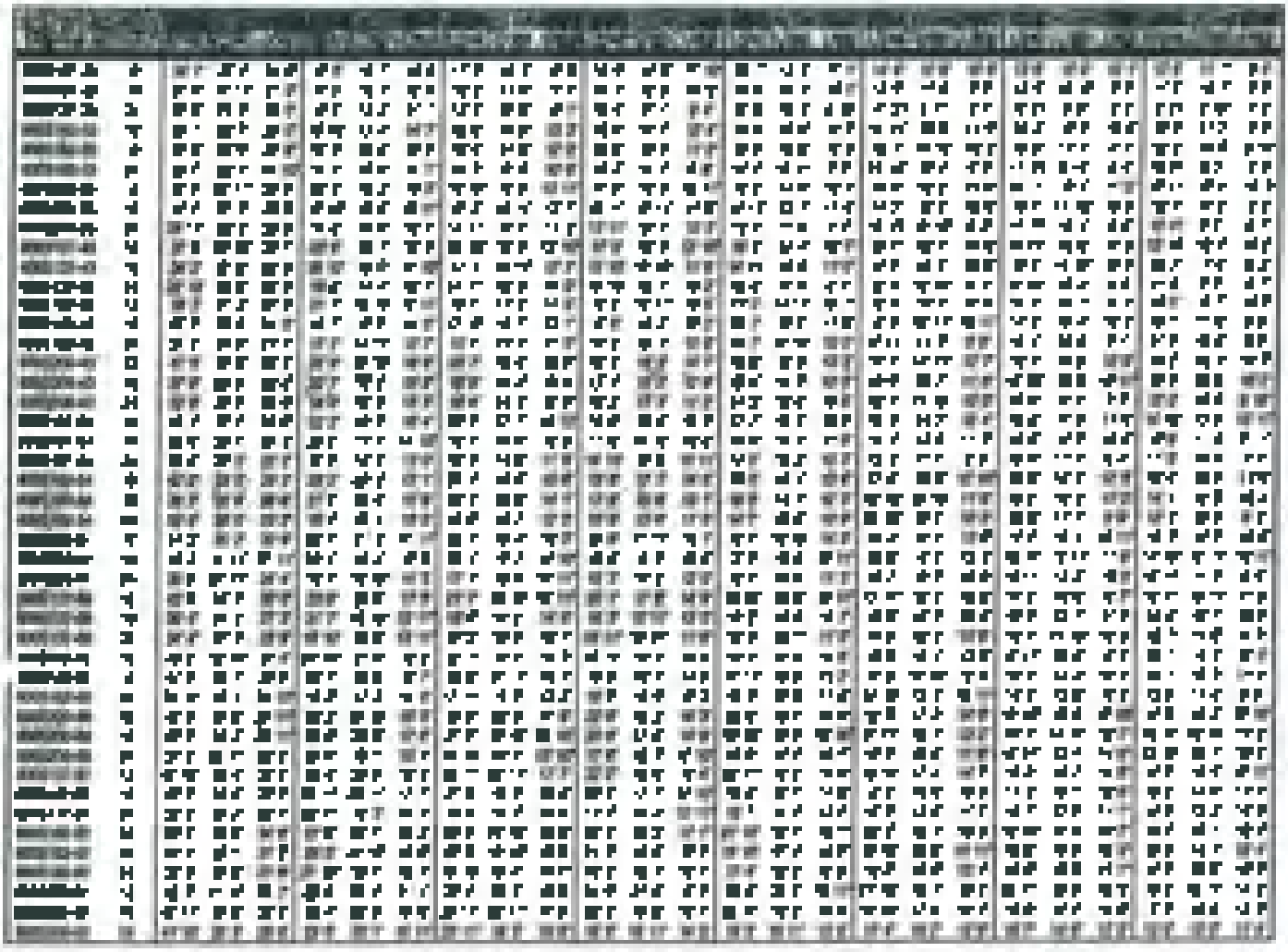
5 psf Lateral Load



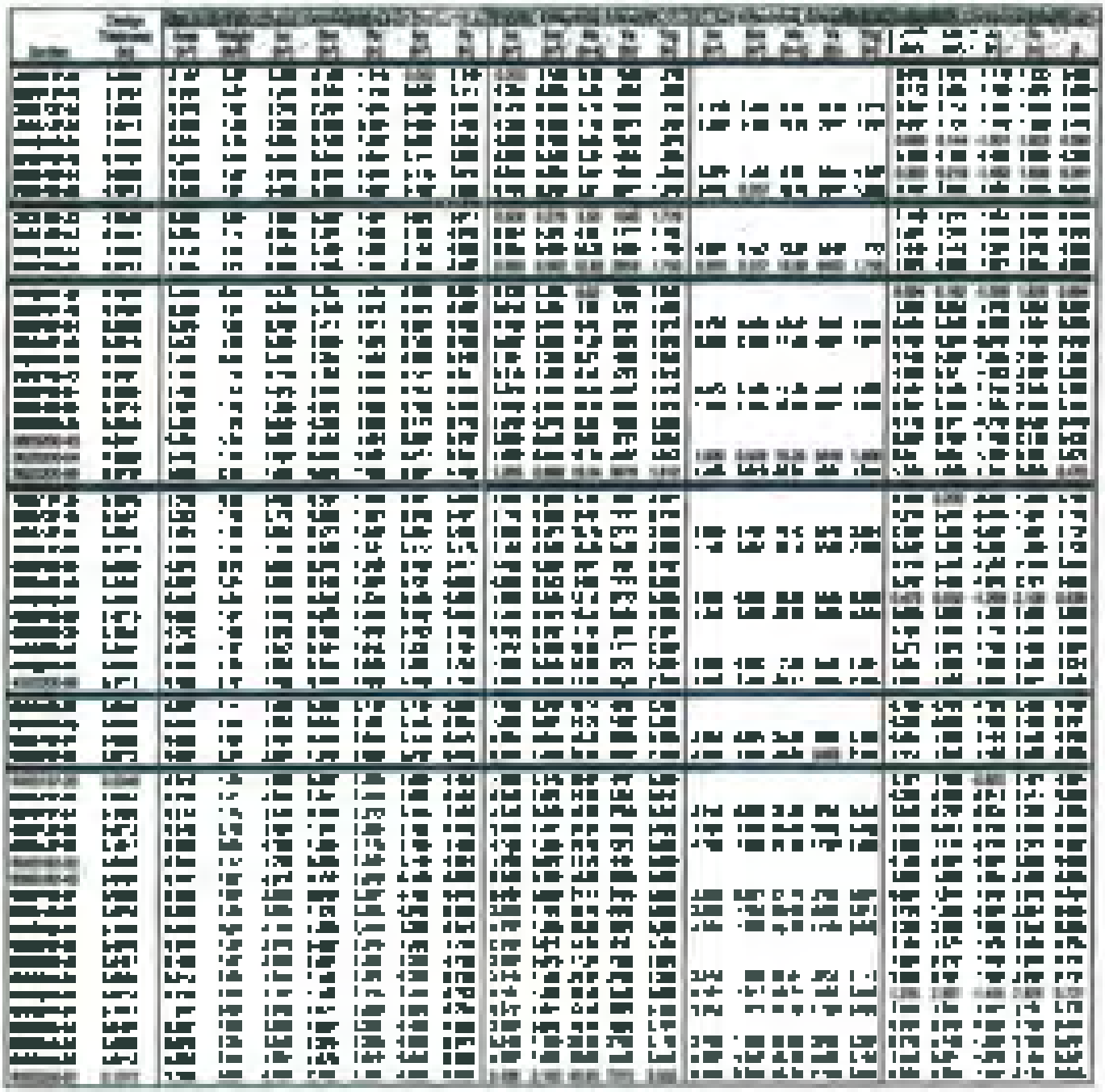
15 psf Lateral Load



Limiting Wall Heights — Curtain Wall



Structural (S) Stud Section Properties



STUD WALL - W2

- JOIST BEARING STUDS:

* FOR OUT-OF-PLANE LOADS CONSIDER COMPONENT & CLADDING WIND PRESSURES TO GOVERN PER ASCE 7-05 SECT. 6.4.2.2

$P_{NET} = \lambda K_{ZT} I p_{NET30}$ [EQ. 6-2]

$\lambda = 1.21$ (FIG. 6-3, EXP. C)

$K_{ZT} = 1.0$

$I = 1.5$

$p_{NET30} = 12.1$ PSF (FIG. 6-3)
to WINDS PRESSURE @ WALL

$P_{NET} = 1.21(1)(1.5)(12.1\text{ psf}) = 21.96$ PSF

$P_{SL} = \left(\frac{28'-9'}{2}\right) \times 6\text{ PSF} \times 20\text{ PSF} = 1,725$ #

$P_{LL} =$ (1) $= 1,725$ #

$P_{TOTAL} = 3,450$ #

$H = 19'-6''$

$P_{ALLOW} = 2,600$ lbs/STUD (FROM SSMA TABLES)

USE (2) 600S162-43 @ JOIST BEARING

$P_{ALLOW} = 2 \times 2,600$ # $= 5,200$ lbs $> 3,450$ lbs

Bechtel Parsons Brinckerhoff

Name: NLVFD Station #63
Project Number: 078502-01
Subject: Stud Wall Design
By: A-B Date: 10/30/07
Checked: Sheet:
File: V33

- Field Studs:

H = 14.5 ft

S = 16" o.c.

Limiting Height = 17'-10" (7/16) > 14.5 ft ∴ OK

P = (14.395 ft)(40 psf)(16"oc / 12"/ft) = 767 lbs
(DL + LL)

P_allow = 2,600 lbs >> 767 # ∴ OK

∴ Check P_PARAPET:

w = 21.96 psf × 1 ft²/144 in² × 16" oc = 2.44 lb/in

M = (2.44 lb/in)(90 in)² / 2 = 9,882 lb-in

M_a = 16.48 k-in > M ∴ OK

- Check Deflection:

EI = 6.216 × 10⁶ lb-in²

w = 2.44 lb/in

Δ = wL⁴ / 8EI = (2.44 lb/in)(90 in)⁴ / 8 × 6.216 × 10⁶ lb-in² = 0.298 in

Δ ≤ L/201 ∴ OK

V54

Figure 6-3 (cont'd)	Net Design Wind Pressures	Walls & Roofs
Enclosed Buildings		

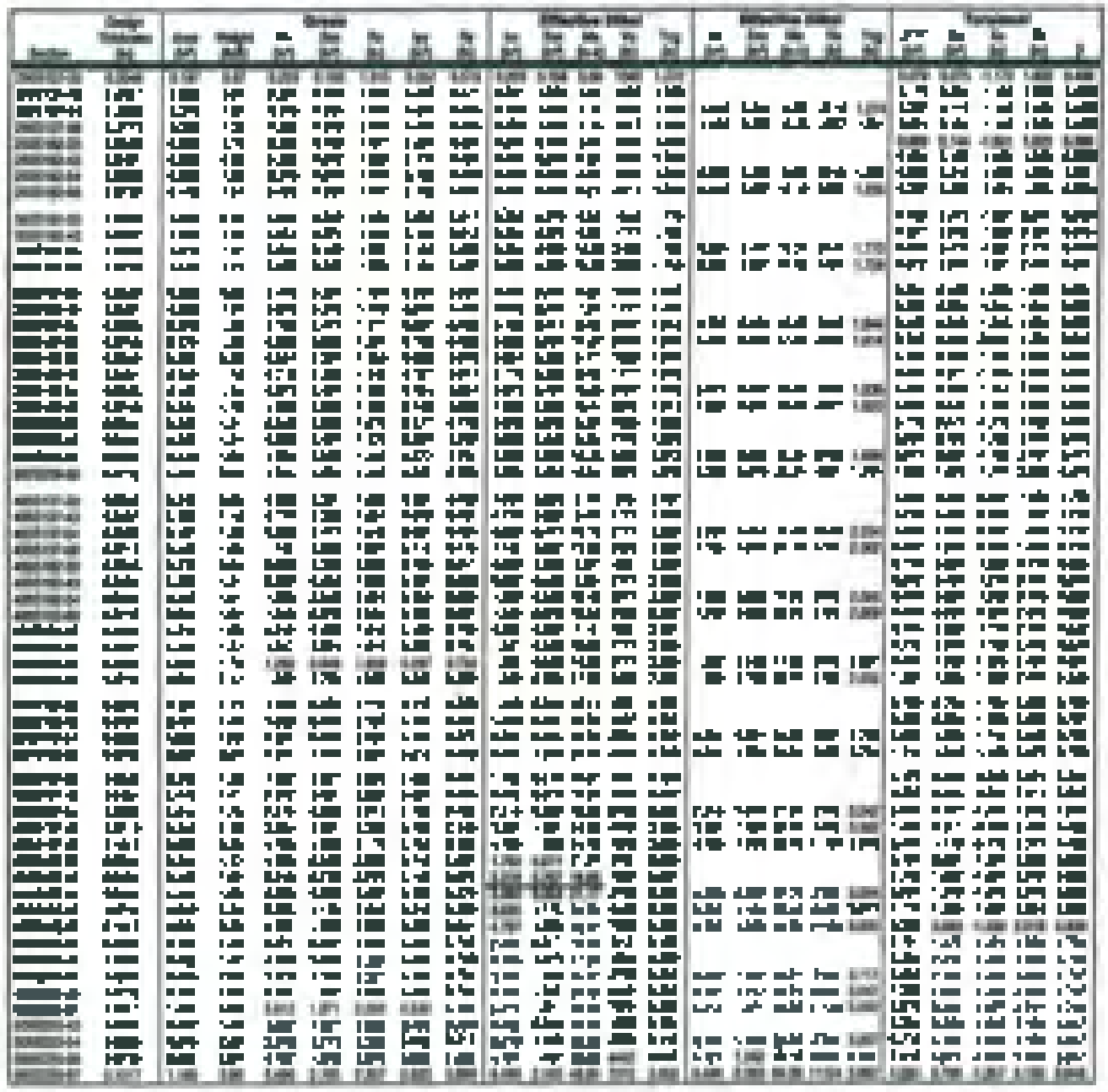
Net Design Wind Pressures, [illegible]

	Zone	Effective Wind Area sq ft	Basic wind Speed V (mph)											
			85		90		100		105		110		120	
[illegible]	5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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	2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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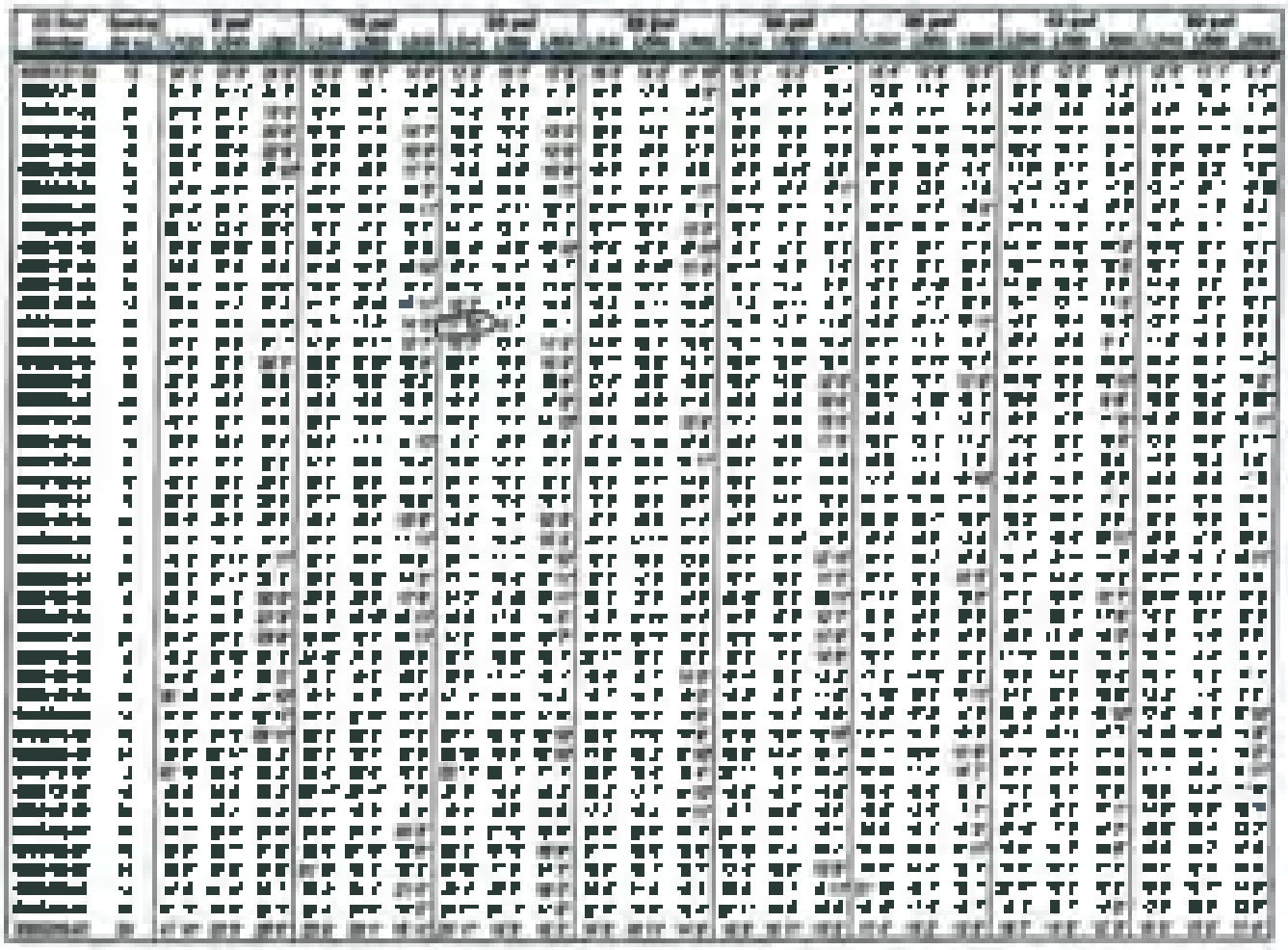
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Section Properties

Structural (S) Stud Section Properties



Limiting Wall Heights — Curtain Wall

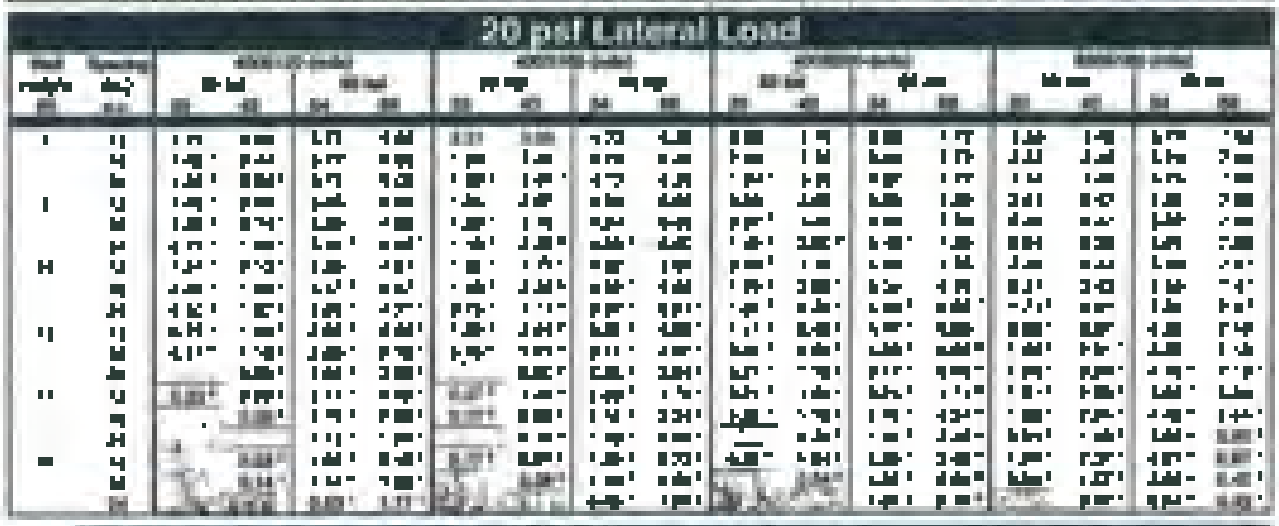


Combined Axial and Lateral Load Tables

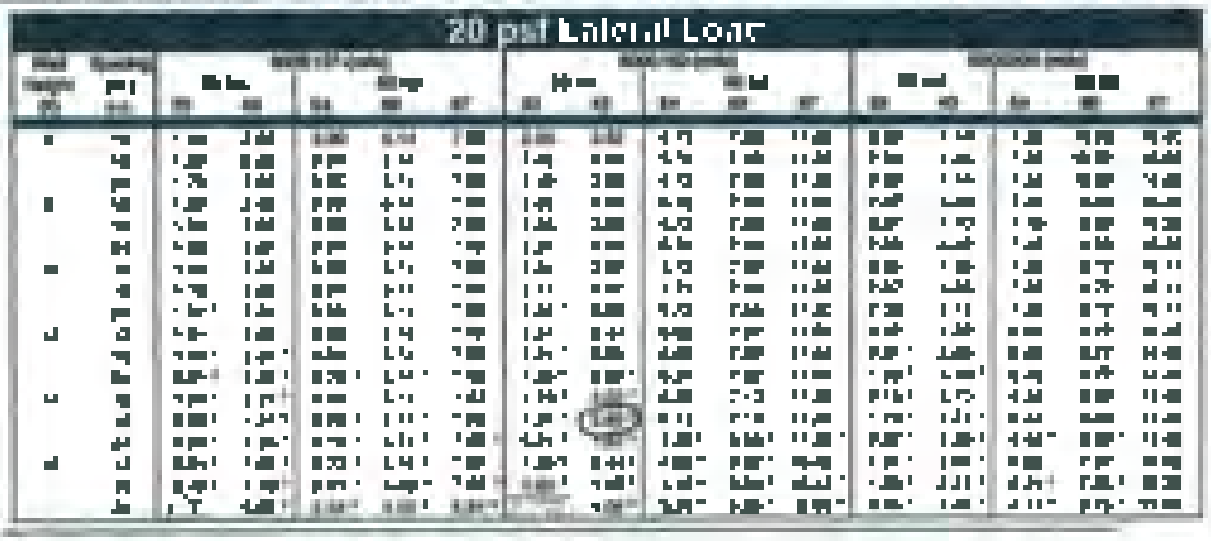
20 psf Lateral Load



20 psf Lateral Load



20 psf Lateral Load



STUD WALL - W3

- NON-BEARING STUDS

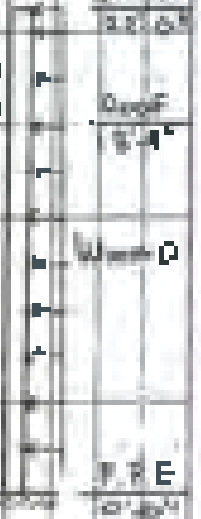
* PER ASCE 7-05 SECTION 13.3.1

$$F_p = \frac{0.4\, a_p\, S_{DS}\, W_p \left(1 + 2\frac{z}{h}\right)}{\left(\frac{R_p}{I_p}\right)}$$

WHERE:

$a_p = 1.0$
$R_p = 2.5$
$S_{DS} = 0.68$ } PER TABLE 13.5-1
$I_p = 1.5$

$\frac{z}{h} = \frac{13.33\,ft}{22\,ft} = 0.61$



$$F_p = \frac{0.4(1.0)(0.68)(15\,psf)}{\left(\frac{2.5}{1.5}\right)}\left(1 + 2(0.61)\right)$$

$F_p = 5.44\ psf$

⇒ USE WIND SINCE IT'S MORE CONSERVATIVE

$W_{wind} = 14.81\ psf \approx 15\ psf$

- LOADS:

$P_{DL} = 3\,ft(20\,psf)\left(\frac{4\,in}{12\,in/ft}\right) = 80\ lbs$
P_{LL}

- $M = P_a = 160\ lbs \times \frac{3\,ft}{12.7\,ft} = 40\ lb\text{-}ft$

Las Vegas, NV 89119
Tel: 702-436-1960

Single Span Beam Analysis

Description: Non Bearing Slid Wall - W3

General Information

Design Span	15.00 ft	Moment of Inertia	2,148 in4
Left Cantilever	0 ft	Beam Material	36,000 psi
Right Cantilever	0.00 ft	Beam Shape / Type	Plate

Uniform Loads

On Center Span...		On Left Cantilever...		On Right Cantilever...	
W1	0.000 k/ft	W1	0 k/ft	W1	0.000 k/ft

Moments

Magnitude	0.00 k-ft	0 k-ft	0 k-ft	0 k-ft
Location	13.500 ft	0 ft	0 ft	0 ft

Query Values

Center Location	0.000 ft	Left Side	0.000 ft	Right Side	15.000 ft
Moment	0.00 k-ft		0.00 k-ft		0.00 k-ft
Shear	0.000 k		0.000 k		0.000 k
Deflection	0.00000 in		0.00000 in		0.00000 in

Summary

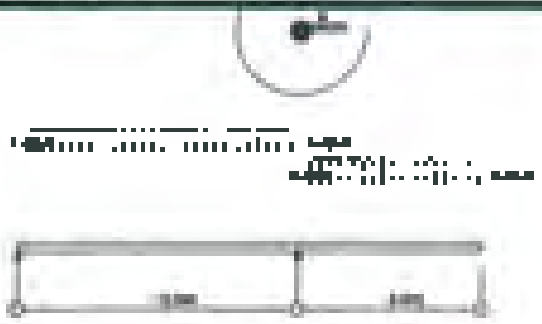
Moments				Shears		Reactions	
Max + @ Center	0.00 k-ft	at	0.000 ft	@ Left	0.000 k	@ Left	0.000 k
Max - @ Center	0.00 k-ft	at	0.000 ft	@ Right	0.000 k	@ Right	0.000 k
@ Left Side	0.00 k-ft			Maximum	0.000 k		
@ Right Side	0.00 k-ft			Deflections			
Maximum =	0.00 k-ft			@ Center	0.000 in	at	0.000 ft
				@ Left Side	0.000 in	at	0.000 ft
				@ Right Side	0.000 in	at	0.000 ft

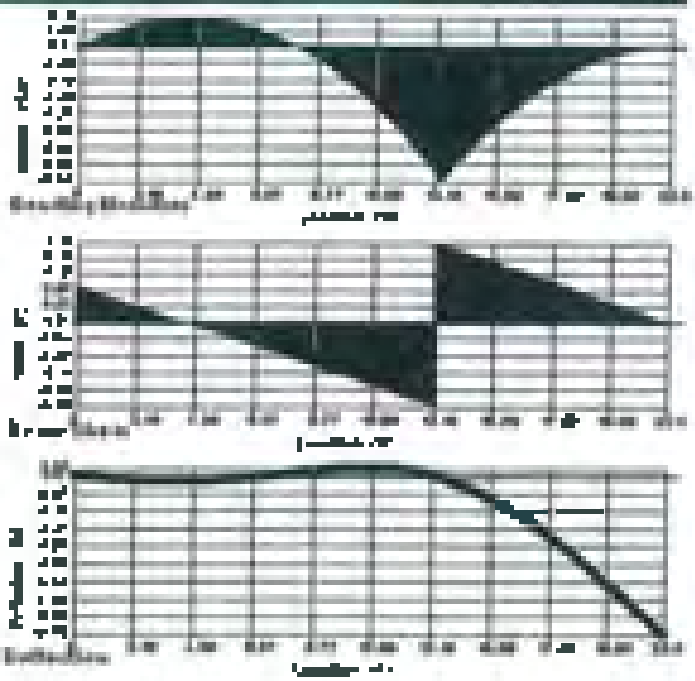
[redacted]

Single Span Beam Analysis

Description: Hor Bearing Seat Wall - W3

Block & Diagram





Dreiker/Perich/Sabatini

Project Name: NWFD Station #53
Project Number: 07502-01
Subject: Stud Wall Design
By: AJB Date: 05/09/07
Checked: Date:
Scale: NWT

Stud Wall - W3

- Non Bearing Studs

$P_{WIND} = 19.8$ psf ≤ 15 psf

$P_{DL} = (3\text{ FT})(20\text{ PLF})(15\%_{12}\text{ FT}) = 80\#$

$P_{LIVE} = 80\#$

$P_{TOTAL} = 160$ lbs

H: 22'-0"

Limiting Height = 19'-7" < 22'-0" ∴ N.G.

∴ Double Studs 6" below Roof Line

- Check Parapet & Roof Line:

$w = 15\text{ psf}\left(16''/12\%\right) + 20\text{ plf}\left(1.67\%\right)$

$M = \frac{20\text{ plf}(8'-8")^2}{7} = 332$ lb-ft

$= 9.02$ k-in

$M_a = 16.68\frac{\text{k-in}}{\text{stud}} \times 2$ studs

$M_a = 33.36$ k-in > 9.02 ... OK

- Check Deflection:

$\Delta = \frac{wL^4}{8EI} = \frac{(1.67\%)(104\text{ in})^4}{8(6.2\text{ k/in}^2 \cdot 10^3\text{ in}^4)} = 0.32$ in $< \frac{l}{289}$ ... OK

2 stud

Reiter/Pereira/Sakalaki

[redacted]

Name: PLVFC Station #53
Job No.: 37502.01
Subject: Stud Wall Design
By: AJB Date: 05/09/07
Check: WL

Stud Wall - W4

- Non Bearing Studs

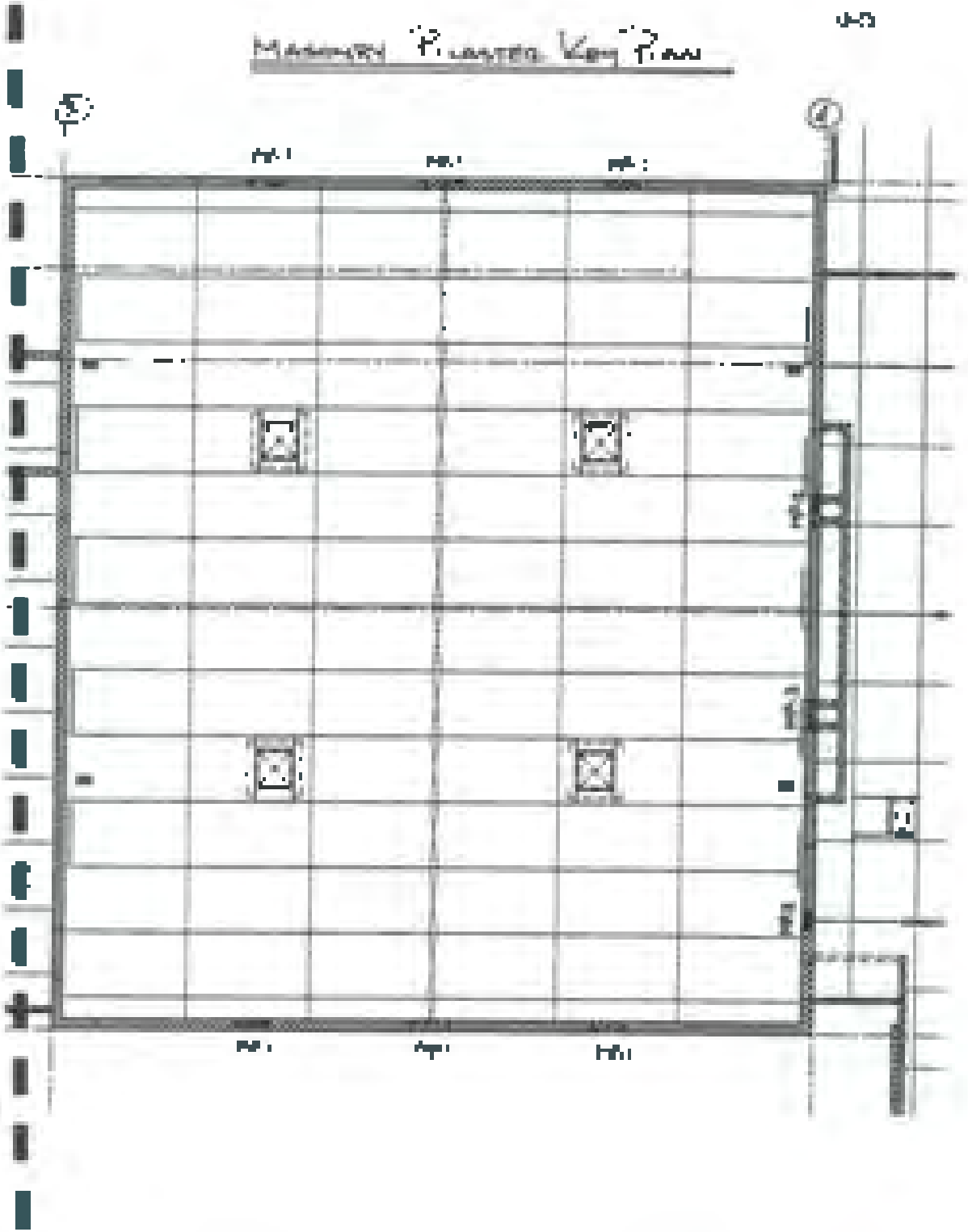
P_{allow} = 14.5 ksf ± (5 psf

P = 160 lbs (same as W3)

H = 19'-0"

Laboring Height = 19'-3" > H ∴ OK

Use 600S162-43 @ 16" oc



MASONRY PILASTERS

MP-1 (APPARATUS BAY DOORS)

P_{DL} = 1330 plf × 13.33 ft = 17,729 lbs

P_{LL} = 600 plf × 13.33 ft = 8000 lbs

* LOADS FROM HEADER DESIGN

(SEE ENGINEERING FOR DESIGN)

USE 32" × (2-8) UNIT MASONRY PILASTER w/ #5 CENTERED IN EA. CELL (FULLY GROUTED)

MP-2

P_{DL} = 1,800 lbs
P_{LL} = 1,350 lbs
} FROM 24 REACTIONS

(SEE ENGINEERING FOR DESIGN)

USE 16" × (1-8) UNIT MASONRY PILASTER w/ #5 CENTERED IN EA CELL (FULLY GROUTED)

Dekker/Perich/Sabatini

Name: FLAGPOLE STATION #3
Name: 02501.01
Name: MASONRY PLANTERS
By: AJB Date: 05/30/07
Name: _____ Date: _____
Name: 1104

MP-3 (HOSE ALCOVE)

- REACTIONS FROM ML-3:

$P_{DL} = \frac{2,342 \text{ plf} \times 16 \text{ ft}}{2} = 18,736 \text{ lbs}$

$P_{LL} = \frac{200 \text{ psf} \times 16 \text{ ft}}{2} = 5,600 \text{ lbs}$

- REACTIONS FROM ML-4:

$P_{DL} = \frac{1,200 \text{ plf} \times 6 \text{ ft}}{2} = 3,600 \text{ lbs}$

$P_{LL} = \frac{700 \text{ plf} \times 6 \text{ ft}}{2} = 2,100 \text{ lbs}$

APPLIED LOADS TO PLANTER:

$P_{DL} = 18,736 + 3,600 = 22,336 \text{ lbs}$

$P_{LL} = 5,600 + 2,100 = 7,700 \text{ lbs}$

(SEE ENGINEERING DESIGN)

USE 24 in (2'-0") wide MASONRY PLANTER w/ #5 CENTERED IN EA. CELL FULLY GROUTED)



Structural Engineering Consultant
[redacted]
Los Angeles, [redacted]
[redacted]
Fax: [redacted]

Title: [redacted]
Project: [redacted]
Client: [redacted]

Job #: [redacted]
Date: [redacted]
Designer: [redacted]

Masonry Flush Wall Pilaster/Column

Page: [redacted]

Description: MP-1

General Information

Wall Height	[redacted]	Pilaster Type	[redacted]	Fm	[redacted] psi
Pilaster Spacing	[redacted]	Pilaster Flange Width	[redacted]	Fy	[redacted] ksi
Pilaster Width	[redacted]	Pilaster Depth	[redacted]	Es / Em	[redacted]
Pilaster Reinforcing	[redacted]			Special Inspection	
Pilaster Location	[redacted]	Concrete weight used	[redacted]	Seismic Detailing	[redacted]
Masonry Density	[redacted]	Load Combination Factors	[redacted]		
Seismic Zone	[redacted]	Wind Exposure	[redacted]		

Applied Loads

Load Type		Girder Load	
Dead Load	[redacted]	Dead Load	[redacted]
Live Load	[redacted]	Live Load	[redacted]
Eccentricity	[redacted]	Eccentricity	[redacted]
Load Type	Axial	Load Type	Axial
Wind Load	[redacted]		

Design Values

Allow Brg. Stress	[redacted]	Pilaster Area			[redacted]
Allow Axial Stress	[redacted]	Pilaster Depth			[redacted]
Reduction Factor	[redacted]	Area Moment			[redacted]
Allow Moment Bending	[redacted]	Sy	[redacted]	[redacted]	[redacted]
Eqn. Axial Load Capacity Interaction	[redacted]	Modulus of Elasticity			[redacted]
Max. Axial Load with Moment	[redacted]	Moment of Inertia			[redacted]

Load Combinations & Stress Ratios

	Moment	Axial			
Governing	Mx	Pu	Mu	Moment Stress	Axial Stress
Pu + Mu	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Max. Axial + Eqv. Moment					
Dc + Lr + Wind	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Dc + Lr + Seismic	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
Maximum Bearing Area	[redacted]		Axial Bearing Length		[redacted]

Summary

Pilaster Design OK

[redacted] Wall height, [redacted] thick wall, [redacted] grout, [redacted] bar Spacing, Special Inspection, Good Structure

Max. Bending + Axial Comp. Stress	[redacted]	psi
Allowable	[redacted]	OK
Max. Axial Only Compressive Stress	[redacted]	psi
Allowable	[redacted]	OK
Max. Axial Bending Stress	[redacted]	psi
Allowable	[redacted]	OK

Summary of Loads & Moments

Girder + Uniform Dead Loads	[redacted]	lbs		@ Base	@ Mid-Ht
Design + Uniform Live Loads	[redacted]	lbs	Design Ec Increase	[redacted]	[redacted]
Axial Applied @ Top Height	[redacted]	lbs	Design Lr Increase	[redacted]	[redacted]
Pu Applied @ Mid-Height	[redacted]	lbs	Total Moment @ Base		[redacted]
Lr Applied @ Mid-Height	[redacted]	lbs	Applied Moment @ Mid-Ht		[redacted]

Structure Point Software
Suite Burnside, Suite 104
Los Angeles, NV 89119
Tel: 702-430-2000
Fax: 702-430-1999

Masonry Flush Wall Pilaster/Column

Description MP-2

General Information

Wall Height	12.50 ft	Pilaster Type	Nominal (N)	No	1
Pilaster Width	16 in	Spacing Pilaster Width	8.000 ft	At	24.000 in
Pilaster Spacing	8 ft	Column Width	16.000 in	Cm = Cm =	0.600
Pilaster Spacing	8.000 ft		Assume 1 in Pilaster	Special Inspection	
Pilaster Location	Center	Column Width Limit	16.000 in	Solid Grouted	
					1,000,000.0 psi
Masonry Type	Solid	Load Duration Factor	1.333	f'	2.50 ksi
Pilaster Zone	2	Wall or Pilaster	1.00		

Applied Loads

Uniform Loads		Axial Loads	
Dead Load	0.00 psf	Dead Load	1,450.00 lb
Live Load	0.00 psf	Live Load	1,200.00 lb
Eccentricity	0.000 in	Eccentricity	0.000 in
Load Type	Axial	Load Type	Axial
Wind Load	0.00 psf		

Design Values

Allow Brg. Stress	570.00 psi	Pilaster Area			0.889 sq.ft
Allow Axial Stress	570.00 psi	Pilaster Depth			3.750 in
Reduction Factor	1.000	Wall Weight			175.000 psf
		wp	0.146 pcf	1	2.1875
Allow Masonry Bending	450.00 psi		0.000 pcf	0.00	0.000
Max Allowed Moment w/o Axial Load	3,000.00 in-lb	Radius of Gyration			3.897 in
Max Axial Load w/o Moment	70,000.00 lb	Moment of Inertia			101.953 in4

Load Combinations & Design Results

	Moment	Axial			
Top or Axial	in-lb	lb	Ratio	Interaction	Axial Stress
DL + LL	450.00	2,650.0	1.000 ok	0.211	54.78
Short Load & Top of Wall					
DL + LL + Wind	787.50	2,650.0	1.000 ok	0.211	54.78
DL + LL + Reverse	112.50	2,650.0	1,000.15	0.211	54.78
Required Bearing Area	4.65 in2		Axial Bearing Length		8.94 in

Summary

Pilaster Design OK

16.250 high, 12.50in thick at 85.00 pcf & 0.00in ecc at center, Standard Inspection, Solid Grouted

Max Bending + Axial Comp Stress	117.57	psi
Allowable	666.35	OK
Max Axial Only Compressive Stress	54.78	psi
Allowable	570.00	OK
Max Shear Bearing Stress	3,000.73	psi
Allowable	31,920.00	OK

Summary of Loads & Reactions

Center = Uniform Dead Loads	1,450.00 lb		@ Top	@ Middle
Center = Uniform Live Loads	1,200.00 lb	Center of Moments	0.00 ft	8.125 ft
Total Weight of Wall	2,650.00 lb	Center of Stiffness	0.00 ft	8.125 ft
Top Eccentricity of Load	0.00 in	Total Moments @ Level		0.00 ft-k
Bot Eccentricity of Load	0.00 in	Relative Stiffness @ Level		1.000

[redacted]

[redacted]

Masonry Flush Wall Pilaster/Column

[redacted]

Design Values

Load Combinations & Stress Details

Summary

Summary of Loads & Moments

ES REPORT™

ER-4943P

Reissued September 1, 2007

ICC-ES Evaluation Services, Inc. • 5360 Workman Mill Road, Whittier, California 90601 • www.icc-es.org

Filing Category: DRYWALL—Steel

BEARING AND NONBEARING STEEL STUDS

STEEL STUD MANUFACTURERS ASSOCIATION
[illegible]

ALLIED STUDCO
[illegible]

CEMCO
[illegible]

CONSOLIDATED FASTENER CORP.
[illegible]

CONSOLIDATED INTERNATIONAL INC.
[illegible]

DAVIDSON INDUSTRIES
[illegible]

DIETRICH INDUSTRIES, INC.
[illegible]

MARCHAN PAR-A-TEEL SYSTEMS
[illegible]

MARINO STEEL FRAMING SYSTEMS
[illegible]

OLIVE HUR BUILDING MATERIALS INC.
[illegible]

SCAFCO CORPORATION
[illegible]

STEEL DEVELOPMENT LLC
[illegible]

UNIMAST INCORPORATED
[illegible]

UNITED METAL PRODUCTS, INC.
[illegible]

WESTERN METAL LATH AND STEEL FRAMING SYSTEMS
[illegible]

1.0 SUBJECT

Bearing and nonbearing steel studs, steel tracks and steel deflection tracks

2.0 DESCRIPTION

The design of the steel studs, tracks and steel deflection tracks in this report complies with Section 2211 of the 1997 Uniform Building Code™ (UBC); Section 2209 of the 2000 International Building Code® (IBC); Section 2210.1 of the 2003 International Building Code® (IBC); and Section 2210.1 of the 2006 International Building Code®.

2.1 General:

Punched, plain sheet and embossed steel studs and tracks are to be manufactured in accordance with the steel thicknesses indicated in Table 1.

Section properties, stud and track properties, steel yield strength, etc., are set forth on pages 8 through 11 and pages 3 through 6 of the accompanying Steel Stud Manufacturers Association catalogs, "Product Technical Information," dated [illegible], and "Supplemental Product Technical Information," dated May 2007. Web punching details and deviations with clipping limits are noted on pages 3A through 3H, and pages 4 and 5, of the accompanying catalogs.

2.2 Nonbearing Wall Heights:

Allowable nonbearing wall heights for interior composite wall design are noted on pages 12 and 13 of the accompanying catalog dated [illegible]. Allowable interior nonbearing wall heights, using non-standard properties of the steel, are noted on pages 14 and 15 of the catalog. Allowable nonbearing wall heights for curtain wall design are noted on pages 16 through 19 of the accompanying catalog.

2.3 Load-Bearing Wall Studs:

Allowable stud sizes, combined with gauge, spacing for various heights and stud spacings, based on maximum bending stress of [illegible] psi, or shear stress of [illegible] psi, or deflection of [illegible] inch, whichever governs, for both axial load bearing capability, with the design

[illegible]

PET.APP.001294

[illegible] with Section C4 in the "Specification for the Design of Cold-formed Steel Structural Members." [illegible] addition, are noted on pages 25 through 30 of the accompanying catalog dated "05."

2.4 Joists

Allowable joist spans for various loads, joist spacings and span conditions are noted on pages 31 through 38 of the accompanying catalog dated [illegible].

2.5 Header Load Tables

Allowable spans for header combinations are noted on pages 39 and 40 of the accompanying catalog dated "05."

2.6 Ceiling Joists

Allowable ceiling joist spans for 4 psf, 5 psf and 10 psf (191 kg/m², 239 kg/m² and 430 kg/m²) load ceiling, L/240 and L/360 deflections, various spacings and span conditions are noted on pages 43 and 44 of the accompanying catalog dated "05."

2.7 Materials

2.7.1 Steel Members: Studs, joists and tracks are formed from sheet having a minimum 50,000 psi (350 MPa) yield point [ASTM A653 SS Grade 33 or ASTM A1003 / A1003M Grade 33] for thicknesses of 0.043 inch (54 mils) [0.054 inch through 0.118 inch (118 mils)] (1.09 mm; [1.37 mm through 3.00 mm]) and a minimum 50,000 psi (345 MPa) yield point [ASTM A 653 SS Grade 50 Class 1 or 2, or ASTM A 1003 SS Grade 50] for thicknesses of 0.033 inch (54 mils)[1.346 mm; through 0.118 inch (118 mils)] (3 mm)

2.7.2 Bypass Bracket: Bypass bracket conforms to ASTM C 36, with thicknesses of [illegible] inch (1.27 mm) and [illegible] inch (1.9 mm).

2.7.3 Steel Screws: [illegible] conforming to ASTM C 1002

2.8 Fasteners and Details:

The catalogs entitled "Product Technical Information," dated [illegible] and "Supplemental Product Technical Information," dated May 2001, which are a part of this report, are available on the ICBO ES web site and on the ICBO ES publication report CD-ROM, under File #3643. Only the items described in Table 3 for the catalog dated [illegible] are part of this report. The entire catalog dated May 2001 is part of this report.

