NORTHEAST UTILITIES

THE CONNECTICUT LIGHT AND POWER COMPANY
WESTERN MASSACHUSETTS ELECTRIC COMPANY
HOLYOKE WATER POWER COMPANY
NORTHEAST UTILITIES SERVICE COMPANY
NORTHEAST NUCLEAR ENERGY COMPANY

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May 11, 1984

Docket No. 50-336
A03831

Director of Nuclear Reactor Regulation
Attn: Mr. James R. Miller, Chief
Operating Reactors Branch #3
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Gentlemen:

Millstone Nuclear Power Station, Unit No. 2
Request for Additional Information on IE Bulletin 80-11
Masonry Wall Design

By letter dated February 24, 1984(1), the NRC Staff requested that Northeast Nuclear Energy Company (NNECO) supply additional information on our December 3, 1982(2) submittal on Masonry Wall Design.

Accordingly, NNECO hereby provides the Staff with our partial response to the February request for additional information, as attached. Specifically, a partial response to Question 1 and our complete response to Question 2 are included.

NNECO's response to Questions 3 and 4 regarding the use of increased factors for allowable stresses and the use of arching action is presently being researched. NNECO anticipates our response to these items will be complete by September, 1984 at which time we will docket this information.

We trust you will find the information and schedule provided herein acceptable.

Very truly yours,

NORTHEAST NUCLEAR ENERGY COMPANY

W. G. Counsil

W. G. Counsil
Senior Vice President

(1) J. R. Miller letter to W. G. Counsil, dated February 24, 1984.

(2) W. G. Counsil letter to R. A. Clark, dated December 3, 1982.

A001
1/40 Add: IE/DEPER/EGCB-1
IE/DEPER/EGCB/S-2
NRR/DE/MEB-2

Docket No. 50-336

Attachment

Millstone Nuclear Power Station, Unit No. 2

Request for Additional Information

on IE Bulletin 80-11

Masonry Wall Design

May, 1984

Question 1: With reference to the reinforcement in masonry walls, the ACI 531-79 Code (1) specifies that the minimum area of reinforcement in a wall in each direction, vertical or horizontal, shall be 0.0007 (0.07%) times the gross cross-sectional area of the wall and that the minimum total area of steel, combined vertical and horizontal, shall not be less than 0.002 (0.2%) times the gross cross-sectional area. Clarify whether the reinforced walls at this plant meet the above requirements. It should be noted that the horizontal reinforcement is installed to satisfy the minimum reinforcement requirement for a reinforced wall.

If the joint reinforcement is used to resist tension in the walls meeting the above minimum requirements, it should follow the working stress design method which limits its (Code) allowable to 30 ksi. Please clarify whether this requirement has been satisfied. If this requirement is not satisfied, identify all affected walls along with the calculated stress value for each wall and indicate specific actions planned to correct this situation.

Indicate if there are any walls that may have been qualified using the tensile resistance of the joint reinforcement but not satisfying the minimum steel requirements. It should be noted that the NRC, at present, does not approve the use of joint reinforcement to qualify this type of wall. (See attached staff position). In view of this, indicate all walls belonging to this category and your intended specific actions to bring these walls in compliance with the staff position.

Response: At Millstone Unit No. 2 there were fifty-seven (57) reinforced masonry walls that were evaluated under IE Bulletin 80-11, Masonry Wall Design. Joint reinforcement at Millstone Unit No. 2 (Dur-o-wall) was not considered to resist tension in the above mentioned walls. The question of the minimum area of reinforcement present in each wall is still under investigation and a response will be forthcoming when the investigation is complete. NNECO intends to provide the Staff with this information during September of 1984.

Question 2: With respect to tornado load (2), specify all walls subject to tornado load (if applicable) and provide a sample calculation (with any explanation necessary to make it understandable). Also indicate how the penetration depth, perforation, and spalling along with the overall structural behavior of the wall were evaluated for a tornado missile impact.

Response: Ten Walls could be subjected to tornado loads at Millstone Unit No. 2. The walls are as follows:

1.32	8.22 (exterior wall)
6.1 (exterior wall)	8.29 (exterior wall)
6.2 (exterior wall)	8.31 (exterior wall)
7.5	10.5
7.12	10.12

The design criteria used to evaluate these walls, specifically the loads and load combinations, were given in Appendix B of the Masonry Wall Design submittal, provided in Reference (2). The load combinations used were D + L + W_t, with W_t being a tornado wind load of 360 mph base.