Structural properties for the adoption resistance of walls are noted on pages 4 through 11 of the catalog dated [illegible] and pages 3 through 5 of the catalog dated May 2001, and Section Properties for deflection determinations are the allowable moment lbs.

Allowable axial loads noted on pages 14 through 19 of the catalog dated [illegible] are based on the following considerations:

- The tabulated portion heights are based on flexure, deflection and shear interaction governed.
- Allowable axial loads of Table Heights and the criterion for web crippling allowable compression loads that condition is [illegible] specified in the [illegible] table [illegible].
- Heights are based on web punchouts of a minimum distance of [illegible] from the edge of bearing and the other web holes.

Pages 41 and 42 of the catalog dated [illegible] are not recognized as part of this report.

2.9 Identification

Each stud is identified with the manufacturer's name, steel strength [illegible] mils [illegible] stamp. Minimum identification requirements for the ICBO ES report number embossment or stamped on the web of each section at a maximum of [illegible] inch [illegible] o.c. spacing.

2.10 Evidence Submitted

Documentation, test and load records, engineering calculations, report and [illegible] submitted to the ICBO ES Accredited Corrosion Test Laboratories and Engineering [illegible], dated January 2001.

3.0 Findings

That the Steel Stud Manufacturers Association Steel Framing and Maintaining Steel Studs, Joists and Tracks described in this report comply with the 1997 Uniform Building Code™, the 2000 International Building Code®, the BOCA National Building Code® 1999 and the 1999 Standard Building Code, subject to the following conditions:

4.1 Studs, joists and tracks are installed in accordance with this report and the manufacturer's instructions.

4.2 Allowable heights and buildings comply with the values tabulated.

4.3 Unbalanced conditions must determine all load bearing members are designed by the [illegible] shall be at least 75 percent of the [illegible] weight is applied.

4.4 Calculations, plans and specifications verifying compliance with this report must be submitted to the building official for each project.

4.5 The studs are manufactured at the facilities listed in Table 1.

4.6 Steel material and specifications, including [illegible] heights and loads, are noted in the accompanying tables. Studs are identified with the manufacturer's name and the accompanying catalog.

This report is subject to re-examination in two years.

[redacted]

[redacted]

[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]

[redacted]

[redacted]

[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
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[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]

ES REPORT™ SUPPLEMENT 1

Reissued September 1, 2004

ICBO Evaluation Service, Inc. • 5360 Workman Mill Road, Whittier, California 90601 • www.icbo.org

Filing Category: 08 04 00—Metal

BEARING AND NONBEARING SPRING STEEL JOISTS AND TRUSSES

STEEL JOIST MANUFACTURERS ASSOCIATION
3 LAUREL MEADOW DRIVE, SUITE 108
CHICAGO, ILLINOIS 60523

1.0 SUBJECT

Bearing and Nonbearing Steel Chord, Joists and Joists Related Details – 2003

Add the following manufacturing locations:

Custom Steel, Inc.
5178 Highview Avenue
Jacksonville, Minnesota 56304

Denver, Inc.
4500 South Lucas Way Drive Street
Seattle, Washington 98178

2.0 LIMITATIONS

No change

3.0 EVIDENCE SUBMITTED

No change

4.0 FINDINGS

That the changes noted herein comply with the 1997 Uniform Building Code™, the 2003 International Building Code®, and the 2003 International Residential Code®, subject to the following conditions:

4.1 through 4.3: No change.

4.4 Manufacturing Locations: The steel joist manufacturers at the facilities noted in Table 2.

4.5 Fire Ratings.

Unless specifically noted in this evaluation report supplement, the original report remains valid and unchanged.

This report supplement is issued solely to the product(s) noted herein and is subject to the terms and conditions set forth in the original report. This report supplement must be presented in its entirety when any portion is made available to a third party.

Add the following manufacturing facilities to Table 2:

Custom Steel Manufacturing, Inc.	5178 Highview Ave., Jacksonville, Minnesota
Denver, Inc.	4500 South Lucas Railroad, Colorado
Denver, Inc.	10000 Lucas Lumber King Drive Street, Seattle, Washington

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STRUCTURAL CALCULATIONS

City Of North Las Vegas
Fire Station 53
North Las Vegas, NV

Lateral

[redacted]

Project: NLVFD Station #5
Job No.: 09502-01
Subject: LATERAL
By: AJB Date: 03/16/09
Checked: Date:
Sheet: 11

SEISMIC COEFFICIENT DETERMINATION

- 2006 IBC & ASCE 7-05
  EQUIVALENT LATERAL FORCE PROCEDURE (9.5 $)
- VALUES OBTAINED FROM THE USGS SEISMIC HAZARD CURVES, RESPONSE PARAMETERS AND DESIGN PARAMETERS SOFTWARE v5.0.5 (SEE ATTACHED DATA SHEETS)
- SITE CLASS D — ZIP CODE = 89032

$S_S = 75.4\%$
$S_1 = 24.7\%$
$F_a = 1.355$
$F_v = 2.101$

$S_{MS} = F_a S_s = 1.355 \cdot 0.754 = 1.02$ (Eq. 9.4.1.2.4-1)

$S_{M1} = F_v S_1 = 2.101 \cdot 0.364 = 0.771$ (Eq. 9.4.1.2.4-2)

$S_{DS} = \frac{2}{3} S_{MS} = 0.681$ (Eq. 9.4.1.2.5-1)

$S_{D1} = \frac{2}{3} S_{M1} = 0.514$ (Eq. 9.4.1.2.5-2)

OCCUPANCY CATEGORY = IV (1604.5)

IMPORTANCE FACTOR, $I_E = 1.5$ (TABLE 11.5-1)

SEISMIC DESIGN CATEGORY = D (TABLE 11.6-1)

[redacted]

SEISMIC SHEAR:

$$V = \frac{S_{D1} I_E W}{R} \quad \text{(Eq 12.8-2)}$$

$$V_{max} = \frac{S_{D1} I_E W}{T R} \quad \text{for } T \le T_L \quad \text{(Eq 12.8-3)}$$

$$V_{max} = \frac{S_{D1} T_L I_E W}{T^2 R} \quad \text{for } T > T_L \quad \text{(Eq 12.8-4)}$$

$$V_{min} = 0.01 W \quad \text{(Eq 12.8-5)}$$

* For $S_1 \ge 0.6g$

$$V_{min} = \frac{0.5 S_1 I_E W}{R} \quad \text{(Eq 12.8-6)}$$

$$T = T_a = C_t h_n^{x} \quad \text{(12.8.2, Eq 12.8-7)}$$

$T_L = 8$ (11.4.5, FIGURE 22-15)

$h_n = 22.5$ FT

$C_t = 0.02$ (ALL OTHER BUILDINGS) } TABLE 12.8-2

$x = 0.75$

$R = 5$ (SPECIAL REINFORCED CMU) (TABLE 12.2-1)

Parker Parish Substation

[redacted]

NWFD STATION #53
07502-01
LATERAL
By A-JB Date 03/26/07
Checked By
Page 43

$T_a = 0.02 (23.5)^{0.75} = 0.107$

$T < T_L$ ∴ USE EQ. 12.8-3

$V = \frac{S_{DS} I_e W}{R} = \frac{0.484 (1.5) W}{5} = \frac{0.201 W}{5}$ controls

$V_{max} = \frac{S_{DS} I_e W}{T R} = \frac{(0.594)(1.5)}{(0.207)(5)} W = 0.795 W$

$V_{min} = \frac{0.5 S_1 I_e W}{R} = \frac{0.5 (0.367)(1.5) W}{5} = 0.055 W$

Out-of-Plane Seismic Forces:

$F_p = 0.40 S_{DS} I W_p \geq 0.1 W_p$ (ASCE 13.11.1)

$= (0.40)(0.681)(1.5) W$

$F_p = 0.409 W$

Bekker/Perich/Kabaliuk

[redacted]

Name: NAVFAC STATION #53
Job No: 07502-01
Subject: LATERAL
By: AJB Date: 05/30/07
Checked: Date:
Sheet: 4

SEISMIC LOAD DISTRIBUTION

ROOF AREAS:

AREA 1 = 8,022 SF
AREA 2 = 5,168 SF
AREA 3 = 1,107 SF

TOTAL AREA = 14,297 SF

WALL AREAS:

AREA 1 = 338 FT · (7'-9" + 7'-6") = 5,070 SF

AREA 2 = 296 FT · (7'-9" + 5'-0") = 4,810 SF

AREA 3 = 103 FT · (7'-9" + 3'-6") = 1,082 SF

SEISMIC WEIGHT:

W_R = 14,297 SF × 20 PSF = 285,940 #

WALL WEIGHT:

W_W = (5,070 SF + 1,082 SF)(15 PSF)
+ (4,810 SF)(99 PSF)

W_W = 544,420 #

TOTAL WEIGHT:

W_T = 285,940 + 544,420 = 830,360 lbs

Baker/Parrish/Kabakjian

[redacted]

Job: United States Post
Job No.: 07502.01
Subject: LATERAL
By: AJB Date: 03/30/07
Chkd: Date:
Sht: 65

TOTAL SEISMIC SHEAR:

$V = 0.104\ W$

$= 0.204\ (830k)$

$V = 169\ k$

From IBC 2006 - 1605.3.2

$V = \frac{169k}{1.4} = 121\ k$

SEISMIC SHEAR PER AREA

$W_{A1} = (8,032\ sf \times 20\ psf) + (5,070\ sf \times 15\ psf)$
$= 236,490\ lbs$

$V_{A1} = \frac{0.204\ (236,490\ lbs)}{1.4} = 34,960\ lbs$

$W_{A2} = (5,168\ sf \times 20\ psf) + (4,810\ sf \times 9\ psf)$
$= 555,500\ lbs$

$V_{A2} = \frac{0.204\ (555,500\ lbs)}{1.4} = 80,948\ lbs$

$W_{A3} = (1,107\ sf \times 20\ psf) + (1,052\ sf \times 15\ psf)$
$= 38,390\ lbs$

$V_{A3} = \frac{0.204\ (38,390)}{1.4} = 5,591\ lbs$

Project Name = 07602-01 - MLVFD Station #54
Conterminous 48 States
2006 International Building Code
Zip Code = 88032
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.01 deg grid spacing

Period (sec)	Centroid Sa (g)	
0.2	0.556	Ss, Site Class B
1.0	0.173	S1, Site Class B

Period (sec)	Maximum Sa (g)	
0.2	0.670	Ss, Site Class B
1.0	0.171	S1, Site Class B

Period (sec)	Maximum Sa (g)	
0.2	0.540	Ss, Site Class B
1.0	0.172	S1, Site Class B

Conterminous 48 States
2006 International Building Code
Zip Code = 88032
Spectral Response Accelerations SMs and SM1
SMs = Fa Ss and SM1 = Fv S1
Site Class D - Fa = 1.355 ,Fv = 2.701

Period (sec)	Sa (g)	
0.2	0.754	SMs, Site Class D
1.0	0.357	SM1, Site Class D

Conterminous 48 States
2006 International Building Code
Zip Code = 88032
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.155 ,Fv = 2.101

Period Sa
(sec) (g)
0.2 0.503 50m, Site Class D
1.0 0.245 90t, Site Class D

Conterminous 48 States
2006 International Building Code
Zip Code = 89032
MCE Response Spectra for Site Class B
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0

Period (sec)	Sa (g)	Sa (inches)
0.000	0.273	0.000
0.083	0.557	0.021
0.200	0.557	0.736
0.214	0.557	0.538
0.400	0.437	0.883
0.500	0.350	0.954
0.600	0.293	1.025
0.700	0.250	1.195
0.800	0.219	1.366
0.900	0.194	1.537
1.000	0.175	1.708
1.100	0.159	1.879
1.200	0.146	2.050
1.300	0.134	2.220
1.400	0.125	2.391
1.500	0.117	2.562
1.600	0.109	2.733
1.700	0.103	2.903
1.800	0.097	3.074
1.900	0.092	3.245
2.000	0.087	3.416

Conterminous 48 States
2006 International Building Code
Zip Code = 89032
Site Modified Response Spectra for Site Class Site Class D
Sms = Fa Ss and Sm1 = Fv S1
Site Class D - Fa = 1.355 ,Fv = 2.101

Period (sec)	Sa (g)	Sd (inches)
0.000	0.302	0.000
0.097	0.754	0.070
0.200	0.754	0.295
0.487	0.754	1.747
0.500	0.735	1.794
0.600	0.617	2.153
0.700	0.525	2.502
0.800	0.459	2.871
0.900	0.408	3.250
1.000	0.367	3.668
1.100	0.334	4.041
1.200	0.306	4.508
1.300	0.283	4.905
1.400	0.262	5.024
1.500	0.245	5.383
1.600	0.230	5.748
1.700	0.216	6.100
1.800	0.204	6.459
1.900	0.193	6.818
2.000	0.184	7.177

Bohlin Perich Sabatini

[redacted]

Job: NCATC Standard #53
Job No: 07502.01
Subject: LATERAL
By: ALB Date: 03/30/07
Check: Date:
Sheet: L4

Wind Loads

- P_{ASD} IBC 2006 = 1609.1.1 & ASCE 7-05 Combined

Basic Wind Speed (3 sec Gust) = 90 mph (Fig. 6-1)
Exposure = C (Sec 6.5.6)
Roof Pitch = 0.25 : 12
Mean Roof Height, h = 28 ft

6.4 METHOD 1 - SIMPLIFIED PROCEDURE

6.4.2.1 - MAIN WIND-FORCE RESISTING SYSTEM

$$p_s = \lambda K_{zt} I_w p_{s30} \quad \text{(Eq 6-1)}$$

WHERE,
HEIGHT ADJUSTMENT FACTOR, $\lambda = 1.33$ (Fig. 6-2)
TOPOGRAPHIC FACTOR, $K_{zt} = 1.0$ (6.5.7.2)
IMPORTANCE FACTOR, $I_w = 1.15$ (TABLE 6-1)

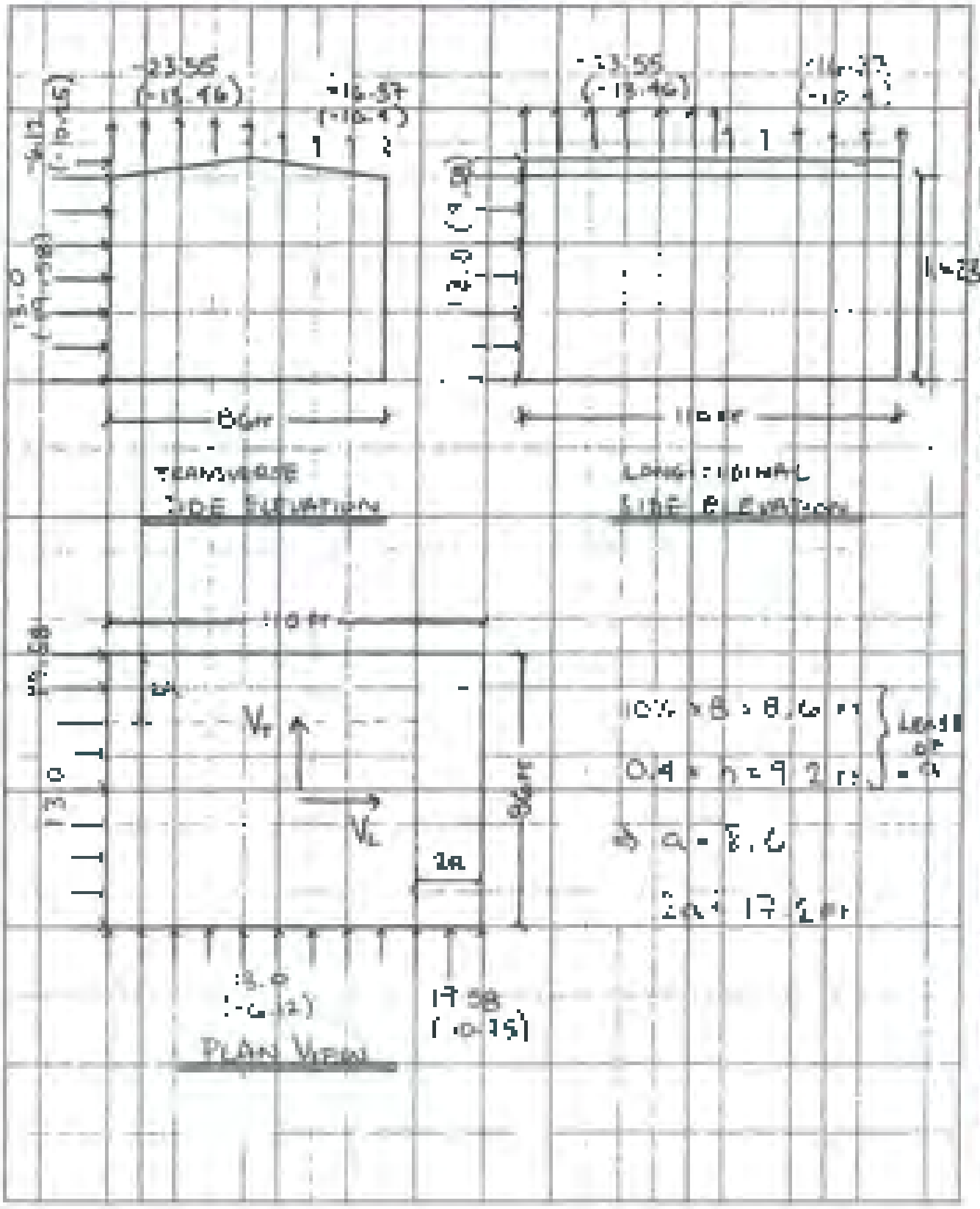
$$p_s = (1.33)(1.0)(1.15)\, p_{s30} = 1.53\, p_{s30}$$

P_{s30} Fig 6-2:

	HORIZ. PRESSURES				VERT PRESSURES				OVERHANG	
	A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
P_{s30}	12.8	-6.7	8.5	-11.0	-15.4	-8.8	0.9	-6.8	-21.4	-16.9
P_s	19.58	-10.25	13.0	-16.83	-23.55	-13.46	1.37	-10.4	-32.74	-25.85

Becker/Parish/Sabatini

NAVFAC Station #53
02502-01
Subject: LATERAL
By: AJB Date: 03/30/07
Checked: Date:
Sheet: 4



Bekker/Perich/Sabatini

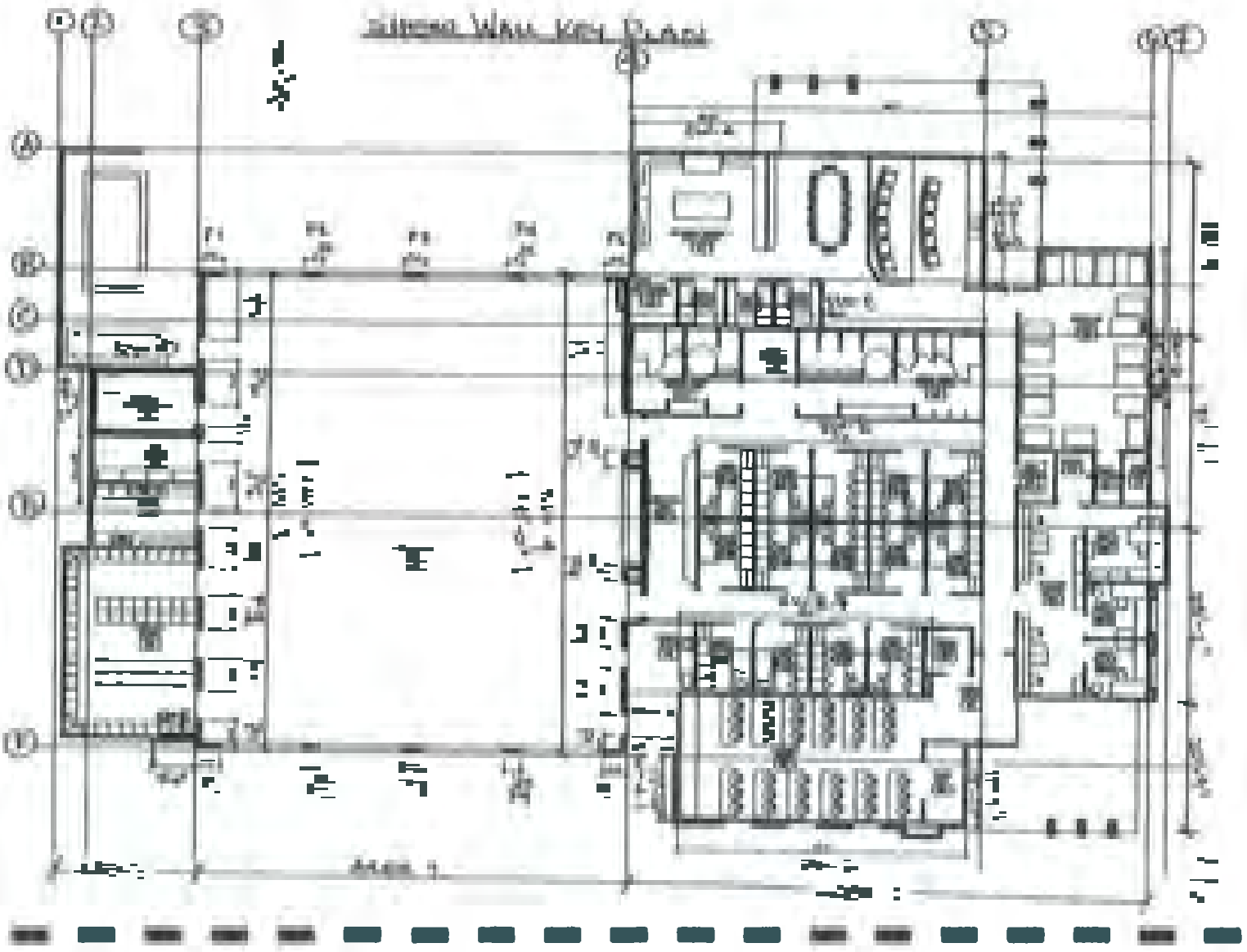
[redacted]

Name: NLVFD STATION #52
Project number: 07502-01
Subject: LATERAL
By: AJB Date: 03/30/07
Checked: Date:
Sheet: 1/1

$V_T = (46.8\text{ FT} \times 23\text{ PSF})\,1.30 + (13.2\text{ FT} \times 23\text{ FT})\,1.58$
$= 28.32\text{ K}$

$V_L = (92.8\text{ FT} \times 23\text{ FT})\,1.30 + (13.2\text{ FT} \times 23\text{ FT})\,1.58$
$= 35.99\text{ K}$

SEISMIC CONTROLS BOTH WAYS (121 K)



Bekker/Perich/Sabatini

[redacted]

Name: NUFD STATION #53
Project Number: 07502-01
Subject: LATERAL
By: AJB Date: 08/03/07
Checked: Date:
Sheet: L15

SHEAR WALL DESIGN - LIGHT GAGE STEEL

- IBC 2006: 2210.5 - LATERAL DESIGN PER AISI-LATERAL

- AISI LATERAL:

- C2 TYPE I SHEAR WALLS
  - SHALL BE FULLY SHEATHED w/ HOLD-DOWN ANCHORS @ EACH END OF WALL SEGMENT.
  - AVAILABLE STRENGTH SHALL BE DETERMINED USING THE SAFETY FACTOR Ω_v (2.5).
  - HEIGHT TO WIDTH ASPECT RATIO (H/W) OF A WALL PIER SHALL BE LIMITED TO A MAXIMUM OF 2:1.
  - WHERE AN ASPECT RATIO (H/W) GREATER THAN 2:1 IS ALLOWED, THE NOMINAL SHEAR STRENGTH SHALL BE MULTIPLIED BY 2 W/H, BUT IN NO CASE SHALL THE ASPECT RATIO EXCEED 4:1.