Sample calculations for three walls are attached. Wall 6.1 is an exterior wall partially blocked by Millstone Unit No. 1 and walls 10.5 and 10.12 are interior walls.

Tornado missiles were not evaluated as part of IE Bulletin 80-11, however, they were evaluated in accordance with the criteria in Appendix 5.D of the Millstone Unit No. 2 FSAR. A copy of this appendix is provided for the Staff's convenience. Localized impact as well as penetration effects were evaluated as part of the original design calculations of the plant.

Docket No. 50-336

Millstone Unit No. 2

Sample Calculation for

Masonry Wall 6.1

May, 1984

CALCULATION COVER SHEET

illstone Unit 2 JOB NO 11867-020 CALC. NO. 8011-6.1

SNRC I.E. Bulletin 80-11 Design Verification of Block Walls 6.1

OR Henri Myers DATE 6/23/81

Thomas Larocque DATE 10-2-81 NO. OF SHEETS 6

RECORD OF ISSUES								
DESCRIPTION	BY	DATE	CHKD	DATE	APPRD	DATE	ACCEPT-ANCE	DATE
Initial Calc. Pages 1-	Hm	6/23/81	TL	10/2/81	RWH	10/3/81		

☐ PRELIMINARY CALC.

☐ COMMITTED PRELIMINARY CALC.

☐ SUPERSEDED CALC.

☒ FINAL CALC.

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-6.1 REV. NO. 0

ORIGINATOR Henry Myers DATE 6/23/81 CHECKED TL DATE 10-2-81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 1

DESIGN CALCULATION CHECKLIST

Load Description

	Load Sheet Date	Description
1. Plant Design	10.6.80	N/A
2. Electrical:	10.9.80	N/A
3. Mechanical:	10.8.80	N/A
4. Control Systems:	9.27.80	N/A

Wall Description

1. Drawing Details: SECTION B, S1010
2. Masonry Type: HOLLOW 2 CELLED
3. Wall Thickness: 12" NOMINAL
4. Number of Wythes: 1
5. Rebar Size and Spacing: 2#5 @ 16"
6. Latest Page Revision of Survey Data and Date: △0 3/2/81
7. Boundary Connection Detail:
8. Safety-Related Attachments or II/I: II/I

CALCULATION SHEET

CALC. NO. 8011 6.1 REV. NO. 0

ORIGINATOR Henry Myers DATE 6/23/81 CHECKED TL DATE 10-2-81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 2

Summary of Results

1. Governing Load Combination: D+L+TORNADO * THE WALL FAILS THE DEPRESSURIZATION LOAD! SINCE NO EQUIP IS DAMAGED BY THE WALL IT DOESN'T MATTER THE ROOM IS NO LONGER PRESSURIZED.

2. Boundary Condition Assumed: SIMPLE

3. Horizontal or Vertical Span Checked: VERT

4. In-Plane Shear Strain vs. Allowable: $\frac{.007-.0062}{17} = .00005$ in/in OK

5. Collar Joint Shear Stress vs. Allowable: N/A

6. Tensile Steel Stress vs. Allowable: 25.6 < 54 ksi

7. Masonry Compressive Stress or Interaction Value vs. Allowable: .82 < 1 [illegible]

8. Masonry Shear Stress vs. Allowable: OK BY INSPECTION

9. Cracked or Uncracked Section: CRACKED

10. Boundary Connections, Pass/Fail: PASS * IF THE TOP IS GROUTED

11. Block Pullout, Shear Stress vs. Allowable: N/A

12. Maximum Displacement Elastic Analysis: $\frac{5(.128)(142)^4}{384(1350)(940)} = .53$"

13. Repair, Required/Not Required:

14. Repair Type: N/A

15. Arching, H_a vs. H: N/A

16. Maximum Displacement Arching Action: N/A

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-6.1 REV. NO. 0

ORIGINATOR [illegible] DATE 6/23/81 CHECKED TL DATE 10-2-81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 3

REFERENCES:

- [x] 1) Specification for Re-evaluation of Concrete Masonry Walls - 11867-020-C003, Rev. 1.
- [x] 2) Survey Procedure No. 11867-020-C002, Rev. 5, (Including Survey Data).
- [x] 3) Loading Evaluation Procedure No. 11867-020-C004, Rev. 0, (Including Load Sheets).
- [x] 4) Floor Response Spectra-Auxiliary Building Calculation No. 8011-001, Rev. 0.
- [ ] 5) Floor Response Spectra-Turbine Building Calculation No. 8011-002, Rev. 0.
- [x] 6) Masonry Block Section Properties Calculation No. 8011-003, Rev. 0.
- [ ] 7) Blockwall Pipe Hanger Load Summary Calculation No. 8011-004, Rev. 0.
- [x] 8) Average Floor Response Spectra for Successive Floors In Auxiliary Building - Calculation No. 8011-005, Rev. 0.
- [x] 9) Millstone Unit 2, Seismic Report.
- [ ] 10) Civil Design Aid No. 8011-01-GPD, Rev. 0.
- [ ] 11) Bechtel 'Blockwall' Program CE-020 (Version D).
- [ ] 12) Design Response Spectra, Pgs. E1 & E2 of FSAR for Millstone Unit 2.
- [ ] 13) Damping Values for Wall Attachments, Pg. E3 of FSAR for Millstone Unit 2.

CALCULATION SHEET

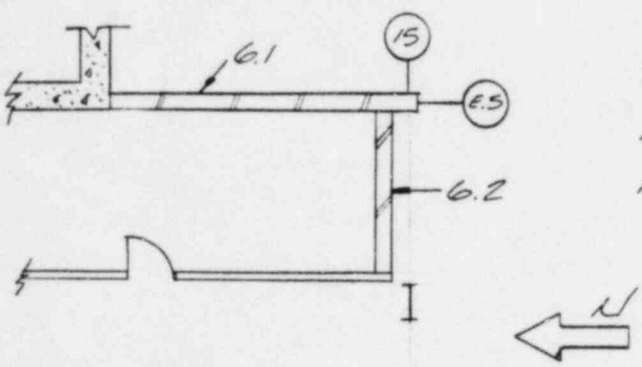
BECHTEL

CALC. NO. 8011-6.1 REV. NO. 0
ORIGINATOR Henry [illegible] DATE 6/16/81 CHECKED [illegible] DATE 6-19-81
PROJECT Millstone Unit 2 JOB NO. 11867-020
SUBJECT [illegible] SHEET NO. 4

WALL DIMENSIONS 20'-4 LENGTH
11'-10 HEIGHT
12" NOMINAL THK, 2 CELLED BLOCK
REINFORCING - 2#5 @16"

LOADING COMBINATIONS
D+L+OBE
D+L+SSE
$D+L+W_t$ WHERE W_t IS TORNADO,
DEPRESSURIZATION,
TORNADO + 1/2 DEPRESSURIZATION,
OR WIND

NO EXTERNAL LOADS FROM ATTACHMENTS



THE DOORS FOR THIS SWITCHGEAR ROOM ARE PRESSURE DOORS ∴ THE ROOM IS NOT VENTED.

PLAN EL 56'-6"

THE PRESSURE ON WALL 6.1 DUE TO DIRECT WIND (WESTERLY) IS 1.84 psi. THE PRESSURE DUE TO DEPRESSURIZATION IS 3 PSI (EASTERLY)

THE MAXIMUM LOAD THE 12" WALL WITH 2#5@16" CAN TAKE AND PRECLUDE CRACKING IS

$$\frac{6\sqrt{1330} \times 4597}{5.46 \times 1.8} \times \frac{8}{142^2} \times \frac{1}{6(11.625)} = .6\,psi$$

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-6.1 REV. NO. 0

ORIGINATOR [illegible] DATE 6/16/81 CHECKED TL DATE 6-19-81

PROJECT [illegible] JOB NO. 11867-020

SUBJECT [illegible] SHEET NO. 5

THE WALL WILL CRACK UNDER EITHER LOAD
CRACKED SECTION
CHECK MAXIMUM STRESSES DUE TO:

1) 1.84 PSI

$w = 1.84 \times 6 \times 11.625 = .128\ K/IN$

$M = wl^2/8 = .128(142)^2/8 = 322.6^{\text{IN-K}}$

$\sigma_M = \frac{322.6 \times 2.4}{940} = .824\ ksi < .85 f'_m = 1.148\ ksi$

$\sigma_{ST} = \frac{322.6 \times (7.73 - 4.31) \times 21.8}{940} = 25.6\ ksi < .9F_y = 54\ ksi$