- C3 TYPE II (PERFORATED) SHEAR WALLS:
  - SACRIFICIAL SEGMENTS SHALL BE LOCATED AT END OF A TYPE II SHEAR WALL.
  - NOMINAL SHEAR STRENGTH SHALL BE BASED UPON EDGE SCREW SPACING NOT LESS THAN 4 INCHES OC
  - TYPE II SHEAR WALLS SHALL NOT HAVE OUT OF PLANE (PERPENDICULAR) OFFSETS.
  - COLLECTORS FOR SHEAR TRANSFER SHALL BE PROVIDED FOR THE FULL LENGTH OF WALL.
  - TYPE II SHEAR WALLS SHALL HAVE UNIFORM TOP OF WALL & BOTTOM OF WALL ELEVATIONS
  - WALL HEIGHT, h, SHALL NOT EXCEED 20 ft

Bekker/Petrick/Sakakini

Name NUVFD STATION #53
Job No. 07502.01
Subject LATERAL
By AHB Date 04/03/07
Chkd By EM

Shear Wall Design - Area 1

East-West Direction

Total Seismic Shear = 39,960 lbs

Roof Area = 8,022 SF

Seismic Load: $\frac{39,960 \text{ lbs}}{8,022 \text{ SF}} = 4.30 \text{ PSF}$

Type I Shear Walls

Shear Wall SW-A

Tributary Area = .4 33'-6 5/8" = 824 SF

Length, L_{SW-A} = 23'-10" (23.83 ft)

Shear, V: $\frac{4.30 \text{ PSF} \cdot 824 \text{ SF}}{23.83 \text{ FT}} = 148.7 \text{ PLF} \approx 149 \text{ PLF}$

A - Sheathing

15/32" APA RATED SHEATHING w/ 8d Screws @ 6" OC Along Panel Edges & 12" OC in Field

Nominal Shear Strength, R_n = 780 PLF

Allowable Strength Value = $\frac{R_n}{\Omega} = \frac{780 \text{ PLF}}{2.0}$

390 PLF > 149 PLF ∴ OK

Nelder/Parish/Schettini

[redacted]

Project: NLWFD Station #53
Project No.: 03507-01
Subject: LATERAL
By: ALB Date: 09/08/07
Check: Date:
Sheet: LS

S-CB2 Wall Sec. A (cont.)

B - UPLIFT @ STEPPED END:

$T = C_s \cdot \frac{V_h}{L_{eff}} = \frac{(4.30\ plf \times 8.24\ ft)(1\ str)}{23.83\ ft}$

$T = 2,230$ lbs

RESISTING DEAD LOAD:

$W_{ROOF} = 20\ psf \times 14.33\ ft \times 23.83\ ft$
$= 6,830$ lbs

$W_{WALL} = 15\ psf \times 15\ ft \times 23.83\ ft$
$= 5,362$ lbs

$W_{TOTAL} = 12,192$ lbs

$UPLIFT = T - 0.9\ \frac{W_{TOTAL}}{2}$

$= 2,230 - 0.9\ (\frac{12,192}{2})$

$= -3,256$ lbs ∴ NO UPLIFT

∴ NO HOLD DOWNS REQ'D

C - ANCHORAGE FOR IN-PLANE SHEAR:

- PER IBC 2006 SECTION 1911
- TABLE 1911.2

Becker/Perich/Sabatini

[redacted]

Project: NLVPD STATION #57
Job No.: 07507-01
By: LATERAL
Date: A-1B 04/02/08
Check: [redacted]
Sheet: 4/4

C. ANCHORAGE (CONT.) C1-A

USE IRY ½" ϕ x 7" ANCHOR BOLTS

- MIN EMBED = 4" < ACTUAL : OK
- MIN EDGE DIST = 3" = ACTUAL : OK

T_{allow} = 950 lbs

V_{allow} = 1,250 lbs

ANCHOR SPACING = 4'-0" oc

V_s = WIND (14° ELEV) = 596 lbs

T_s = 596 lbs

$$\left(\frac{V_s}{V_a}\right)^{5/3} + \left(\frac{T_s}{T_a}\right)^{5/3} \leq 1.0 \quad \text{(EQ. 19-1)}$$

$$\left(\frac{596}{1,250}\right)^{5/3} + \left(\frac{596}{950}\right)^{5/3} = 0.75 \leq 1.0 \quad \text{: OK}$$

USE ½" ϕ x 7" ANCHOR BOLTS @ 4'-0" oc

Dekker/Perich/Sabatini

[redacted]

PLUED STATION #53
09502-01
LATERAL
By AJB Date 04/02/97
Check Date
Sheet 19 of

2 Compression Chord Force:

Ref AISC - C4: Concentrically Loaded Compression Members

T = C = 2,230 kips

Use 2 - 6 x 8 SYP #2 33 Back to Back

(See ASTM/CW Property Data Sheet)

$I_x = 3.586\ in^4$

$A_c = 0.689\ in^2$

$r = 2.282\ in$

$P_n = A_c F_n$ (Eq. C4-1)

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (1.9 \times 10^6\ psi)}{\left(\frac{0.84 \times 180\ in}{2.282\ in}\right)^2}$ (Eq. C4.1-1)

$F_e = 46,003\ psi$

$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{33\ ksi}{46\ ksi}} = 0.85 < 1.5$

$\Rightarrow F_n = \left(0.658^{\lambda_c^2}\right) F_y$ (Eq. C4-2)

$= \left(0.658^{(0.85)^2}\right) 33,000\ psi$

$F_n = 24,388\ psi$

- 12. (CONT.)

$P_A = 0.69\ in^2 \times (24,350\ psi) = 6,803\ lbs$

$P_a = \frac{P_A}{\Omega} = \frac{16,703\ lbs}{1.8} = 4,335\ lbs$

$\Omega T = 2.5 (2,732\ lbs) = 5,575\ lbs < P_a$: OK

USE 6005162-33 BACK-TO-BACK STUDS @ END OF SW-A

3. [illegible] gypsum board is applied to one exposed side of wall (see Table G2.1-2) with screw spacing at 7 inches (178 mm) at edges and 12 inches (305 mm) in the field, shear wall nominal strengths are permitted to be increased by 50%.
4. See Section G2.1 for requirements for sheathing applied to both sides of wall.
5. Shear wall height-to-width ratios shall not exceed those given in Table 4.1, but shall not exceed 2:1 for wall with full height sheathing on one side and 1:1 for all other cases. Nominal shear strength shall be multiplied by 2b/h. See Section G2.1.
6. Shear values permitted for use in seismic design where the seismic response modification factor, R, is taken equal to or less than 5. Applies to shear transfers in Section G1.1.
7. For SI: 1 inch = 25.4 mm, 1 pound = 0.305 m, 1 lb = 4.45 N.

TABLE G2.1-2
NOMINAL SHEAR STRENGTH, (R_n), FOR WIND AND SEISMIC LOADS
FOR SHEAR WALLS FACED WITH GYPSUM BOARD[1,2,3,4]
(Pounds Per Foot)

Wall Construction	Min. Aspect Ratio (h/w)	Description	Screw Spacing (in.)		Nominal Shear Strength (plf)
			Edge	Field	
½" gypsum board on one side of wall studs max. 24" O/C	2:1	Gypsum board applied perpendicular to framing with wood blocking behind the horizontal joint and with joints staggered between the first row and the second or applied vertically with all edges attached to framing members	2	7	260
			4	4	425
			4	12	305
			8	12	230

1. Nominal shear strengths shall be multiplied by the resistance factor (φ) to determine design strengths or divided by the safety factor (Ω) to determine allowable shear strengths as set forth in Section G2.1.
2. See Section G2.1 for requirements for sheathing applied to both sides of wall.
3. Unblocked assemblies are permitted provided the nominal shear strength values above are reduced by 50%.
4. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.

TABLE G2.1-3
NOMINAL SHEAR STRENGTH, (R_n), FOR SEISMIC LOADS FOR SHEAR WALLS [4,5]
(Pounds Per Foot)

Assembly Description	Min. Aspect Ratio (h/w)	Fastening Spacing at Panel Edges (in.)				Designation/ Thickness of Sheathing	Required Sheathing Blocked
		6	4	3	2		
15/32" Structural 1 sheathing (4-ply), one side	2:1	490	660			33 or 43	B
	1:1	980	1330	1715	2180	43 or 54	B
7/16" OSB, one side	2:1	700	935	-	-	33	B
	2:1	825	1235	1545	2060	43 or 54	B
	1:1	940	1410	1760	2350	54	B
	1:1	1230	1845	2310	3080	68	BD
0.019" metal sheet, one side	1:1	350		-	-	33 (min.)	B
0.027" steel sheet, one side	1:1	-	1350	1685	2170	33 (min.)	B

1. Nominal shear strength shall be multiplied by the resistance factor (φ) to determine design strength or divided by the safety factor (Ω) to determine allowable shear strength as set forth in Section G2.1.
2. Screws at the ends of the panel shall be installed 2" inches (305 mm) from the corners, unless otherwise noted.
3. Shear wall height-to-width aspect ratios (h/w) greater than 2:1, but not exceeding 3:1 are permitted provided the nominal shear strength are multiplied by 2w/h. See Section G2.1.
4. See Section G2.1 for requirements for sheathing applied to both sides of wall.

SECTION 1913
CONCRETE

1913.1 General. Concrete in members of moment-resisting frame primarily designed to resist earthquake forces shall conform to the requirements of this chapter for plain or reinforced concrete.

1913.2 Proportions and materials. Concrete proportions shall be established that allow suitable placement procedures using the delivery equipment selected and shall result in a finished surface free of defects interfering with the strength required by the code.

1913.3 Aggregates. Coarse aggregate of hard, dense rock shall conform to ASTM C 33 except as modified by Section 1913.3.1.

1913.3.1 Lightweight concrete. Lightweight concrete shall be defined as concrete that complies with the provisions of Sections 1913.3.1.1 through 1913.3.1.3.

1913.3.1.1 Slag. The maximum percentage of expanded slag shall be limited to that which is prequalified by previous satisfactory field performance or laboratory trial batches in which the required strength, durability and wearing qualities are demonstrated.

1913.3.1.2 Clarification. Where No. 3 or smaller bars are used, there shall be a minimum clearance between parallel reinforcing bars of 1½ inches (38 mm). Where bars larger than No. 3 are provided, there shall be a minimum clearance between parallel bars equal to the diameter of the bars used. Where four layers of steel are provided, the vertical spacing between layers shall be not less than 6 inches (152 mm). The clear distance between columns shall not be less than 1½ times the cross-sectional dimension of the column or 1½ times the maximum size of the coarse aggregate, whichever is greater.

Exception: Subject to the approval of the building official, required clearances shall be reduced where it is demonstrated by procedures or tests that adequate placement of the bars and the design will be achieved.

1913.3.1.3 Splices. Lap splices of reinforcing bars shall not be less than the tension lap splice required with a minimum clearance of 3 inches (51 mm) between bars. The use of contact lap splices is permitted only for support of the reinforcing at a point of inflection approved by the building official, based on satisfactory preconstruction tests that show that adequate consolidation of the bars will be achieved, and provided that the splice is confined to the middle portion of the member and is prohibited in the vicinity of the beam-column joint.

1913.3.1.4 Splices of welded wire fabric. Splices of welded wire fabric shall be made so that adjacent sheets will maintain a uniform distance apart and the reinforcing will be effective between the splices.

1913.3.2 Pre-construction tests. Where required by the building official, a test panel shall be shot, cured, cored, or sawed and tested prior to commencement of the project. The sample panel shall be representative of the proposed and in-place job conditions as closely as possible. The panel shall remain and reinforcing shall not be less than the minimum specified by the structural design. It shall be shot at the same angle using the same nozzleman and with the same equipment and mixture that will be used on the project. The aggregate used in preconstruction testing shall be the same as that proposed for the work, including the same admixtures as approved by the building official.

1913.3.3 Rebound. Any rebound or accumulated loose aggregate shall be removed from the surface to be covered prior to placing the initial or any succeeding layers of concrete. Rebound shall not be used as aggregate.

1913.4 Joints. Except where prohibited herein, shotcrete work shall be stopped or started at edges of openings. Where shotcrete edges are stopped in a thin edge, the edge shall be tapered to a feather edge. Joints between successive days' work or between shotcrete and other concrete shall be made only at locations shown on the approved construction documents, unless joints are permitted. Before placing additional material, laitance or

Table 1913
MAXIMUM ALLOWABLE STRESSES FOR PRESTRESSED CONCRETE (psi)

Stress Classification	Maximum Compressive Stress	Maximum Tensile Stress	Symbols	Maximum Concentric Prestress (psi)					
				f_{ci} = [illegible]		f_{ci} = [illegible]		f_{ci} = [illegible]	
				Precast	Cast-in-place	Precast	Cast-in-place	Precast	Cast-in-place
S_1	$0.6f_{ci}$	$3\sqrt{f_{ci}}$	C	300	150	300	300	300	300
S_2	$0.6f_{ci}$	$6\sqrt{f_{ci}}$	C	300	1,000	300	1,000	300	1,000
S_3	$0.6f_{ci}$	$12\sqrt{f_{ci}}$	C S	300 1,000	1,000 1,000	300 1,000	1,000 1,000	300 1,000	1,000 1,000
S_4	$0.6f_{ci}$ $0.6f_{ci}$	$6\sqrt{f_{ci}}$ $6\sqrt{f_{ci}}$	C S	1,000 1,000	1,000 1,000	1,000 1,000	1,000 1,000	1,000 1,000	1,000 1,000
S_5	$0.6f_{ci}$ $0.6f_{ci}$	$6\sqrt{f_{ci}}$ $6\sqrt{f_{ci}}$	C S	1,000 1,000	1,000 1,000	1,000 1,000	1,000 1,000	1,000 1,000	1,000 1,000
S_6	$0.6f_{ci}$	$3\sqrt{f_{ci}}$	C	300	1,000	300	1,000	300	1,000
S_7	$0.6f_{ci}$	$6\sqrt{f_{ci}}$	C	300	1,000	300	1,000	300	1,000
S_8	$0.6f_{ci}$	$6\sqrt{f_{ci}}$	C	1,000	1,000	1,000	1,000	1,000	1,000

For SI: 1 inch = 25.4 mm, 1 pound per square inch = 6.895 kPa, °C = [illegible]

PET.APP.001329

www.clarkwestern.com

2001 North American Specification w/2004 Supplement ASD
04/16, 400/54

[2] 6"DTF20 (1.5in)(54ksi) FOR: 600S162-54 [2] Back-to-Back

INPUT PROPERTIES:

Web Height =	6.000	in	Steel Thickness =	0.0346	in
Top Flange =	1.625	in	Inside Corner Radius =	0.0795	in
Bottom Flange =	1.625	in	Yield Stress, Fy =	33.0	ksi
Stiffening Lip =	0.500	in	Fy With Cold-Work, Fya =	33.0	ksi

OUTPUT PROPERTIES:

Effective Section Properties, Strong Axis

Inboard Axis from Top Fiber (Yeg)	3.0347	in
Moment of Inertia for Deflection (Ied)	3.3896	in^4
Section Modulus (Sxc)	1.1348	in^3
Allowable Bending Moment (Ma)	1901.26	ft-Lb

Gross Section Properties at Full Section, Strong Axis

Inboard Axis from Top Fiber (Yeg)	3.0000	in
Moment of Inertia (Ixx)	3.6858	in^4
Gross Sectional Area (A)	0.6998	in^2
Radius of Gyration (Rx)	2.2919	in

Section Properties, Weak Axis

Gross Moment of Inertia (Iyy)	0.3458	in^4
Radius of Gyration (Ry)	0.7153	in

Other Section Property Data

Member Weight per Foot of Length	2.3431	lbf
Maximum Shear Force in Web (Unpunched)	1278	lb
Fyc for use in Interaction Equation C5-7	7266	lb

Lateral Buckling Properties for Flexure

Moment of Inertia of Compression Portion (yc)	0.1458	in^4

Allowable Web Crippling Loads (lb) Total for [2] Members

	1.00in Brg (Interior)		2.00in Brg (End)	
	Cond 1	Cond 3	Cond 2	Cond 4
[2] Back-to-Back	774	386	1075	538

Bekker/Parjeh Consultants

[redacted]

Project: WILLARD Structural
Job No.: 03502.01
Subject: LATERAL
By: AJB Date: 04/03/07
Check: Sheet:
Ref: L22

SHEAR WALL SW-C

TRIBUTARY AREA = 2442 SF

LENGTH, $L_{SW.C}$ = 58'-0"

SHEAR, V = 1.30 PSF × 2,442 SF = 10,501 lbs

UNIT SHEAR, $v = \frac{V}{L_{SW.C}} = \frac{10,501 \text{ lbs}}{58 \text{ FT}} = 181 \text{ PLF}$

A - SHEATHING:

15/32" APA RATED SHEATHING w/ #8 SCREWS @ 6" O.C. ALONG PANEL EDGES AND AT 12" O.C. IN FIELD

$v_{ALLOW} = \frac{R_A}{L} = \frac{780}{2.5} = 312 \text{ PLF} > 181 \text{ PLF}$ OK

B - UPLIFT @ SHEAR WALL ENDS:

$T = C = \frac{Vh}{L_{SW}} = \frac{(10,501 \text{ lbs})(15 \text{ FT})}{58 \text{ FT}} = 2,715 \text{ lbs}$

RESISTING DEAD LOAD:

W_{ROOF} : 20 PSF × 29.875 FT × 58 FT = 34,655 lbs

W_{WALL} = 10 PSF × 15 FT × 58 FT = 8,700 lbs

W_{TOTAL} = 34,655 + 8700 = 43,355 lbs

Dekker/Perich/Sabatini

[redacted]

NWTC STATION #53
07502-0
LATERAL
AAB 04/03/07
L23

B - UPLIFT (CONT.) SW-C

$$\text{UPLIFT} = T - 0.9\frac{W_{TOTAL}}{2} = 2,716 - 0.9\frac{43,355}{2}$$

$$= -16,794 \text{ lbs} \quad \therefore \text{ NO UPLIFT}$$

∴ NO HOLD DOWNS REQ'D

C - ANCHORAGE FOR IN-PLANE SHEAR:

USE 1/2" ϕ x 7" ANCHOR BOLTS

$T_{allow} = 950$ lbs
$V_{allow} = 1,250$ lbs

SPACING @ 4'-0"

$T_d = 4(181 \text{ PLF}) = 724$ lbs
$V_d = 724$ lbs

$$\left(\frac{724}{1250}\right)^{5/3} + \left(\frac{724}{950}\right)^{5/3} = 1.0 \quad \therefore \text{ OK}$$

USE 1/2" ϕ x 7" ANCHOR BOLTS @ 4'-0" OC.

D - COMPRESSION CHORD FORCE

$\Omega T = \Omega C = (2.5)\ 2,716 \text{ lbs} = 6,790$ lbs
(SEE SW-A FOR (2)600S162-33 CALCULATIONS)

$P_a = 9,335$ lbs $>$ 6,790 lbs ∴ OK

USE 2-600S162-33 BACK TO BACK STUDS @ ENDS

PET.APP.001282

Dekker/Perich/Sabatini

[redacted]

NUFD Stations #57
075021-01
LATERAL
A-LE 04/03/07
E-W

SHEAR WALL SW-E

TRIBUTARY AREA = 2547 SF

LENGTH, L_{SW-E} = 47'-0"

SHEAR, V = 4.30 PSF × 2547 SF = 10,952 lbs

UNIT SHEAR, $U = \frac{10,952 \text{ lbs}}{47 \text{ ft}} = 233$ PLF

A - SHEATHING

7/32" APA RATED SHEATHING W/ #8 SCREWS AT 6" OC ALONG PANEL EDGES AND AT 12" OC IN FIELD.

$U_{ALLOW} = \frac{780 \text{ PLF}}{2.5} = 312 \text{ PLF} > 233 \text{ PLF} \therefore$ OK

B - UPLIFT & SHEARWALL ENDS

$T = C = \frac{Vh}{L_{SW}} = \frac{(10,952 \text{ lbs})(15 \text{ ft})}{47 \text{ ft}} = 3,495$ lbs

RESISTING DEAD LOAD

$W_{ROOF} = 20 \text{ PSF} \times 29.5 \text{ ft} \times 47 \text{ ft} = 27,805$ lbs

$W_{WALLS} = 10 \text{ PSF} \times 15 \text{ ft} \times 47 \text{ ft} = 7,050$ lbs

$W_{TOTAL} = 27,805 + 7,050 = 34,855$ lbs

$UPLIFT = 3,495 \text{ lbs} - 0.9\left(\frac{34,855}{2}\right) = -12,190$ lbs

NO UPLIFT

NO HOLD DOWN REQ'D

Dewberry Perlich Saboteini

[redacted]

Project: MW-10 SPACING #53
Job No.: 09502-01
Subject: LATERAL
By: AJB Date: 04/09/07
Chkd: Date:
Sheet: 675

C - ANCHORAGE FOR IN-PLANE SHEAR:

USE 7/8" ϕ x 7½" ANCHOR BOLTS

T_{allow} = 1,500 lbs
V_{allow} = 2,750 lbs

SPACING @ 9'-0" o.c.

T_s = A_T (233 plf) = 932 lbs
V_s = " = 932 lbs

$\left(\frac{932}{2,750}\right)^{5/3} + \left(\frac{932}{1500}\right)^{5/3} = 0.62 > 1.0$ ∴ OK

USE 7/8" ϕ x 7½" ANCHOR BOLTS @ 9'-0" o.c.

D - COMPRESSION CHORD FORCE:

ΩT = 2.5 (3,495 lbs) = 8,736 lbs

(SEE SHEET 8 FOR COMPRESSION CHORD CALCULATIONS)

P_a = 9,335 lbs > 8,736 lbs ∴ OK

USE (2) 600S 162-33 BACK-TO-BACK STUDS

SHEAR WALL SW-E.7

TRIBUTARY AREA = 1,796 SF

LENGTH, $L_{SW.E.7}$ = 41'-4" (41.33 FT)

SHEAR, V = 430 PLF × 1,796 SF = 7,508 lbs

UNIT SHEAR, $v = \frac{7,508 \text{ lbs}}{41.33 \text{ ft}} = 182 \text{ PLF}$

A - SHEATHING:

15/32" APA RATED SHEATHING w/ 8d SCREWS @ 6" o.c. ALONG PANEL EDGES AND @ 12" o.c. IN FIELD.

$v_{allow} = \frac{780}{2.5} = 312 \text{ PLF} > 182 \text{ PLF}$ ∴ OK

B - UPLIFT @ SHEAR WALL END:

$T = C = \frac{Vh}{L_{SW}} = \frac{(7,508 \text{ lbs})(15 \text{ ft})}{41.33 \text{ ft}} = 2,725 \text{ lbs}$

RESISTING DEAD LOAD:

$W_{ROOF} = 20 \text{ psf} \times 24.5 \text{ ft} \times 41.33 \text{ ft} = 20,252 \#$

$W_{WALL} = 10 \text{ psf} \times 15 \text{ ft} \times 41.33 \text{ ft} = 6,200 \#$

$W_{TOTAL} = 20,252 + 6,200 = 26,452 \text{ lbs}$

$\text{UPLIFT} = 2,725 - 0.9\frac{26,452}{2} = -9,198 \text{ lbs}$

∴ NO UPLIFT

NO HOLD DOWNS REQ'D

Bakker/Petrich/Sabatini

[redacted]

Project: NUVFO STATION #53
Job No.: 07502.01
Subject: LATERAL
By: ALB Date: 04/05/07
Check: Date:
Sheet: 627

C - ANCHORAGE FOR IN-PLANE SHEAR

USE (1) #4 1/2" ø x 3" ANCHOR BOLTS

T_{allow} = 950 lbs
V_{allow} = 1,250 lbs

SPACING @ 4'-0" oc

T_s = 4ft (182 plf) = 728 lbs
V_s = " = 728 lbs

$\left(\frac{728}{1250}\right)^{5/3} + \left(\frac{728}{950}\right)^{5/3} = 1.0$ ∴ OK

USE 1/2" ø x 3" ANCHOR BOLTS @ 4'-0" oc

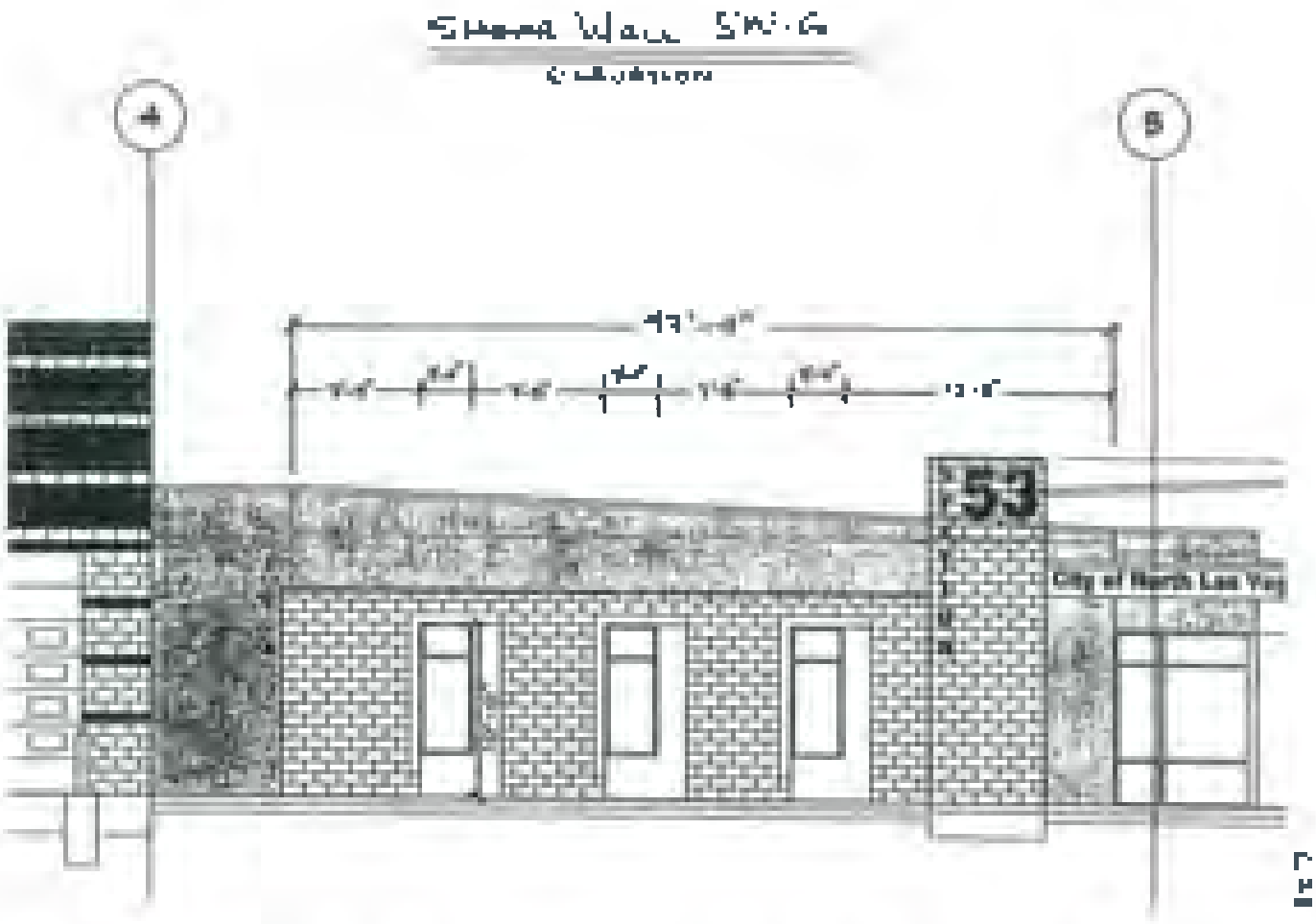
D - COMPRESSION CHORD FORCE:

$T = 2.5(2,725 \text{ lbs}) = 6,813$ lbs

(SEE SHEET A for COMPRESSION CHORD FORCES.)

P_a = 11,335 lbs > 6,813 lbs ∴ OK

USE (2) 6005162-33 BACK-TO-BACK STUDS


4
B
53
City of North Las Veg

Dokker/Perich/Sahatjian

Project: METRO STATION #53
Job No.: 07502-01
Subject: LATERAL
By: AJB Date: 04/25/07
Checked: Date:
Sheet: L27

- TYPE II SHEAR WALL

- SHEAR WALL SW-G

Tributary Area: 10.375 · 47.33 = 491 SF

% of Full Height Sheathing = (3'-4" + 7'-8" + 3'-8" + 15'-9") / 47'-9" = 0.81 ≥ 65%

Max. Opening Height / Story Height = 15'-0" / 15'-0" = 0.6

From AISI Table C3.7-1:

$C_d = 0.83$

V = 4.30 PSF · 491 SF = 2,111 lbs

$v = \frac{V}{C_d \Sigma L_i} = \frac{2{,}111 \text{ lbs}}{(0.83)(38.9')} = 66$ PLF

- SHEATHING:

15/32" APA RATED SHEATHING w/ #8 SCREWS @ 6" oc ALONG ALL PANEL EDGES AND @ 12" oc IN FIELD.

Values: $\frac{780 \text{ PLF}}{2.5} = 312$ PLF > 66 PLF ∴ OK

PET.APP.001288

SHEAR WALL SW-G (CONT.)

K = UPLIFT @ SHEAR WALL END:

$$T = \frac{Vh}{C_a \Sigma L} = \frac{(2,111 \text{ lbs})(15 \text{ ft})}{(0.85)(38-33 \text{ ft})} = 995 \text{ lbs}$$

∴ USE 5/HDBS HOLD DOWNS w/ 7/8" ϕ ANCHORS

w/ T_{allow} = 8,580 lbs ≥ 995 lbs ∴ OK

L - ANCHORAGE FOR IN-PLANE SHEAR:

USE 1 EA 1/2" ϕ x 7" ANCHOR BOLTS

T_{allow} = 950 lbs
V_{allow} = 1,250 lbs

T_s = 4 ft x 66 plf = 264 lbs
V_s = = 264 lbs

$$\left(\frac{264}{1,250}\right)^{5/3} + \left(\frac{264}{950}\right)^{5/3} = 0.19 < 1.0 \therefore \text{OK}$$

USE 1/2" ϕ x 7" @ 4'-0" oc ANCHOR BOLTS

Rutter·Torrich·Tabatini

Name: NJFD Station #53
Number: D7502-01
Type: LATERAL
Initials: MB Date: 04/06/07
Check: Sheet:
Job: 634

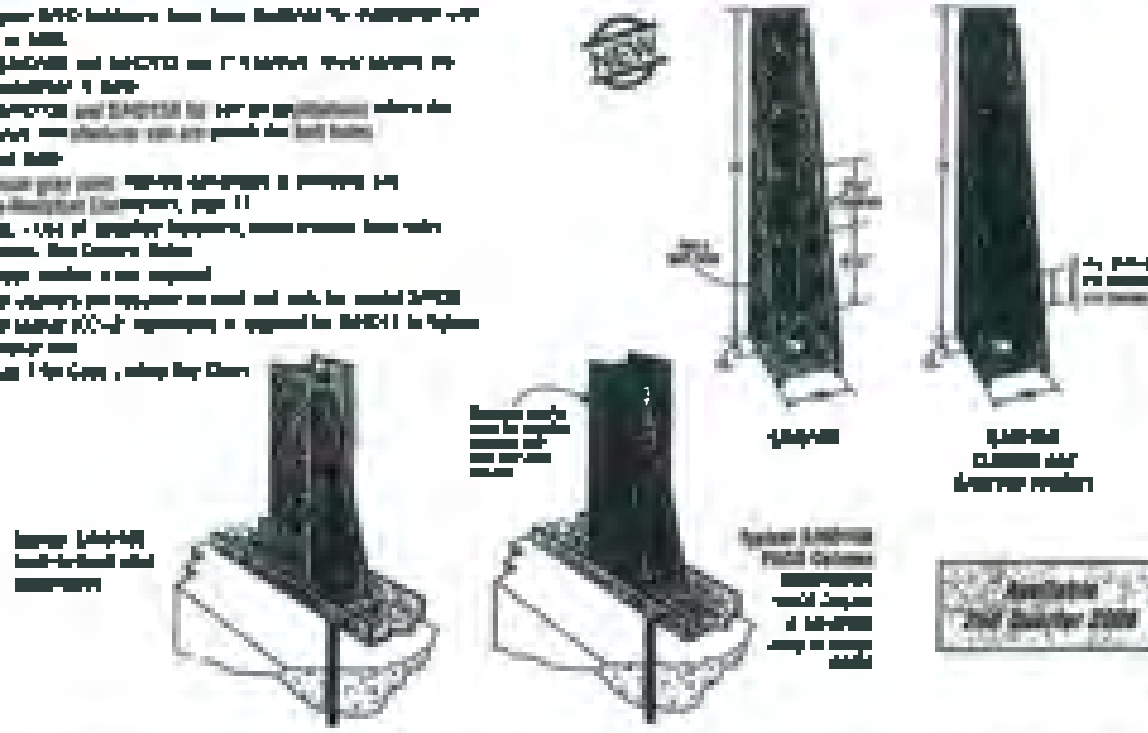
(2) - COMPRESSION CHORD FORCE:

$\Omega T = 2.5\,(995\ \text{lbs}) = 2,488\ \text{lbs}$

(SEE SHEET 4 FOR COMPRESSION CHORD CALCS.)

$P_a = 9,335\ \text{lbs} > 2,488\ \text{lbs}$ ∴ OK

USE (2) 6005162-33 BACT-TO-BACK STUDS





Becker/Parish/Zabelini

[redacted]

Name NUFO STATION 453
Job 07502.01
Subject LATERAL
By AJB Date 08/09/07
Checked
Sheet L33

NORTH - SOUTH DIRECTION

TOTAL SEISMIC SHEAR = 34,460 lbs

ROOF AREA = 8,022 sf

SEISMIC LOAD = $\frac{34,460 \text{ lbs}}{8,022 \text{ sf}}$ = 4.30 psf

TYPE I SHEAR WALLS

SHEAR WALL SW-4.1

TRIBUTARY AREA = 353 sf

LENGTH, $L_{SW-4.1}$ = 13'-10" (13.83 ft)

SHEAR, $v = \frac{4.30 \text{ psf} \times 353 \text{ sf}}{13.83 \text{ ft}}$ = 110 plf

A - SHEATHING

15/32" APA RATED SHEATHING w/ 8d SCREWS AT 6" oc ALONG PANEL EDGES AND AT 12" oc IN FIELD.

NOMINAL SHEAR STRENGTH, R_n = 780 plf

AVAILABLE STRENGTH $v_{allow} = \frac{R_n}{\Omega} = \frac{780}{2.5}$

v = 312 plf > 110 plf ∴ OK

Baker·Perich/Sabatini

NAVFAC Station #53
07502-01
LATERAL
By AJB Date 01/09/07
Chkd By Date
Sheet 63 of

SHEAR WALL SW-9.1 (CONT.)

B - UPLIFT @ SHEARWALL ENDS:

$T = C = \frac{Vh}{L_{SW}} = \frac{(4{,}330\ plf \times 853\ lf)(15\ ft)}{13.83\ ft}$

$T = 1{,}696\ lbs$

RESISTING DEAD LOAD:

$W_{WOOD} = 20\ psf \times 6\ ft \times 17.92\ ft = 2{,}090\ lbs$

$W_{WALL} = 15\ psf \times 15\ ft \times 17.92\ ft = 3{,}920\ lbs$

$W_{TOTAL} = 2{,}090 + 3{,}920 = 6{,}010\ lbs$

$UPLIFT = T - 0.9\left(\frac{W_{TOTAL}}{2}\right) < 1.6W + 0.5(W_{SNOW})$

$= -1{,}059\ lbs$ ∴ No Uplift

∴ No Hold Downs Reqd

C - ANCHORAGE FOR IN-PLANE SHEAR

USE (2) ½"ø x 7" ANCHOR BOLTS

$T_{allow} = 950\ lbs$

$V_{allow} = 1{,}250\ lbs$

ANCHOR SPACING @ 4'-0" oc

$V_2 = 4\ ft \times 110\ plf = 440\ plf$

$T_2 = 4\ ft \times 110\ plf = 440\ plf$

C - ANCHORAGE, SW-9.1 (CONT.)

$$\left(\frac{V_e}{V_a}\right)^{5/3} + \left(\frac{T_e}{T_a}\right)^{5/3} \leq 1.0 \quad (\text{Eq. } 19\text{-}1)$$

$$\left(\frac{440}{1,250}\right)^{5/3} + \left(\frac{140}{950}\right)^{5/3} = 0.15 < 1.0 \therefore \text{OK}$$

USE ½" ϕ ANCHOR BOLTS @ 4'-0" OC

D - COMPRESSION CHORD FORCE:

$\Omega T = 2.5(1,646 \text{ lbs}) = 4,115 \text{ lbs}$

(SEE SW-A FOR (2)6005162-33 CALCULATIONS)

$P_a = 9,335 \text{ lbs} \geq 4,115 \text{ lbs} : \text{OK}$

USE (2) 6005162-33 BACK-TO-BACK STUDS @ 16" OC

Dekker/Perich/Sabatini

[redacted]

Project: Silver Station #53
Project No.: 07502.01
Subject: LATERAL
By: AJB Date: 04/09/07
Check: Check
Sheet No.: 636

Shear Wall : SW-4.9

Tributary Area = 1,835 SF

Length, $L_{SW-4.9}$ = 13'-11" (13.92 FT)

Shear, V = 4.30 PSF x 1,835 SF = 7,891 lbs

Unit Shear, $U = \frac{V}{L} = \frac{7,891 \text{ lbs}}{13.92 \text{ FT}} = 567$ PLF

1) - Sheathing:

Both Sides - 15/32" APA Rated Sheathing w/ 8 screws at 6" on Along Panel Edges and at 12" on In Field.

$U_{allow} = \frac{780 \text{ PLF} + 780 \text{ PLF}}{2.5} = 624 \text{ PLF} > 567 \text{ PLF}$ OK

2) - Uplift & Shear Wall Ends:

$T = C = \frac{Vh}{L_{SW}} = \frac{(7,891 \text{ lbs})(15 \text{ FT})}{13.92 \text{ FT}}$

$T = 8,503$ lbs

Resisting Dead Load:

$W_{ROOF} = 20 \text{ PSF} \times 6 \text{ FT} \times 13.92 \text{ FT} = 1,670$ lbs

$W_{WALL} = 15 \text{ PSF} \times 9 \text{ FT} \times 13.92 \text{ FT} = 1,879$ lbs

$W_{TOTAL} = 1,670 + 2,132 = 3,802$ lbs

Dekker/Perich/Sabatini

[redacted]

NUFD STATION #53
07502-01
LATERAL
AB 04/09/07
CST

SW - 4.9 - UPLIFT (CONT.)

$$\text{UPLIFT} = T - 0.9\frac{\text{DEAD LOAD}}{2}$$

$$= 8,503 \text{ lbs} - 0.9\left(\frac{4,892}{2}\right)$$

$$\therefore \text{UPLIFT} = 6,342 \text{ lbs}$$

∴ USE S/HDB5 HOLD DOWN W/ 5/8" Ø ANCHOR

$T_{allow} = 8,580$ lbs >> 6,342 lbs ∴ OK

5 - ANCHORAGE FOR IN-PLANE SHEAR

USE 1RY 5/8" Ø x 7" ANCHOR BOLTS

$T_{allow} = 1,500$ lbs
$V_{allow} = 2,350$ lbs

ANCHOR SPACING AT 16" (1.33 FT)

$T_a = 1.33 \text{ FT} \times 567 \text{ PLF} = 754$ lbs
$V_s = 754$ lbs

$$\left(\frac{V_s}{V_{allow}}\right)^{5/3} + \left(\frac{T_a}{T_{allow}}\right)^{5/3} \le 1.0 \quad \text{(Eq. 19-1)}$$

$$\left(\frac{754}{2,350}\right)^{5/3} + \left(\frac{754}{1,500}\right)^{5/3} = 0.43 < 1.0 \therefore \text{OK}$$

USE 5/8" Ø ANCHOR BOLTS @ 16" O.C.

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NLF D Series #53
D7501-01
LATERAL
AJB 04/09/97
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3) - COMPRESSION CHORD FORCE

$P_{UT} = 21,503 \text{ lbs} \Rightarrow 21,258 \text{ lbs}$

USE (4) 600S162-43 STUDS

(SEE AISI WIN/CW PROPERTY DATA SHEET)

$I_x = 4 \times 2.3158 \text{ in}^4 = 9.26 \text{ in}^4$

$A_c = 4 \times 0.4969 \text{ in}^2 = 1.99 \text{ in}^2$

$r = 2.28 \text{ in}$

$P_n = A_e F_n$ (EQ C4-1)

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29 \times 10^6 \text{ psi})}{\left(\frac{1.0 \times 180 \text{ in}}{2.28 \text{ in}}\right)^2} = 45,912 \text{ psi}$

$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{33}{16}} = 0.85 > 1.5$ ∴ OK

$F_n = (0.658^{\lambda_c^2}) F_y = (0.658^{0.85^2}) 33,000 \text{ psi}$

$= 24,388 \text{ psi}$

$P_n = 1.99 \text{ in}^2 \times 24,388 \text{ psi} = 43,655 \text{ lbs}$

$P_a = \frac{P_n}{\Omega} = \frac{43,655 \text{ lbs}}{1.8} = 24,253 \text{ lbs} > 21,258 \text{ lbs}$

∴ OK

USE (4) 600S162-43 STUDS @ EACH SW END

www.clarkwestern.com

2001 North American Specification w/2004 Supplement ASD
DATE: 7/31/2007

SECTION DESIGNATION: 800S162-43 Single

INPUT PROPERTIES:

Web Height =	8.000 in	Base Thickness =	0.0451 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0713 in
Bottom Flange =	1.625 in	Yield Stress, Fy =	33 ksi
Stiffening Lip =	0.500 in	Fy after Cold-Work, Fya =	38.1 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

OUTPUT PROPERTIES:

Effective Section Properties, Bending Axis

Neutral Axis from Top Fiber (Yeg)	3.997	in
Moment of Inertia for Deflection (Ix)	3.3158	in^4
Section Modulus (Sx)	0.7873	in^3
Allowable Bending Moment (Ma)	1999.91	Ft-Lb

Gross Section Properties at Full Section, Strong Axis

Neutral Axis from Top Fiber (Yog)	3.9999	in
Moment of Inertia (Ix)	3.3158	in^4
Gross Cross-Sectional Area (A)	0.4498	in^2
Radius of Gyration (Rx)	2.7484	in

Section Properties, Weak Axis

Gross Neutral Axis (Xog) from Web Face	0.4158	in
Gross Moment of Inertia (Iy)	0.1484	in^4
Radius of Gyration (Ry)	0.5782	in
Effective Section Modulus (Sfy)	0.1080	in^3
Effective Neutral Axis (Xeg) from Web Face	0.4798	in
Allowable Moment (May)	177.85	Ft-Lb

Other Section Property Data

Net Area at Punchout	0.3782	in^2
Member Weight per Foot of Length	1.5306	lbf
Allowable Shear Force in Web (Unpunched)	1416	lb
Allowable Shear Force in Web (Punched)	1340	lb
Data for Use in Interaction Equation (Ca)	4864	lb

Torsional Properties

Dist. from Shear Center to Neutral Axis (Xo)	-1.0809	in
St. Venant Torsion Constant (J x 1000)	0.3030	in^4
Warping Constant (Cw)	1.0823	in^6
Radius of Gyration (Ro)	3.5648	in
Torsional Flexural Constant (Beta)	0.8763	

Allowable Web Crippling Loads (lb)

	1.00in END BEND		3.50in INT BEND	
	Cond 1	Cond 2	Cond 3	Cond 4
Single Member	2568	1965	N/S	N/S
Reduction Rc	0.823 + 0.060 wh/t <= 1.0		0.808 + 0.053 wh/t <= 1.0	

Note: Rc applies only Cond 1 and 2

Dokken/Parrish/Sabatini

[redacted]

Name NLVFD Station #53
Appl. No. 23502.01
Subject LATERAL
By AJB Date 04/09/07
Chkd. By Date
Sht. No. L1.0 of

Shear Wall SW-5

Tributary Area = 2,189 SF

Length, L_{SW-5} = 22 FT

Shear, V = 4.30 PLF × 2,189 SF = 9,413 lbs

Unit Shear, $U = \frac{V}{L} = \frac{9,413 \text{ lbs}}{22 \text{ FT}} = 428 \text{ PLF}$

A - Sheathing:

Both Sides - 15/32" NFA Rated Sheathing w/ 8" screws @ 6" o.c. along panel edges and @ 12" o.c. in field.

$V_{allow} = \frac{R_A}{\Omega} = \frac{1(980 \text{ PLF})}{2.5} = 624 \text{ PLF} > 428 \text{ PLF}$ ∴ OK

B - Uplift @ Shear Wall Ends:

$T = C \cdot \frac{Vh}{L_{SW}} = \frac{(9,413 \text{ lb})(15 \text{ FT})}{22 \text{ FT}} = 6,418 \text{ lbs}$

Resisting Dead Load:

W_{ROOF} = 20 PSF × 6 FT × 22 FT = 2640 lbs
W_{WALL} = 15 PSF × 15 FT × 22 FT = 4,950 lbs
W_{TOTAL} = 2640 lbs + 4950 lbs = 7,590 #

$UPLIFT = T - 0.9\left(\frac{W_{TOTAL}}{2}\right) = 6,418 - 0.9\left(\frac{7590}{2}\right)$
$= 3,003 \text{ lbs}$

Baker Perich/Sabatini

[redacted]

NLVFD STATION #53
07501-0
LATERAL
AUB 04/09/07
CHK

SHEAR WALL SW-5 (CONT.)