2) 3 PSI

$w = 3 \times 6 \times 11.625 = .209\ K/in$

$M = wl^2/8 = .209(142)^2/8 = 527.4^{\text{K-IN}}$

$\sigma_M = \frac{527.4 \times 2.4}{940} = 1.346\ ksi > .85 f'_m = 1.148\ ksi$ NG

$\sigma_{ST} = \frac{527.4 \times (7.73 - 4.31) \times 21.8}{940} = 41.8\ ksi < 54\ ksi$

THE WALL FAILS, HOWEVER THE FAILURE WOULD BE AWAY FROM SR EQUIPMENT.

CHECK THE WALL UNDER EARTHQUAKE LOADS

CALCULATION SHEET

BECHTEL

CALC. NO. B011-6.1 REV. NO. 0

ORIGINATOR Henry Myers DATE 6/16/81 CHECKED TL DATE 6-19-81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT 16 Building Boil SHEET NO. 6

INPUT FOR BLOCKWALL PROGRAM

$w = \frac{.0111 \times 69.75}{15.625} = .049^{k}/in$

ASSUME THERE ARE 2#5@16", REF CALC B011-003 PAGE 6A FOR SECTION PROPERTIES

ALLOWABLE STRESSES

OBE		SSE
335	AFMA	.504
.593	AFMB	1.114
.141	AAFGT	.177
.254	AFGT	.254
32	AFYT	54
32	AFYC	54
.067	ASH	.084

ALLOWABLE AXIAL COMPRESSION

OBE

$.359 - .013(142/11.625)^3 = .335$ PSI

SSE

$.546 - .02(142/11.625)^3 = .504$ PSI

THE WALKDOWN INDICATES NO ATTACHMENTS ∴ MASS PTS ARE 0.

DET. MAX AXIAL LOAD DUE TO A VERTICAL EARTHQUAKE

$P = .049(142)(1 + 2/3(.604)) = 9.8^{k}$ SSE $a = a_{RIGID_{AVE}}$ AT 54'-6

$P = .049(142)(1 + 2/3(.315)) = 8.4^{k}$ OBE

RESULTS REF COMPUTER OUTPUT # T60160-1

OBE

```
***SEISMIC MOMENT=     72.3KIPS.IN

                    *****RESULTS OF ANALYSIS*****
MASONRY COMPRESSIVE BENDING STRESS=     .1846KSI        ALLOWABLE =      .593KSI
MASONRY AXIAL COMPRESSIVE STRESS=     .0110KSI          ALLOWABLE =      .335KSI
TENSILE STEEL STRESS=    3.7599KSI                      ALLOWABLE =    32.000KSI
COMPRESSIVE STEEL STRESS=     .0000 KSI                 ALLOWABLE =    32.000KSI
MASONRY SHEAR STRESS=     .0040KSI                      ALLOWABLE =      .067KSI
MAXIMUM DEFLECTION =   .022577 IN.
INTERACTION VALUE= .3
```

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-6.1 REV. NO. 0

ORIGINATOR Henry Moon DATE 6/16/81 CHECKED TL DATE 6-19-81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT IE Bulletin 8011 SHEET NO. 7

SSE REF COMPUTER OUTPUT # T60160-3

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***SEISMIC MOMENT-    118.3KIPS.IN