USE S/HDB8 HOLD DOWN W/ 7/8" ø ANCHORS

W/ T_{allow} = 8,580 lbs > 3,003 lbs UPLIFT : OK

C - ANCHORAGE FOR IN-PLANE SHEAR

USE 5/8" ø x 7" ANCHOR BOLTS

T_{allow} = 1,500 lbs
V_{allow} = 2,750 lbs

ANCHOR SPACING @ 16" o.c. (1.33 ft)

T_s = 1.33 ft x 428 plf = 569 lbs
V_s = " " = 569 lbs

$\left(\frac{569}{2,750}\right)^{5/3} + \left(\frac{569}{1,500}\right)^{5/3} = 0.27 < 1.0$: OK

USE 5/8" ø x 7" ANCHOR BOLTS @ 16" o.c.

D - COMPRESSION CHORD FORCE:

$\Sigma LT = 2.5(6,418 \text{ lbs}) = 16,045 \text{ lbs}$

(SEE SW-4A FOR COMPRESSION CHORD CALCS)
P_a = 21,253 lbs > 16,045 lbs : OK

USE (4)600S162-43 STUDS @ EA. SW END.

PET.APP.001300

Becker/Perich/Sabatini

[redacted]

Job Name: NUFO Station #53
Job No.: 07501-01
Subject: LATERAL
By: AJB Date: 04/09/07
Client: [redacted]
Sheet No.: L43 of [redacted]

TYPE II SHEAR WALL

SHEAR WALL SW-6

TRIBUTARY AREA = 1,600 SF

% FULL-HEIGHT SHEATHING =
$\frac{5'-3\frac{1}{2}" + 6'-0" + 9'-0" + 6'-1"}{35'-3"} = 0.575$

$\frac{\text{MAX. OPENING HEIGHT}}{\text{TOTAL HEIGHT}} = \frac{10'-2"}{15'-0"} = 0.68$

FROM AISI TABLE C3.2-1:

$C_a = 0.76$

$V = 4.30 \text{ PSF} \times 600 \text{ SF} = 2,580 \text{ lbs}$

$U = \frac{V}{C_a \Sigma L_i} = \frac{2,580 \text{ lbs}}{0.76(20.375')} = 166 \text{ PLF}$

A - SHEATHING:

15/32" APA RATED SHEATHING w/ #8 SCREWS AT 6" o.c. ALONG PANEL EDGES AND AT 12" o.c. IN FIELD.

* PER NOTE #3 ON TABLE C2.1-3 - SHEAR WALL ASPECT RATIO GREATER THAN 2:1 ARE PERMITTED AS LONG AS Δ_{allow} IS MULTIPLIED BY $2w/h$, BUT NOT EXCEED 4:1.

PET.APP.001362

Job: NLVFD STATION #53
Job No: 03502-01
Sheet: LATERAL
By: AHB 04/09/03
Check:
Ref: 6W

SHEAR WALL SW-6 : 3 SHEATHING (CONT)
SAME AS PANEL WIDTH

$2\ w/h = 2\ (4\text{FT}/5\text{FT}) = 0.533$

$R_n = 0.533\ (980\ \text{PLF}) = 416\ \text{PLF}$

$U_{allow} = \frac{416\ \text{PLF}}{2.5} = 166\ \text{PLF}$ ∴ OK

B - UPLIFT @ SHEAR WALL ENDS:

$T = \frac{Vh}{C_a \Sigma L_i} = \frac{(2,580\ \text{lbs})(15\ \text{ft})}{3.76\ (20.33\ \text{ft})} = 2,500\ \text{lbs}$

USE S/HDBS HOLD DOWN W/ 7/8" ø ANCHORS

W/ $T_{allow} = 8,580\ \text{lbs} > 2,500\ \text{lbs}$ ∴ OK

C - ANCHORAGE FOR IN-PLANE SHEAR:

USE (2) 1/2" ø x 7" ANCHOR BOLTS

$T_{allow} = 950\ \text{lbs}$
$V_{allow} = 1,250\ \text{lbs}$

SPACING @ 4'-0" OC

$T_{@\ 4\ \text{FT}} \times 166\ \text{P.L.F} = 664\ \text{lbs}$
$V_{@} = 664\ \text{lbs}$

SHEAR WALL SW-6.1 - ANCHORAGE (CONT.)

$\left(\frac{6,649\ lbs}{L250\ lbs}\right)^{1/2} + \left(\frac{6,649\ lbs}{950\ lbs}\right)^{5/3} = 0.90 > 1.0 \therefore$ OK

USE 1/2" ø x 7" @ 4'-0" oc ANCHOR BOLTS

D - COMPRESSION CHORD FORCE:

$\Omega T = 2.5(2,500\ lbs) = 6,250\ lbs$

(SEE SW-A FOR COMPRESSION CHORD CALCS.)

$P_a = 9,335\ lbs > 6,250\ lbs \therefore$ OK

USE (2) 600S162-33 BACK-TO-BACK STUDS @ SW-6.1

5
4

[redacted]

NOVEO STATION #53
09502-01
LATERAL
418 09/21/17
LH2

NORTH - SOUTH DIRECTION

Total Seismic Shear = 80,944 lbs

Shear Along Grids ③ + ④:

$V_{3-4} = \frac{80,944 \text{ lbs}}{2} + 40,472 \text{ lbs}$

$h = 20.67 \text{ ft}$ (248 in)

$M = Vh = (40,472 \text{ lbs})(20.67 \text{ ft})$

$= 836,556 \text{ lb-ft}$ (1,003,710 in-lb)

Roof Loads:

DL = 20 psf

LL = 20 psf

40 psf

$P = 40 \text{ psf} \times 35 \text{ ft} \times 78 \text{ ft} = 109 \text{ K}$

- Check if Wall is Cracked

$f_a = \frac{P}{A} = \frac{109,200 \text{ lbs}}{(11.625 \text{ in})(936 \text{ in})} = 10.09 \text{ psi}$

$S = \frac{bd^2}{6} = \frac{11.625 \text{ in}(936 \text{ in})^2}{6} = 1,668,510 \text{ in}^3$

$f_b = \frac{M}{S} = \frac{1,003,710 \text{ lb-in}}{1,668,510 \text{ in}^3} = 0.60 \text{ psi}$

Baker/Parish/Stanislaw

[redacted]

Project No. SLVED Station #53
Project Name 07501 01
Subject LATERAL
By AJB Date 07/24/07
Checked By — Page —
Sheet 6 of —

$$\frac{P}{A} + \frac{M}{S} = 10.04 \text{ psi} < 0.60 f_y$$



$f_c = 9.44$ psi $\qquad f_t = 1065$ psi

∴ Wall is Un-Cracked

Allowable Stresses:

$$F_b = (1.33)(0.33 f'_m) = 1.33(0.33 \cdot 1500 \text{ psi}) = 658 \text{ psi}$$

$$F_s = 1.33 F_s = 1.33(24{,}000 \text{ psi}) = 31{,}920 \text{ psi}$$

$$f_v = \frac{V}{bd} = \frac{40{,}972 \text{ lbs}}{(11.625 \text{ in})(92 \text{ in})} = 38.45 \text{ psi}$$

$$\frac{M}{Vd} = \frac{1{,}003{,}710 \text{ lb-in}}{(40{,}972 \text{ lbs})(92 \text{ in})} = 0.027 < 1.0$$

$$\Rightarrow F_v = \left(\frac{1}{3}\right)\left[4 - \left(\frac{M}{Vd}\right)\right]\sqrt{f'_m} = \frac{1}{3}\left[4 - 0.027\right]\sqrt{1500}$$

$F_v = 51.29$ psi $>> f_v$ ∴ OK

Bekker/Perich/Sabatini

[redacted]

Project: NLVFD STATION #53
Proj. No.: 07502-01
Subject: LATERAL
By: AJB Date: 07/24/07
Checked: Date:
Sheet: L44

- MINIMUM REINF. REQUIRED:

* PER BERMS I.B.C.

$A_{s(min)} = 0.0007(1.625 in)(12 in/ft) = 0.0137 in^2/ft$

USE #4 @ 12" HEDGE DUTY DUR-O-WALL TRUSS @ 16" OC AND 2-#5 @ BOND BEAMS PER DETAIL 8/S202 HORIZONTAL REINFORCEMENT.

BOND BEAMS: minimum 2 #5

DUR-O-WALL: $0.066 in^2 \times \left(\frac{12 in}{16 in}\right) = 0.0495 in^2/ft$

$A_{min2} = 2.48 + 0.0495 = 2.49 in^2/ft > 0.0137 in^2/ft$

∴ OK

VERT. REINF:

$A_v = \#6 @ 24" OC = 0.44 in^2 \times \frac{12 in/ft}{24 in} = 0.22 in^2/ft$

VERT. REINF REQ'D: $(0.002 \times A_g) + A_{vh}$

$= 0.239 + 0.093 in^2/ft$

$= 0.182 in^2/ft < 0.22 in^2/ft$

∴ OK

Bekker/Petrick/Sabatini

[redacted]

Name: NLVFD Station #53
Job No.: 03502.01
By: CATERAL
Page: A-18 Date: 07/29/03
Location: [redacted] Time: [redacted]
Subject: ESR

- OUT OF PLANE DESIGN:

$$M = \frac{wL^2}{8} = \frac{20\ psf\ [10.67\ ft]^2}{8} = 1,048\ lb\text{-}ft = 12,818\ lb\text{-}in$$

$$A_s = \frac{M}{f_s j d} = \frac{12,818\ lb\text{-}in}{(31,720\ psi)(0.9)(5.8\ in)} \quad \text{(SEE NOTES IN APP.)}$$

$$A_s = 0.0748\ in^2 < 0.22\ in^2$$

∴ #6 @ 24" o.c. is ADEQUATE

- COMPRESSIVE STRESS:

$$r = \frac{t}{\sqrt{12}} = \frac{11.63}{\sqrt{12}} = 3.36\ in$$

$$\frac{h}{r} = \frac{248\ in}{3.36\ in} = 73.8 < 99$$

$$\Rightarrow F_a = \frac{1}{9} f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{(Eq. 2-2)}$$

$$= \frac{1}{9}(1500)\left[1 - \left(\frac{248}{140(3.36)}\right)^2\right]$$

$$F_a = 271\ psi$$

Bekker/Perich/Sabatini

Project Name: NLVFD STATION #53
Project Number: 09502-01
Subject: LATERAL
By: AJB Date: 04/24/07
Check by: Date:
Sheet: 4 of

- Comb. of Plane Stresses:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0$$

$$\frac{10.64}{271} + \frac{0.60}{658} = 0.032 < 1.0 \quad \therefore OK$$

- Buck. of Plane Stresses:

$$f_a = \frac{2M}{K_{cd}}$$

$$K_{cd} = \frac{f_a}{F_{cm}} = \frac{10.64 \text{ psi}}{1160 \text{ psi}(1.50)(1)} = \frac{29.64 \text{ psi}}{.750 \text{ ksi}} = 1.48$$

$$\rho_t = \frac{A_s}{bd} = \frac{0.94 \text{ in}^2}{(12 \text{ in})(5.81 \text{ in})} = 0.0063$$

$$\rho_{bal} = 0.1353$$

$$K = \sqrt{2\rho_t n + (\rho_t n)^2} - \rho_t n = 0.1522$$

$$J = 1 - \frac{K}{3} = 0.8459$$

$$f_b = \frac{2(12.8 \text{ k-in})}{(0.8459)(0.1522)(12 \text{ in})(5.81 \text{ in})^2} = 181.72 \text{ psi}$$

$$\frac{f_a}{F_{cm}} + \frac{f_b}{F_b} = \frac{0.04}{271} + \frac{181.72}{658} = 0.313 < 1.0 \quad \therefore OK$$

REINFORCEMENT IS ADEQUATE

Shearwall "B & F" - Plate Design

Axis 1

Floor	h	L	h/L	A_v	K_c	K_v/K_t	K_t	Force	Shear	Moment	Stress

Keller Pecub Zahalani

[redacted]

NLVFD Stations #23
07501.0: ____
LATERAL
A-18 — 09/23/07
Chkd: ____ Date: ____
ENR

∴ East - West Direction

Total Seismic Shear = 80,994 lbs

Base Shear on each Wall = 40,472 lbs

Shear Walls B & F

- Piers 1 & 5:

V_u = 12,700 lbs × 1.5 = 19,050 lbs (IBC 1604.5.1)

L = 4'-0"

A - Shear Stress:

$f_v = \frac{V}{bd} = \frac{19,050\ lbs}{(14.625in)(93.2in)} = 37.93\ psi$

B - Allowable Shear Stress:

$\frac{M}{Vd} - \frac{H}{2} = \frac{25.67ft\,(12in/ft)}{93.2in} - 1.74 > 1.0$

$F_v = 1.5\sqrt{1,500\ psi} = 58\ psi > f_v$ ∴ OK

C - Min. Area of Reinforcement:

$A_v = \frac{V}{F_s d} = \frac{19,050\ lbs\ (12in/ft)}{(133 \times 24,000psi)(93.2in)} = 0.164\ in^2/ft$

USE 2-#5 @ 48" o.c. (570 @ 0-0-w @ 32" o.c.)

$A_v = 0.169\ in^2/ft > 0.164\ in^2/ft$ ∴ OK

Dekker/Perich/Sabatini

[redacted]

Job: S. UFO Station #52
Job #: 07502-01
Subject: LATERAL
By: AJB Date: 01/23/07
Check: [redacted]
Sheet: 4 of [redacted]

- PIERS 2 & 4:

$V_u = 3,840 \text{ lbs} \times 1.5 = 5,760 \text{ lbs}$

$L = 2.67 \text{ ft}$

A - SHEAR STRESS:

$f_v = \frac{V}{bd} = \frac{5,760 \text{ lbs}}{(1.625 \text{ in})(28.89 \text{ in})} = 17.18 \text{ psi}$

B - ALLOWABLE SHEAR STRESS:

$\frac{M}{Vd} = \frac{H}{d} = \frac{(20.67 \text{ ft})(12 \text{ in/ft})}{28.89 \text{ in}} = 8.6 > 1.0$

$F_v = 58 \text{ psi} > 17.18 \text{ psi} \therefore \text{OK}$

C - MIN AREA OF REINFORCEMENT:

$A_v = \frac{V}{F_s d} = \frac{(5,760 \text{ lbs} \times (12 \text{ in/ft}))}{(1.33 \times 24,000 \text{ psi} \times (28.89 \text{ in}))}$

$A_v = 0.073 \text{ in}^2/\text{ft}$

USE 2-#5 @ 48" oc + STD @ 5.0 & @ 32" oc.

$A_v = 0.169 \text{ in}^2/\text{ft} > 0.095 \text{ in}^2/\text{ft} \therefore \text{OK}$

Becker·Perich/Sabatini

[redacted]

Project: NLVFD Station #53
Project Number: 07502-01
Subject: LATERAL
By: AJB Date: 04/23/08
Checked: Date:
Sheet: L66

∴ FOR B:

V_u = 7,390 lbs × 1.5 = 11,085 lbs

L = 3.33 ft

A - SHEAR STRESS:

$$f_v = \frac{V}{bd} = \frac{11{,}085 \text{ lbs}}{(11.625 \text{ in})(35.96 \text{ in})} = 26.5 \text{ psi}$$

B - ALLOWABLE SHEAR STRESS:

$$\frac{M}{Vd} \quad \frac{M}{d} = \frac{20.67 \text{ ft} \left(12 \text{ in/ft}\right)}{35.96 \text{ in}} = 6.90 > 1.0$$

$F_v = 58$ psi > 26.5 psi ∴ OK

C - MIN. AREA OF REINFORCEMENT:

$$A_v = \frac{V}{f_s d} = \frac{11{,}085 \text{ lbs} \times 12 \text{ in/ft}}{(1.33)(29{,}000 \text{ psi})(35.96 \text{ in})} = 0.116 \text{ in}^2/\text{ft}$$

USE 2-#5 @ 18" OC + 3/8 H.O.W @ 32" OC

$A_s = 0.169 \text{ in}^2/\text{ft} > 0.116 \text{ in}^2/\text{ft}$ ∴ OK

Dekker/Perich/Sabatini

[redacted]

Project Name: NUMMI Stamping #53
Project No.: 0450.2-01
Subject: LATERAL
By: AJB Date: 04/29/97
Sheet No.: Of:
Ref: UBC

OVERTURNING AT EACH PIER

- PIERS 1 & 5:

$T \cdot C = \frac{(12{,}700 \text{ lbs})(20.67 \text{ ft})}{4 \text{ ft}} = 65{,}623 \text{ lbs}$

RESISTING LOADS:

P_{ROOF}: 20 PSF × 3 FT = 60 PLF (3 FT = 6 FT / 2)

WALL: 133 PSF × 13.33 FT = 1,773 PLF (13'-4" = 13.33 FT)

$R_A = \left(\frac{4 \text{ FT}}{2} + \frac{13'-4"}{2}\right)(1{,}773 \text{ PLF}) \times 0.90 = 13{,}835 \text{ lbs}$ (0.90 = NON-BEARING TILT-UP OPENING)

NET UPLIFT = $-65{,}623 \text{ lbs} + 13{,}835 \text{ lbs}$

UPLIFT = $-51{,}792 \text{ lbs}$

USE 1RH 4-#6 @ EA END OF PIER

$f_t = \frac{P}{A} + \frac{51{,}792}{1.77 \text{ in}^2} = 29{,}261 \text{ PSI}$

$F_t = 0.6(60{,}000 \text{ PSI}) = 36{,}000 \text{ PSI} > f_t$:OK

* WEIGHT OF INTERSECTING WALL & END ASSUMED AS LESS CRITICAL THEN HALF THE WALL DEAD LOAD ABOVE THE DOOR OPENINGS.

Architect/Permit/Subpoint

[redacted]

Page Name: NLVFD STATION #53
Page No.: 09502-01
Subject: LATERAL
By: AEB Date: 01/24/07
Sheet: _____ of _____
Job No.: 257

- P.ERS 2 : 4 :

$$T = C = \frac{3,840 \text{ lbs} \left(20.67 \text{ ft}\right)}{2.67 \text{ ft}} = 29,728 \text{ lbs}$$

RESISTING LOADS:

ROOF: 20 PSF x 3 FT = 60 PLF
WALL: 133 PCF x 18.83 FT = 1,773 PLF

$$R_{DL} = \left(\frac{2.67 \text{ ft}}{2} + \frac{18 \text{ in}}{2}\right)\left(1,833 \text{ PLF}\right) \times 0.90 = 13,206 \text{ lbs}$$

(½ WALL + ROOF EXCESS)

NET UPLIFT = -29,728 lbs + 13,206 lbs = -16,522 #

USE STRAP 2 - #6 @ EA. END OF P.ERS

$$F_t = \frac{P}{A} = \frac{16,522 \#}{0.88 \text{ in}^2} = 18,795 \text{ psi}$$

$F_t = 0.6 \left(60,000 \text{ psi}\right) = 36,000 \text{ psi}$ ∴ $f_t < F_t$ OK

- PIER 3:

$$T = C = \frac{(7{,}390\ \text{lbs})(20.67\ \text{ft})}{3.33\ \text{ft}} = 45{,}871\ \text{lbs}$$

RESISTING LOADS:

Roof: 40.0 PLF
Walls: 1,273 PLF

$$R_{DL} = \left(\frac{3.33\ \text{ft}}{2} + \frac{13'\text{-}4''}{2}\right)(1{,}038\ \text{PLF}) \times 0.90 = 13{,}750\ \text{lbs}$$

NET UPLIFT = -45,871 lbs + 13,750 lbs

UPLIFT = -32,121 lbs

USE 4 - #6's @ EA END OF PIER

$$f_t = \frac{32{,}121\ \text{lbs}}{1.77\ \text{in}^2} = 18{,}148\ \text{psi}$$

F_t = 36,000 psi > f_t ∴ OK

Shear Wall Design - Area 3

- North-South Direction

Seismic Base Shear = 5,591 lbs

Shear Wall SW-2

$V = \frac{5,591 \text{ lbs}}{2} = 2,796 \text{ lbs}$

$M = Vh = 2,796 \text{ lbs} \times 12.5 \text{ ft} = 34,950 \text{ lb-ft}$

$\frac{M}{Vd} = \frac{34,950 \text{ lb-ft} (12 \text{ in/ft})}{(2,796 \text{ lbs})(132 \text{ in})} = 1.14 > 1.0$

$\Rightarrow f_v = 1.5\sqrt{f'_m} = 1.5\sqrt{1,500 \text{ psi}} = 58 \text{ psi}$

$F_b = 1.33 (0.33 f'_m) = 658 \text{ psi}$

$F_s = 1.33 (24,000 \text{ psi}) = 31,920 \text{ psi}$

- Flexural Design:

Use Det 1 - #5 @ 8ins OF Wall

$\rho = \frac{A_s}{b_w d} = \frac{0.31 \text{ in}^2}{(7.625 \text{ in})(132 \text{ in})} = 0.000308$

$\rho_n = 0.0003 (29) = 0.0089$

$K = \sqrt{2\rho_n + (\rho_n)^2} - \rho_n = 0.1248$

Dekker/Perich/Sabatini

Project: NLVFD Station #53
Job No.: 07502-01
Subject: LATERAL
By: AMB Date: 11/24/08
Chkd: [redacted]
Sheet: 6 of [redacted]

$k = 1 - \frac{k}{3} = 0.9584$

$f_b = \frac{2M}{k\, b_w\, d^2} = \frac{2\,(34{,}950\ \text{lb-ft})(12\ \text{in/ft})}{(0.9584)(0.1248)(9.625\ \text{in})(32\ \text{in})^2}$

$f_b = 528\ \text{psi} \quad < F_b \quad \therefore$ OK

$f_s = \frac{M}{d\, A_s} = \frac{(34{,}950\ \text{lb-ft})(12\ \text{in/ft})}{(0.9584)(32\ \text{in})(0.31\ \text{in}^2)}$

$f_s = 16{,}694\ \text{psi} \quad < F_s \quad \therefore$ OK

- MIN. AREA OF SHEAR REINFORCEMENT:

$A_v = 0.0007\,(B)(12) = 0.0672\ \text{in}^2/\text{FT}$

USE 2-#5 @ 48" OC AT BOND BEAMS AND STD. NO. 9 BAR O-WIRE TIES @ 16" OC

$A_s = 0.036\ \text{in}^2/\text{FT} + 0.045\ \text{in}^2/\text{FT} = 0.081\ \text{in}^2/\text{FT}$

$A_s > A_v \quad \therefore$ OK

Dekker/Perich/Sabatini

[redacted]

Job: NLWFD Station #13
Job #: 03501-01
Subject: LATERALS
By: ALB Date: 01/31/03
Check: Date:
Sheet: 2 of

- East - West Direction.....

SHEAR Wall SW: D

$V = 2,796$ lbs

$M = 2,796$ lbs $\times 12.5$ ft $= 34,950$ lb-ft

$\frac{M}{Vd} = \frac{34,950 \text{ lb-ft} (12 \text{ in})}{(2,796 \text{ lbs})(104 \text{ in})} = 0.735 < 1.0$

$\therefore \overline{v}_{uc} = \frac{1}{2}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \leq \left(12.0 - \frac{45M}{Vd}\right)$

$= \frac{1}{2}\left(4 - 0.735\right)\sqrt{1,500 \text{ psi}} \leq \left(120 - 45(0.735)\right)$

$= 63.23$ psi $<$ 86.98 psi $\therefore$ ok

$\overline{v}_u = 68.23$ psi

$\overline{v}_b = 658$ psi

$F_v = 31,920$ psi

Dekker/Perich/Sabatini

Project: Wood Structures
Job No.: 07502.01
Subject: LATERAL
Sheet No.: A-06 of 24
Checked: ____ Date: ____
By: LKE

= FLEXURAL DESIGN: ...