                    *****RESULTS OF ANALYSIS*****
MASONRY COMPRESSIVE BENDING STRESS=     .3022KSI          ALLOWABLE =     1.114KSI
MASONRY AXIAL COMPRESSIVE STRESS=     .0128KSI            ALLOWABLE =      .504KSI
TENSILE STEEL STRESS=    9.4294KSI                        ALLOWABLE =    54.000KSI
COMPRESSIVE STEEL STRESS=     .0000 KSI                   ALLOWABLE =    54.000KSI
MASONRY SHEAR STRESS=     .0066KSI                        ALLOWABLE =      .084KSI
MAXIMUM DEFLECTION =   .036961 IN.
INTERACTION VALUE= .3
```

CHECK BOUNDARY CONNECTIONS

AT BOTTOM SHEAR IN MORTAR

$M = wl^2/8 = 118.3 \therefore w = .047^{k/in}$

$V = wl/2 = 3.33^k$

$\tau = V/A = 3.33/356.8 = .009\ ksi < .02\ ksi$

THE TOP FRAMES INTO A STEEL BEAM - ANGLES WELDED TO STEEL ~EVERY OTHER BLOCK, HOWEVER, THERE IS NO GROUT BETWEEN THE STEEL AND MASONRY - FIX TOP CONNECTION.

Millstone Unit No. 2

Sample Calculation for

Masonry Walls 10.5 and 10.12

May, 1984

CALCULATION COVER SHEET

Millstone Unit 2 JOB NO. 11867-020 CALC. NO. 8011- 10.5 | 10.12

USNRC I.E. Bulletin 80-11 Design Verification of Block Walls 10.5/10.12

TOR [signature] DATE 8.7.81

R [signature] DATE 7 Aug. 81 NO. OF SHEETS 75

RECORD OF ISSUES

DESCRIPTION	BY	DATE	CHKD	DATE	APPRD	DATE	ACCEPT-ANCE	DATE
Initial Calc. Pages 1- 74	[signature]	8.7.81	R. Man	7 Aug. 81	[signature]	8/8/81		

☐ PRELIMINARY CALC.

☐ SUPERSEDED CALC.

☐ COMMITTED PRELIMINARY CALC.

☒ FINAL CALC.

BECHTEL

CALCULATION SHEET

CALC. NO. 8011 - 10.5 | 10.12 REV. NO. 0

ORIGINATOR J. E. Wat DATE 7.14.81 CHECKED R. Marrm DATE 21 JULY 81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 1

DESIGN CALCULATION CHECKLIST

Load Description

	10.5 , 10.12 Load Sheet Date	Description
1. Plant Design	10/6/80 , 12/16/80	NONE , DRAIN PIPE
2. Electrical:	12/3/80 , 12/3/80	CABLE TRAY , CONDUITS
3. Mechanical: * NOTE: BELOW	3/4/81 , 3/5/81	NONE
4. Control Systems:	9/27/80 , 9/20/80	NONE

5. TORNADO DEPRESSURIZATION & 360 MPH WIND (FSAR)

Wall Description

1. Drawing Details: 25203 – 51006, 51010, 54046, 51013, 11079, 51126, 11075
2. Masonry Type: 8" x 8" x 16" BLOCK, 2 CELL, GROUT @ REBAR
3. Wall Thickness: 8" (NOMINAL) - 7.625"
4. Number of Wythes: 1
5. Rebar Size and Spacing: 1 #5 @ 16"
6. Latest Page Revision of Survey Data and Date: WALL 10.5 : 11/24/80 REV. 1; WALL 10.12 : 11/3/80 REV. 1
7. Boundary Connection Detail: 25203 - 11079 (5) NOTE: FIELD VERIFICATION TOP BOUNDARY DOES NOT EXIST
8. Safety-Related Attachments or II/I: BOTH

* NOTE: WALL HAS NO JET IMPINGEMENT
RE: GROUP MEETING #10 NOTES (6.2.81)

GPD-2706 8/76 (ED-69)

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-105/10.12 REV. NO. 0

ORIGINATOR J. E. W[illegible] DATE 7.14.81 CHECKED R. Mauri DATE 21 JULY 81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 2

Summary of Results (UNREPAIRED)

1. Governing Load Combination: $D + L + W_T$

2. Boundary Condition Assumed: SIMPLY SUPPORTED

3. Horizontal or Vertical Span Checked: VERTICAL

4. In-Plane Shear Strain vs. Allowable: $5.625(10)^{-5}$ vs 0.0008 (OBE); $8.125(10)^{-5}$ vs 0.0013 (SSE) OK

5. Collar Joint Shear Stress vs. Allowable: N/A

6. Tensile Steel Stress or ~~Masonry Tensile Stress~~ vs. Allowable: 133.72 ksi vs. 54.0 ksi NG

7. Masonry Compressive Stress or ~~Interaction Value~~ vs. Allowable: 2.97 ksi vs. 1.11 ksi NG

8. Masonry Shear Stress vs. Allowable: 0.056 vs. 0.084 OK

9. Cracked or Uncracked Section: CRACKED

10. Boundary Connections, Pass/Fail: STRESSES EXCEEDED

11. Block Pullout, Shear Stress vs. Allowable: N/A

12. Maximum Displacement Elastic Analysis: STRESSES EXCEEDED

13. Repair, Required/Not Required: REQUIRED

14. Repair Type: DECREASE VERTICAL SPAN

15. Arching, H_a vs. H: N/A (CONCRETE BOUNDARY TOO FLEXIBLE)

16. Maximum Displacement Arching Action: N/A

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-16.5|10.12 REV. NO. 0

ORIGINATOR J. E. Web DATE 7.14.81 CHECKED R. Mann DATE 21 JULY 81

PROJECT Millstone Unit 2 JOB NO. 11867-020

SUBJECT I.E. Bulletin 80-11 SHEET NO. 3

REFERENCES:

- [x] 1) Specification for Re-evaluation of Concrete Masonry Walls - 11867-020-C003, Rev. 1.
- [x] 2) Survey Procedure No. 11867-020-C002, Rev. 5, (Including Survey Data).
- [x] 3) Loading Evaluation Procedure No. 11867-020-C004, Rev. 0, (Including Load Sheets).
- [x] 4) Floor Response Spectra-Auxiliary Building Calculation No. 8011-001, Rev. 0.
- [ ] 5) Floor Response Spectra-Turbine Building Calculation No. 8011-002, Rev. 0.
- [x] 6) Masonry Block Section Properties Calculation No. 8011-003, Rev. 0.
- [ ] 7) Blockwall Pipe Hanger Load Summary Calculation No. 8011-004, Rev. 0.
- [ ] 8) Average Floor Response Spectra for Successive Floors In Auxiliary Building - Calculation No. 8011-005, Rev. 0.
- [x] 9) Millstone Unit 2, Seismic Report.
- [x] 10) Civil Design Aid No. 8011-01-GPD, Rev. 0.
- [x] 11) Bechtel 'Blockwall' Program CE-020 (Version D).
- [ ] 12) Design Response Spectra, Pgs. E1 & E2 of FSAR for Millstone Unit 2.
- [ ] 13) Damping Values for Wall Attachments, Pg. E3 of FSAR for Millstone Unit 2.