USE 1-#5 @ END OF WALL

$$\rho = \frac{A_s}{b_w d} = \frac{0.31\ in^2}{(11.25\ in)(20\ in)} = 0.00139$$

$$\rho n = (0.00139)(19) = 0.00578$$

$$K = \sqrt{2\rho n + (\rho n)^2} - \rho n = 0.102$$

$$j = 1 - \frac{K}{3} = 0.966$$

$$f_c = \frac{2M}{j K b_w d^2} = \frac{2(34,950\ lb\cdot ft)(12\ in/ft)}{(0.966)(0.102)(11.25\ in)(20\ in)^2}$$

$f_c = 26.83\ psi$ < F_c ∴ OK

$$f_s = \frac{M}{j d A_s} = \frac{34,950\ lb\cdot ft\ (12\ in/ft)}{(0.966)(20\ in)(0.31\ in^2)}$$

$f_s = 6,865\ psi$ < F_s ∴ OK

MIN. AREA OF SHEAR REINFORCEMENT:

$A_v = 0.0672\ in^2/ft$

USE 2-#5 @ 18" OC @ BOND BEAMS AND STD. NO. 4 DUR-O-WALL TRUSSES @ 16" OC

$A_s = 0.0975\ in^2/ft$ > A_v ∴ OK

Beckwith Perich / Sabatini

[redacted]

Project Name: NAVFD Station #52
Project No.: 07502-01
Title: LATERAL
By: AJB Date: 04/30/08
Check: Date:
Sheet: 6&7 of:

Shear Wall Sw-1

L = 7.0' (EA)

V = 2,796 lbs

v = V/L = 2,796 lbs / 7 ft = 399 plf

A - Sheathing:

Both Sides - 15/32" APA Rated Sheathing w/ 8 screws @ 6" o.c. at Panel Edges and at 12" o.c. in Field.

$v_{allow} = R_n/\Omega = 2{,}930 \text{ lbs}/2.5 = 1{,}624 \text{ plf} > 399 \text{ plf}$ OK

B - Uplift @ Shear Wall Ends:

$T = C = Vh/L = (2{,}796 \text{ lbs})(12.5 \text{ ft})/7 \text{ ft}$

T = 4,993 lbs

Resisting Dead Load:

$W_{roof} = 20 \text{ psf} \times 3 \text{ ft} \times 7 \text{ ft} = 420 \text{ lbs}$

$W_{wall} = 15 \text{ psf} \times 12.5 \text{ ft} \times 7 \text{ ft} = 1{,}313 \text{ lbs}$

$W_{total} = 420 + 1{,}313 = 1{,}733 \text{ lbs}$

Dekker/Perich/Sabatini

Project: NAVFD STATION #53
Project No.: 03501-01
Subject: LATERAL
By: AJB Date: 04/30/03
Check: Date:
Sheet: LOAD

UPLIFT = T - 0.9 (W total / 2) = 4,993 - 0.9 (1,733 / 2)

UPLIFT = 4,214 lbs

USE 5/4000 psi Hilti Dewn w/ 5/8" ø Anchors

T allow = 8,580 lbs >> 4,214 lbs ∴ OK

E - Anchorage For In-Plane Shear:

USE 2EA 5/8" ø x 7" Anchor Bolts @ 16" o.c.

T allow = 1,500 lbs
V allow = 2,750 lbs

T_a = 1.33T = 399 psi = 531 psi
V_a = 1.33V = 309 psi = 53 psi

(581 lbs / 2,750 lbs)^5/3 + (531 lbs / 1,500 lbs)^5/3 = 0.342 < 1.0

∴ OK

USE 5/8" ø x 7" Anchor Bolts @ 16" o.c.

D - Compression Chord Force:

ΩT = 2.5 (4,993 lbs) = 12,483 lbs

(See SW-49 for Compression Chord Calculations)

P_a = 24,253 lbs > 12,483 lbs ∴ OK

USE (4) 6003 x 43 - 33 BACK-TO-BACK STUDS

PET.APP.001323

Baker/Parsons Submittal

[redacted]

Name: NLVPD Station #53
Project No.: 07502-01
Type: LATERAL
By: AJB Date: 04/25/07
Check: Date:
Sheet: L03

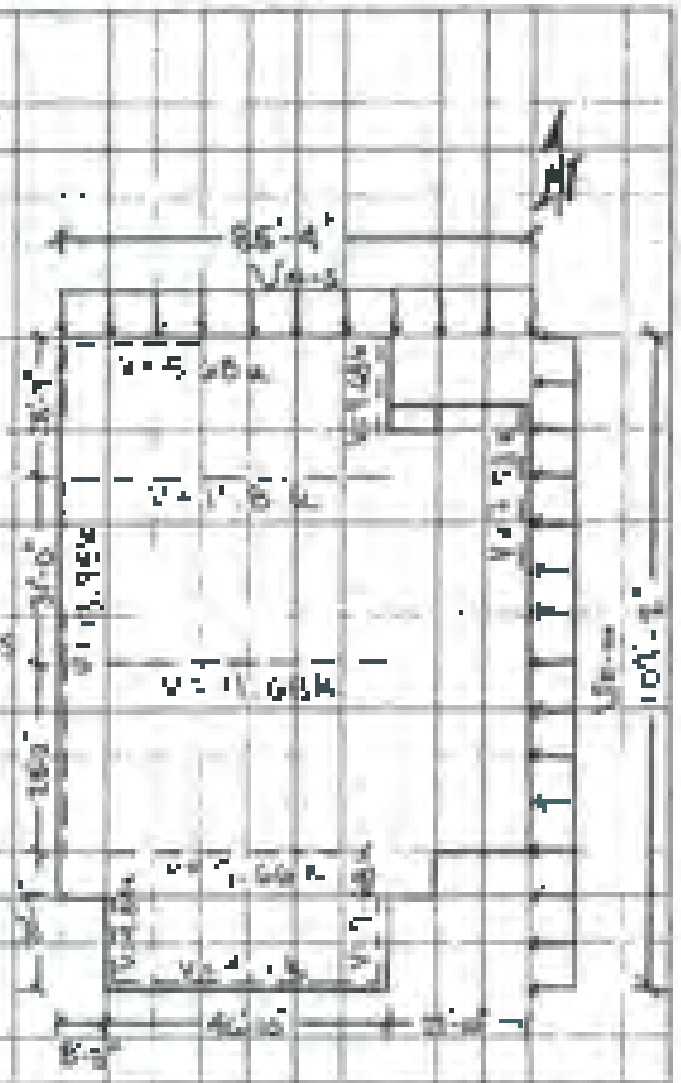
Roof Diaphragm

Area 1:

V = 34,460 lbs

ASCE 7-05: 12.3.2.1 -
Horizontal Irregularities:
- Reentrant Corner
- Diaphragm Discontinuity

12.3.3.4: Increase in forces due to irregularities for Category D Design Forces shall be increased 25% for connections of diaphragm to vertical elements. Collectors and their connections also.


85'-4"
V = 0.5 kips
V = 1.7 kips
V = 11.6 kips
V = 1.6 kips
V = 1.6 kips
V = 1

Dekker/Perich/Sabatini

Name: NLWFD STATION #53
Number: 04502-01
Subject: LATERAL
By: AJB Date: 04/25/07
Check: Date:
Sheet: 1 of 6

USE TRY 1.5B - 22 GA METAL DECK ~ 5/8" ϕ PUDLS (1 PUDL / 3rd PANEL) + #10 SMS, @ SIDE LAP FASTENERS PER SPAN (@ 12" oc)

$\Delta_{allow} = 330 LF > \Delta_{actual} (130 LF)$ ∴ OK

- CHECK DEFLECTION:

$$G' = \frac{K_\Delta}{3.78 + \frac{0.2\,D_B}{SPAN} + 3\,K_\Delta \cdot SPAN}$$

$$G' = \frac{}{3.78 + \frac{0.3(.072)}{6ft} + 3(.230)\cdot 6ft}$$

$G' = 14.14 \frac{k}{in}$

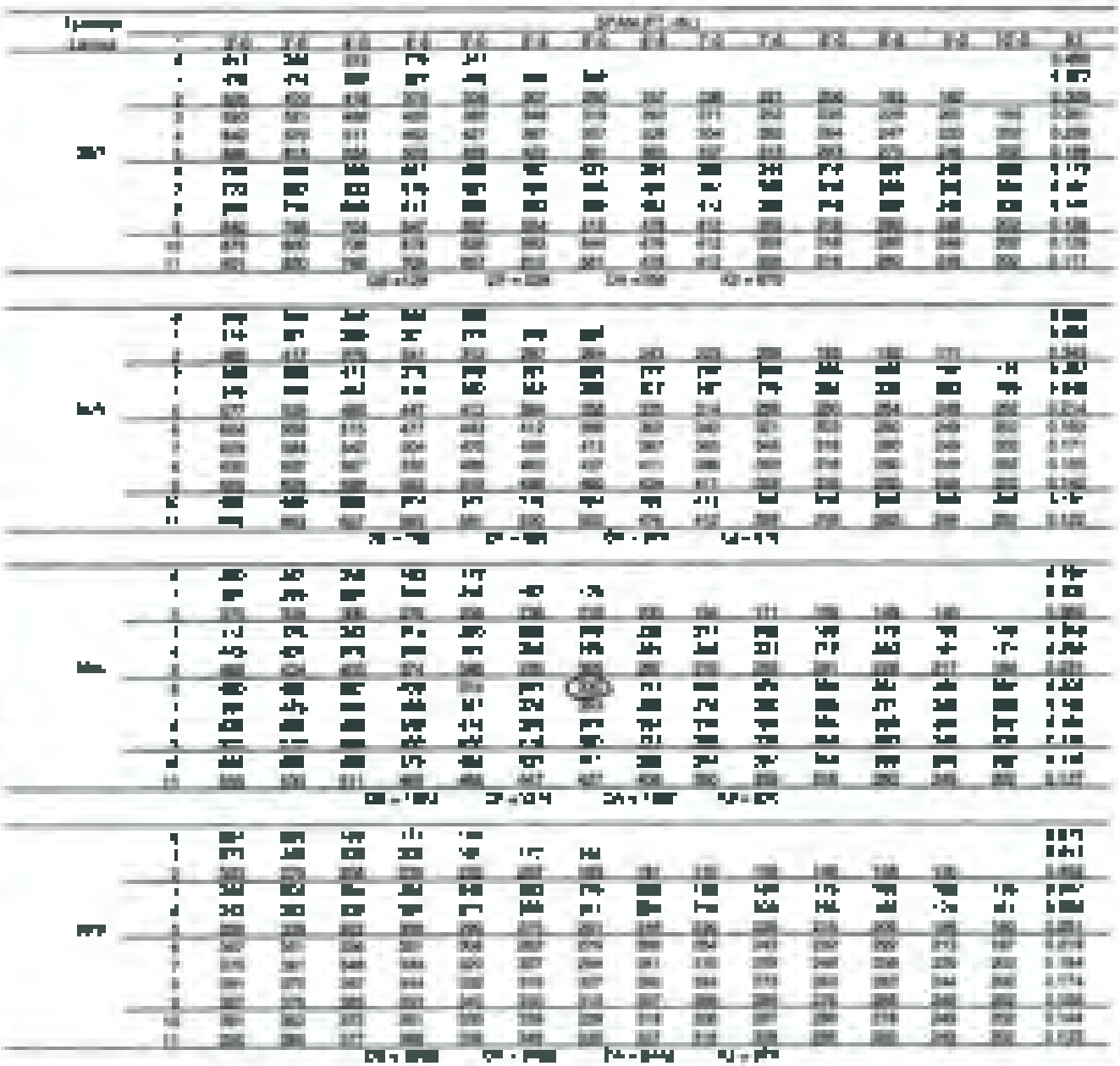
$$\Delta_L = \frac{\omega L^2}{8BG'} = \frac{(0.36\,\frac{k}{ft})(109.9\,ft)^3}{8(85.33\,in)(14.14\,\frac{k}{in})}$$

$\Delta_L = 0.17$ in. $< \frac{L}{240}$ ∴ OK

1.5 (B, F, A) 22 ALLOWABLE DIAPHRAGM SHEAR STRENGTH (PLF)

SUPPORT FASTENERS: 5/8" puddle welds[1]

SIDELAP FASTENERS: #10 TEK screws



[1] In addition to welds, fillet welds or ...

[2] ...

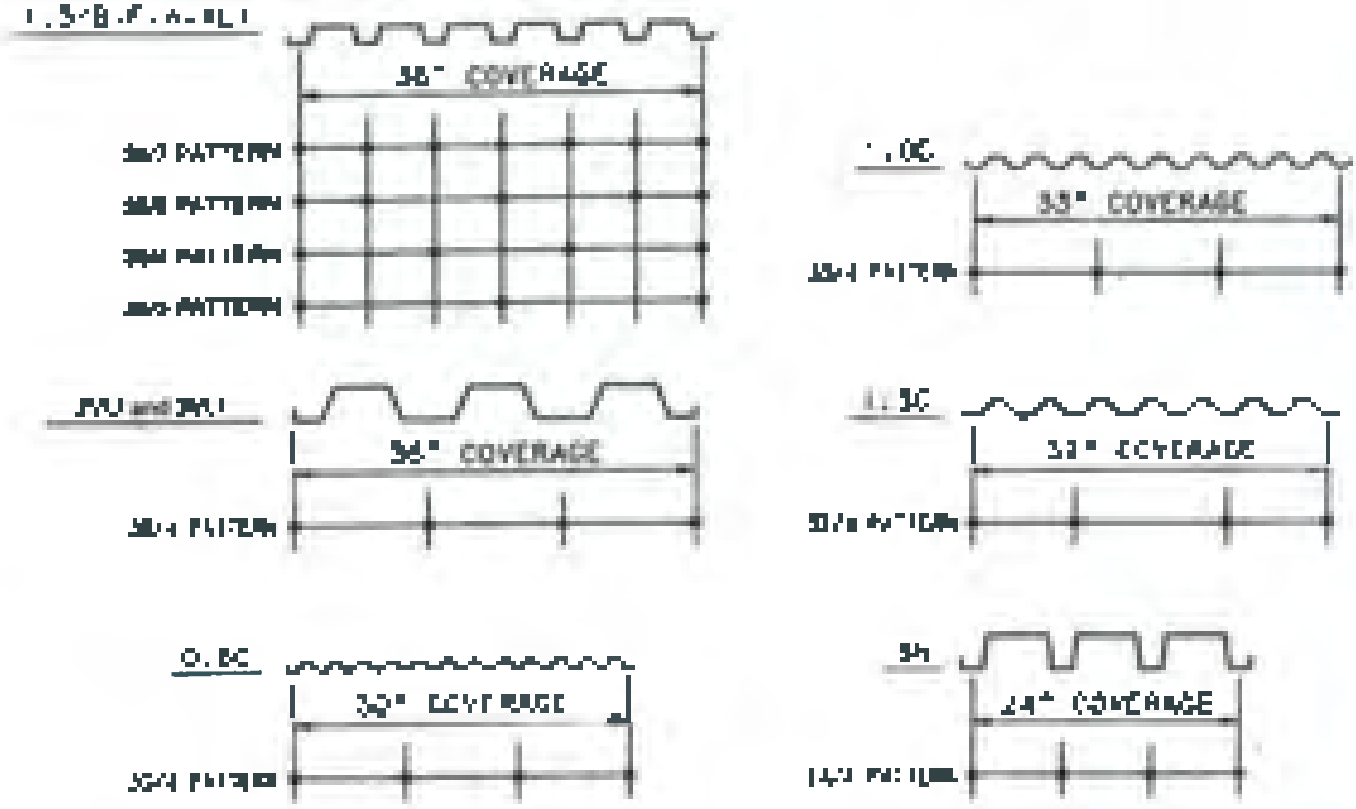
[3] ...

$$q = \frac{V}{\left(\frac{L}{1000G'} + \frac{2L}{W}\right)} \quad \text{Equation}$$

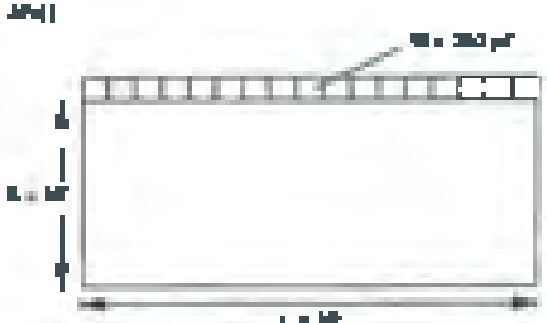
Where:

Equation 3g, $G' = C_1 C_2$

TYPICAL FASTENER LAYOUT



DIAPHRAGM SHEAR STRENGTH AND STIFFNESS DESIGN EXAMPLE



$$G = \frac{K_D}{3.78 + \frac{0.30\,t_s}{Span} + 36\,C_s\,Span} = \frac{[illegible]}{3.78 + \frac{0.30(0.028)}{5} + 36(0.0171)(5)} = [illegible]$$

$$\Delta_{shr} = \frac{wL^2}{8GBD} = \frac{0.250(360)^2}{8(1,000)(5)(36)} = 0.45''$$

Strength Check

$$R = wL/2 = \frac{250(360)}{2} = 7500\ lbs \qquad \beta = \frac{7500}{48} = 156\ plf < 781\ plf$$ OK

Dekker/Perich/Sabatini

[redacted]

NLRD STATION #53
03702-01
CALCULATIONS
BY AGB DATE 04/04/07
CHK'D [illegible] SHT
LL9

AREA 2:

V = 80,999 lbs.

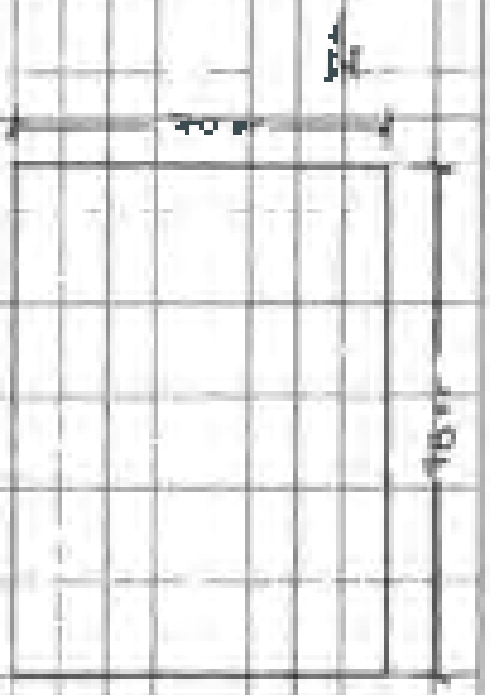
$\frac{V}{2}$ = 40,472 lbs

- NORTH-SOUTH DIRECTION:

$U = \frac{40,472 \text{ lbs}}{78 \text{ FT}} = 518 \text{ PLF}$

- EAST-WEST DIRECTION:

$U = \frac{40,472 \text{ lbs}}{70 \text{ FT}} = 578 \text{ PLF}$



USE 1.5B-18 GA METAL DECK w/ 5/8" ø PUDDLE WELDS (36/7 PATTERN) w/ #10 TEK SCREWS @ SIDE LAP FASTENERS PER SPAN (@ 12" oc.)

$U_{allow} = 619 \text{ PLF} > U_{actual} (578 \text{ PLF})$ ∴ OK

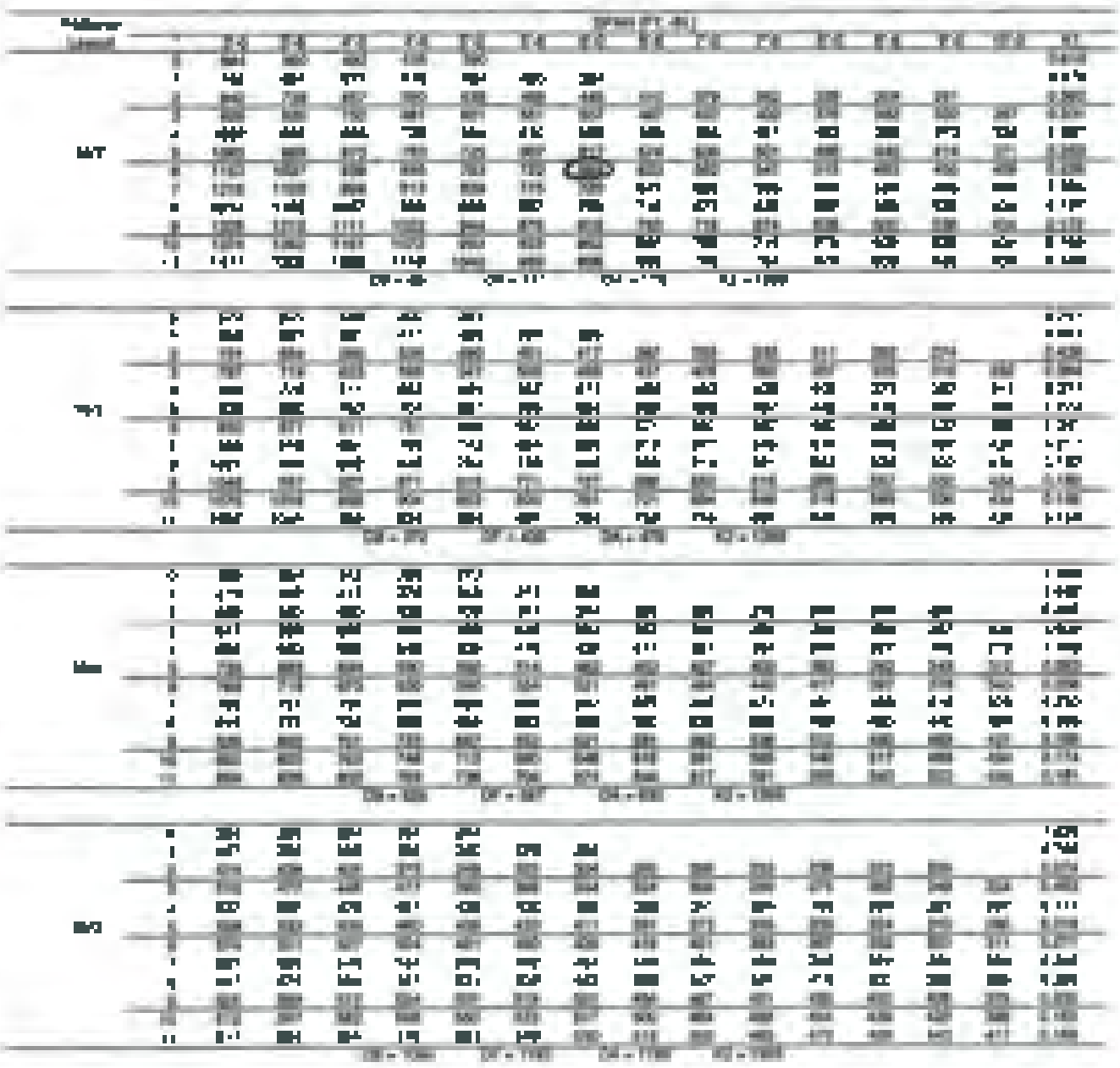
- CHECK DEFLECTION:

$G' = \frac{1.39B}{3.78 + \frac{0.3(68)}{6 \text{ FT}} + 3(0.224) \times 6 \text{ FT}} = 127.1$

$\Delta_L = \frac{(0.578 \text{ K/F})(78 \text{ FT})^3}{8(70 \text{ FT})(127.1)} = 0.0494 \text{ INCH} < L/240$

∴ OK

1.5 pcf, P, Aj 18 ALLOWABLE DIAPHRAGM SHEAR STRENGTH (PLF)
SUPPORT FASTENERS: 5/8" puddle welds[1]
SIDELAP FASTENERS: #10 TEK screws



[1] [illegible]
[2] [illegible]



Dekker/Perich/Sabatini

NEED STARBUCKS #53
07502-01
LATERAL
A-1B 04/25/07
L71

AREA 3:

V = 5,591 lbs

PER 3.3.4 INCREASE IN FORCES DUE TO DISCONTINUITIES FOR CAT D, INCREASE FORCES BY 25% FOR DIAPHRAGM CONNECTIONS TO VERTICAL MEMBERS AND FOR COLLECTORS & THEIR CONNECTIONS

V = 5,591 × 1.25 = 6,989 lbs

- NORTH - SOUTH DIRECTION:

$\frac{V}{2} = \frac{6,989 \text{ lbs}}{2} = 3,495 \text{ lbs}$

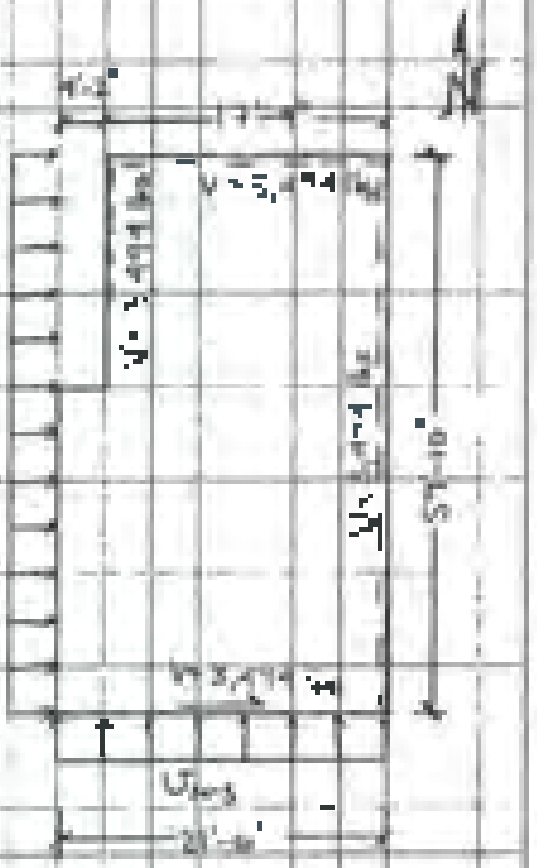
$v_{max} = \frac{3,495 \text{ lbs}}{57.83 \text{ ft}} = 58 \text{ plf}$

- EAST - WEST DIRECTION:

$v_{max} = \frac{3,494 \text{ lbs}}{21 \text{ ft}} = 163 \text{ plf}$ CONTROLS

USE 1.5 B-22 GA METAL DECK W/ 5/8" PUDDLE WELDS (36/4 PATTERN) W/ #10 TEK SCREWS @ SIDE LAP FASTENERS PER SPAN (2 1/2" OC)

$v_{allow} = 330$ PLF > v_{demand} (163 PLF) : OK


N
V = 5,494 lbs
V = 5,494 lbs
V = 3,495 lbs
V = 3,495 lbs
V = 3,495

Miller Zeriah Gaboldoni

NLNFD Station #53
03552-01
Lateral
By ADB Date 04/25/07
Checked Date
Sheet L32

- Check Deflection:

$$G = \frac{870}{3.98 + \frac{0.3(1.032)}{G_m} + 3(0.205) \text{ at } 6 FT}$$

$$G = 14.25 \text{ k/in}$$

$$\Delta_E = \frac{W L^3}{8 E G} = \frac{(10.03 \text{ kip})(59.83 \text{ in})^3}{8(21 \text{ k/in}^2)(14.25 \text{ k/in})}$$

$$\Delta_E = 0.238 \text{ in} < L/240 : 0.249 \text{ in}$$

Rockwell/Perich/Robertini

Project Name: NLVFD Station #3
Project Number: 97502-01
Subject: LATERAL
By: AJB Date: 01/25/07
Checked: Date:
Sheet: L13

Diaphragm Chord Design - Area 1

- From Area 1 Diaphragm

w_L = 305 plf (controls - North-South Direction)

$$M = \frac{w_L l^2}{8} = \frac{305 plf (85.33 ft)^2}{8} = 459,626 \text{ lb-ft}$$

$$T = C = \frac{M}{b} = \frac{459,626 \text{ lb-ft}}{129.17 in} = 4,210 \text{ lbs}$$

USE ~~1~~ 4 L5 x 3 x 1/4 CHORD MEMBER - BEARING

$A = 1.94 in^2$

$F_y = 36 ksi$

$F_t = 0.6 F_y = 0.6(36 ksi) = 21.6 ksi$

$$f_t = \frac{P}{A} = \frac{4,210 \text{ lbs}}{1.94 in^2} = 2,170 psi < F_t \quad \text{OK}$$

CHECK L3 x 3 x 1/4 - NON-BEARING

$A = 1.44 in^2$

$$f_t = \frac{4,210 \text{ lbs}}{1.44 in^2} = 2,924 psi < F_t \quad \text{OK}$$

USE L5 x 3 x 1/4 @ BEARING WALLS
L3 x 3 x 1/4 @ NON-BEARING WALLS

Ammann & Whitney

[redacted]

Project: Wood Story #53
Job No.: 0325.2-01
Subject: LATERAL
By: AUB Date: 04/25/07
Check: Date:
Sheet: 1.36

Diaphragm Chord Design - Area 2

$V_2 = 80{,}999$ lbs

$w = \frac{80{,}999 \text{ lbs}}{78 \text{ ft}} = 1{,}038$ plf

$M = \frac{wL^2}{8} = \frac{(1{,}038 \text{ plf})(78 \text{ ft})^2}{8} = 789{,}379$ lb-ft

$T = C = \frac{M}{b} = \frac{789 \text{ k-ft}}{70 \text{ ft}} = 11.27$ k

USE DKY 2-#5 ℄ Bond Beam

$f_t = \frac{P}{A} = \frac{11.27 \text{ k}}{0.61 \text{ in}^2} = 18.48$ ksi

$F_t = (0.6) F_y = 0.6(60 \text{ ksi}) = 24 \text{ ksi} > 18.5$ ksi

∴ OK

∴ 2-#5 ℄ Bond Beam is adequate for loads in all directions in Area 2

Diaphragm Chord Design - Area 2

* From Area 3 Diaphragm

$w = 163$ plf (Controls in East West Direction)

$$M = \frac{wL^2}{8} = \frac{(163\text{ plf})(70.11\text{ ft})^2}{8} = 73,033\text{ lb-ft}$$

$$T = C = \frac{M}{b} = \frac{73,033\text{ lb-ft}}{21.5\text{ ft}} = 3,394\text{ lbs}$$

* Same As Area 1 Chord

USE L5x3x1/4 @ Bearing Walls
L3x3x1/4 @ Non-Bearing Walls

Bekker/Porich/Sabatini

Project: Raised Section #53
Project No.: 03502-01
Client: LATERAL
By: AJB Date: 04/26/05
Check: Date:
Sheet: L26

Diaphragm Chord Splice Design

IBC 2204.1 - Connections:

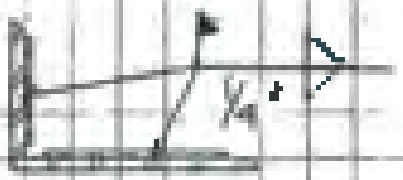
$V_{max.} = 40,472$ lbs (Area 2)

- Check weld:

$P = (k_{in})(4)(0.928)$

$P = 40.8k > 40.5k$ ∴ OK

(See Detail C3/S301)

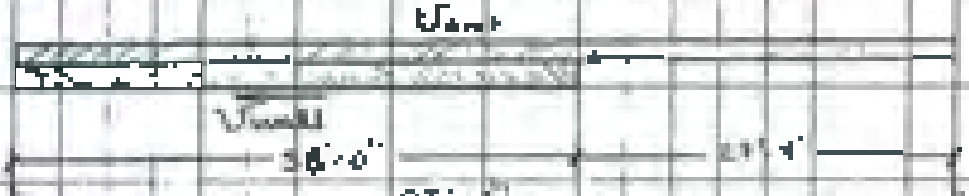


Dekker/Perich/Sabatini

Project: NLVFD Station #53
Project No.: 02502.01
Subject: LATERAL
By: AJS Date: 04/26/01
Checked: Date:
Sheet: L-7

Collector Force Design - Area 1

SW-C2



V = 11,800 lbs

$v_{diaph} = \frac{11,800 \text{ lbs}}{85.33 \text{ ft}} = 138 \text{ plf}$

$v_{wall} = \frac{11,800 \text{ lbs}}{36 \text{ ft}} = 103 \text{ plf}$



Net Unit Shear

Maximum Collector Force = 1331 lbs

USE SIMPSON MSTC40 STRAP TIE w/ T = 4,600 lbs

Dokker/Parish/Sabatini

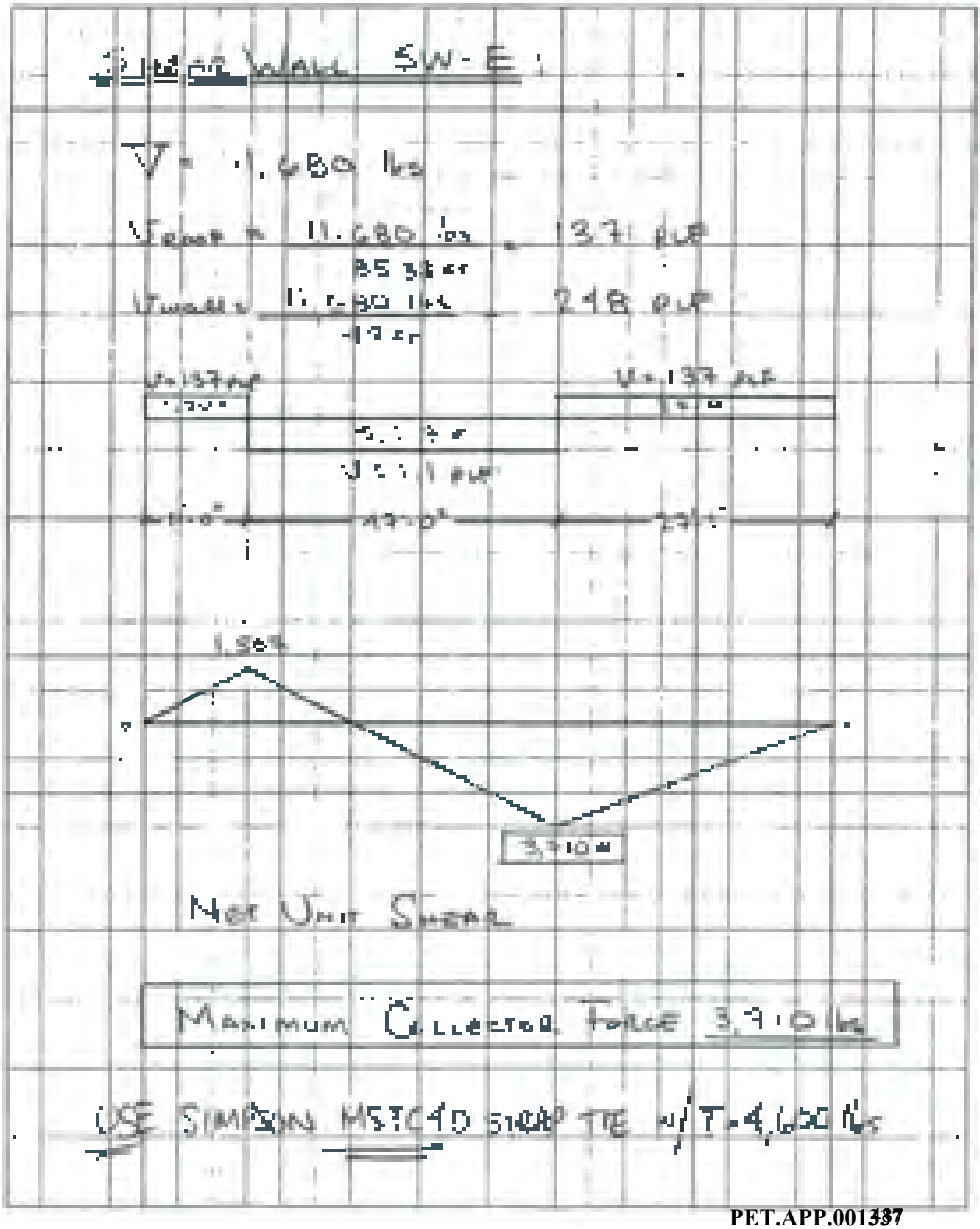
Name NLVFD Station #53
Number 07502-01
Type LATERAL
By AJB Date 09/24/07
Chkd _____ Date _____
Sheet 1 of 2 _____

Lateral Load SW-E:

V = 1,680 lbs

$V_{base} = \frac{11,680 \text{ lbs}}{85.33 \text{ sf}} = 137 \text{ psf}$

$V_{wall} = \frac{11,680 \text{ lbs}}{47 \text{ sf}} = 248 \text{ psf}$



Net Unit Shear

Maximum Collector Force 3.91 Kips

USE SIMPSON MSTC40 STRAP TIE w/ T=4,000 lbs

Dekker/Perich/Sabatini

[redacted]

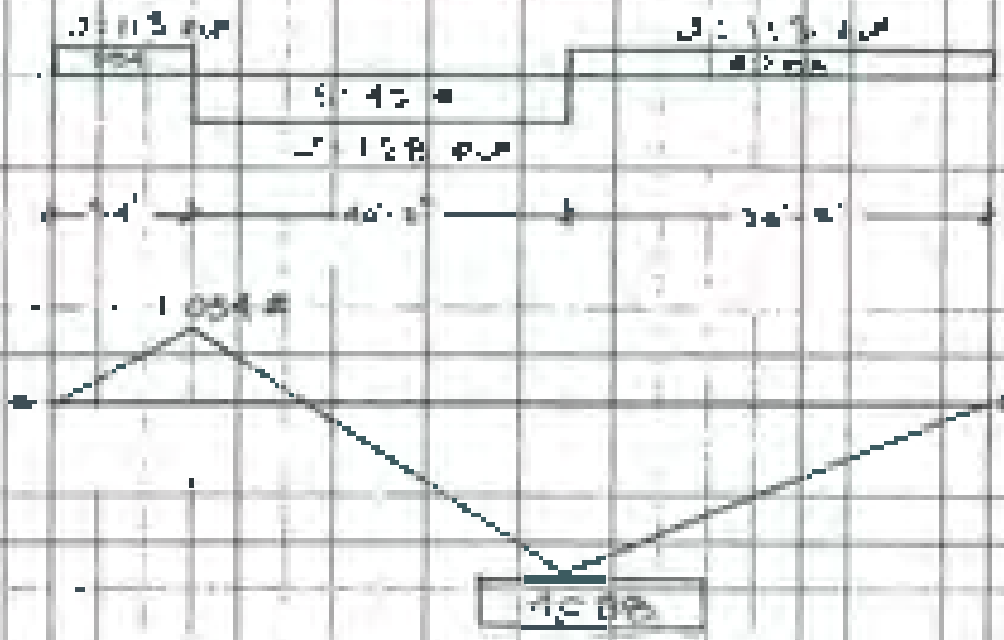
Project: NLVFD Station #53
Project No.: 07502-01
Subject: CALCS
By: AJB Date: 11/14/07
Check by: No.:
Sheet: 279

Shear Wall Sw. E.7

V = 1,660 lbs

$v_{floor} = \frac{1,660 \text{ lbs}}{85.33 \text{ ft}} = 19.5 \text{ plf}$

$v_{wall} = \frac{1,660 \text{ lbs}}{40.7 \text{ ft}} = 39.1 \text{ plf}$

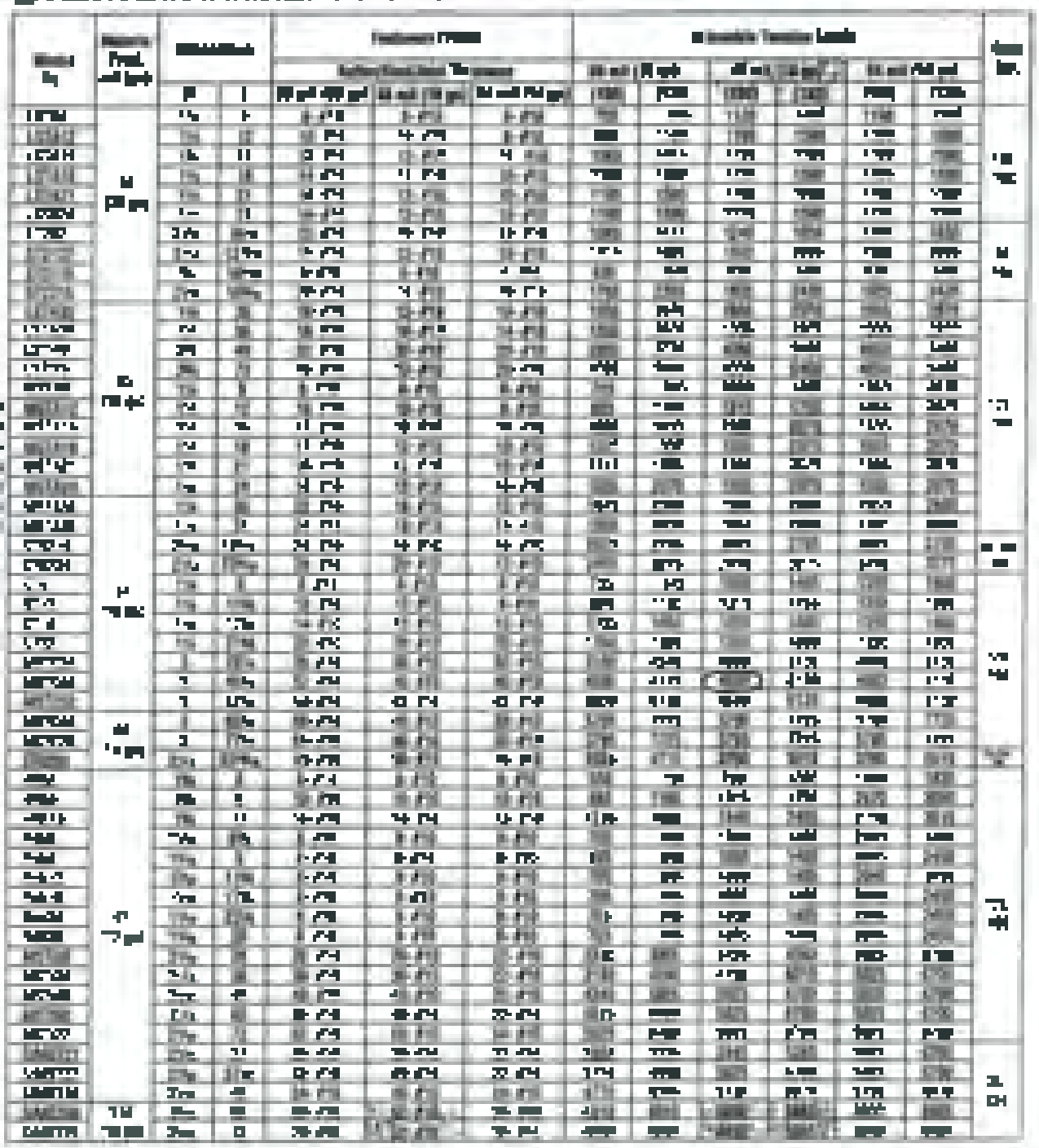


Net Uplift Shear

Maximum Collector Force 4,088 lbs

USE Simpson MSTC40 strap tie w/ T = 9,600 lbs

SIMPSON



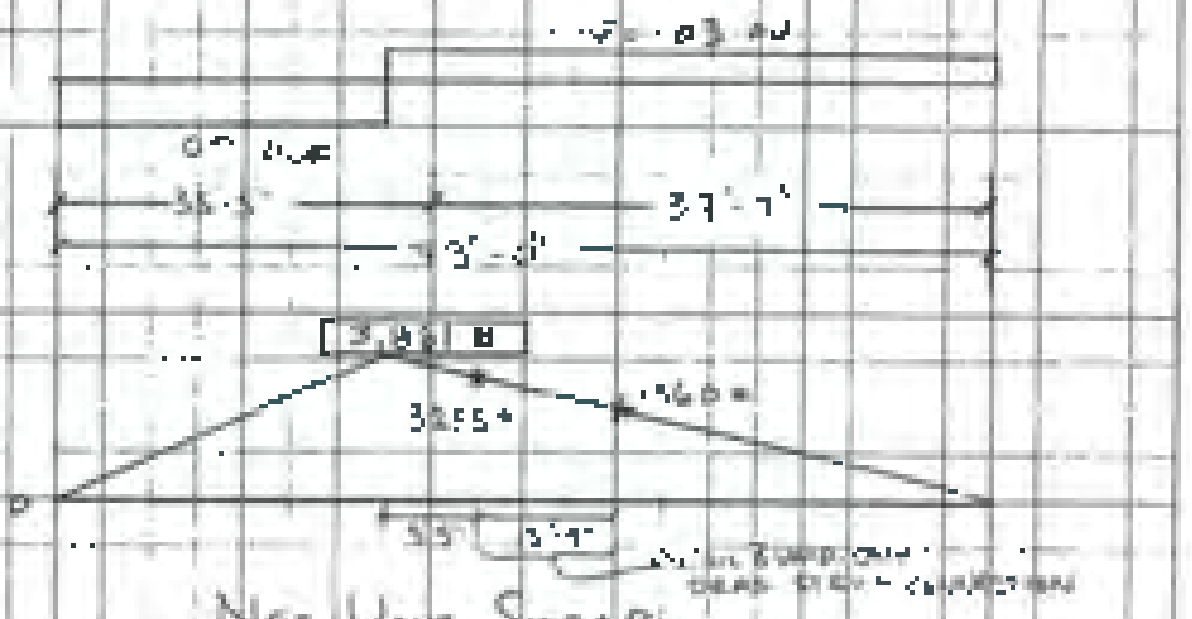
Project Name: NUVEO STATION #53
Project Number: 07502-01
Subject: LATERAL
By: AMB Date: 04/30/07
Checked: Date:
Sheet: 1.81

SHEAR WALL SW-6:

V = 7,530 lbs

V_WALL = 7,530 lbs / 73'-4" = 103 PLF

V_WALL = 7,530 lbs / 35'-5" = 212 PLF



NET UNIT SHEAR

MAXIMUM CONCRETE FORCE 3,821 lb

* DECK SUPPORT ANGLE IS ADEQUATE FOR FORCE TRANSFER

L3x3x1/4 NON BEARING ANGLE
(SEE DIAPHRAGM CHORD DESIGN)