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Wa[illegible] DATE 7.14.81 CHECKED R. Mann DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 80-11 SHEET NO. 4

WALL 10.5, 10.12 TURBINE @ ELEV. 45'-0

WALL DIMENSIONS:

LENGTH: 131'-0"
HEIGHT: VARIES, MAX. 128"
THICKNESS: 8" NOMINAL
REBAR: 1 #5 @ 16"

SECTION PROPERTIES:

AS = 0.62	YCU = 3.06
ASP = 0.0	TTU = 3.32
DS = 2.56	YCCR = 1.25
DP = 0.0	TTCR = 5.13
H = 7.625	AAXIAL = 324.8
B = 45.75	ASHEAR = 156.1
IUCR = 770.4	AC = 55.8
ICR = 118.1	UNIT WEIGHT = .0076 K/IN/16" BLOCK

ALLOWABLE AXIAL COMPRESSIVE STRESS - AFMA

OBE - $359 - 0.013\left(\frac{107.5}{7.625}\right)^3 = 323$ psi = .323 ksi

SSE - $540 - 0.02\left(\frac{107.5}{7.625}\right)^3 = 484$ psi = .484 ksi

GPD-2706 8/76 (ED-69)

CALCULATION SHEET

BECHTEL

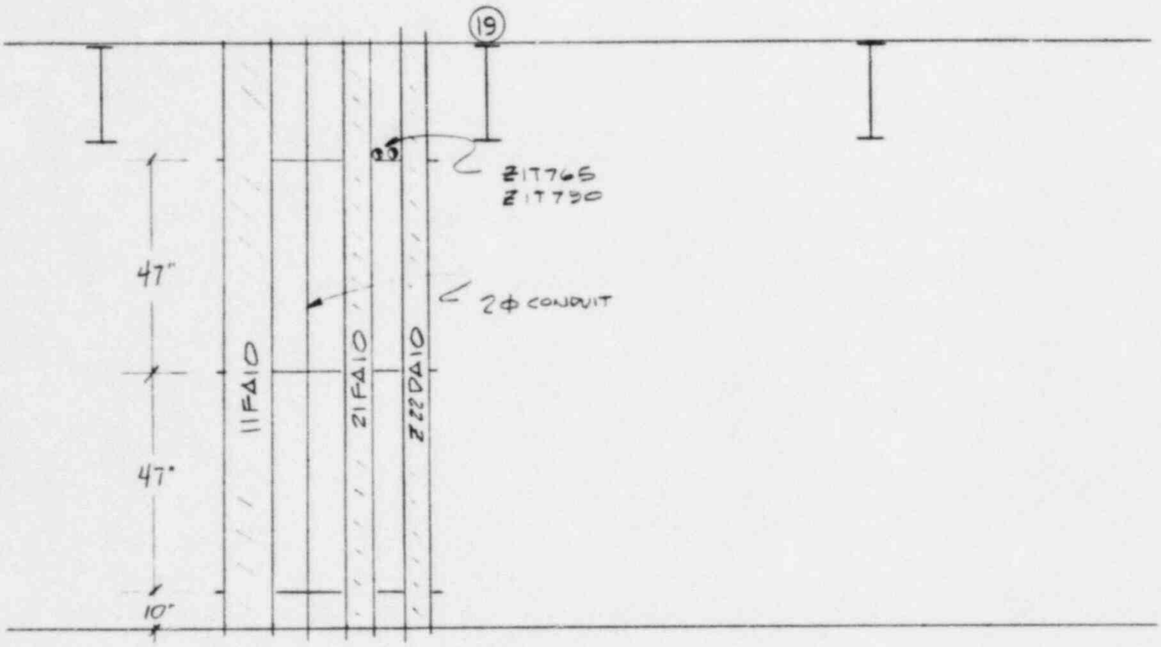
CALC. NO. 801-10.5/10.12 REV. NO. 0

ORIGINATOR J. Eric Webb DATE 7.14.81 CHECKED R. Mann DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 5

PICK WORST '6t' SECTION



NOTE - THIS LOADING CASE WAS PICKED AS WORST '6t' SECTION. ALTHOUGH LOAD DUE TO STAIRWAY (NORTH OF COLUMN 18) WOULD PROBABLY BE WORST LOADED SECTION, THE WALL IN VICINITY OF STAIRWAY IS NOT SAFETY RELATED. THEREFORE FAILURE DUE TO STAIRWAT WOULD NOT HARM S/R ITEMS.

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10 S|10.12 REV. NO. 0

ORIGINATOR J. E. West DATE 7·14·81 CHECKED R. Mann DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 6

CHECK LOADING CONDITIONS

D + L + E'

ACCELERATIONS — SSE

REFERENCES: 1. - DWG. 51006 SEC B
2. - SHEET 3, REF. 4

USE CALCULATION of AVERAGE ACCELERATIONS (SHEET NO. 7)

UNCRACKED (β = 2%) PEAK = 4.383
ZPA = .530

CRACKED (β = 7%) PEAK = 2.23
ZPA = .53

LOADS FOR '6t' SECTION

Z1T765	- RUBBER & FOAM	NO LOAD
Z1T790	- "	NO LOAD
11FA10	- 12" CABLE TRAY (MAX. SPAN 8')	17 #/FT.
21FA10	- 6" CABLE TRAY (MAX. SPAN 8')	13 #/FT.
2 φ CONDUIT	- MAX. SPAN 16'	7 #/FT.
* Z22DA10	- 6" CABLE TRAY (MAX. SPAN 8')	17 #/FT.

* NOTE: CABLE TRAY Z22DA10 WAS FOUND ON AN ADDITIONAL WALKDOWN BY FIELD (REFERENCE I.R. 276) SPAN & LOAD WERE ASSUMED FROM MAXIMUMS of OTHER CABLE TRAYS

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Wat DATE 7.14.81 CHECKED R. Mani DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 7

FIND AVERAGE ACCELERATIONS (ELEV. 45'-0)

SSE β = 8% (UNCRACKED)

	38'-6"	54'-6"	45'-0"	54'-0"
PEAK	3.993	4.564	4.22	4.546
ZPA	.462	.560	.502	.557

AVERAGE PEAK = 4.383
ZPA = .530

SSE β = 7% (CRACKED)

	38'-6"	54'-6"	45'-0"	54'-0"
PEAK	2.03	2.319	2.147	2.310
ZPA	.46	.562	.501	.559

AVERAGE PEAK = 2.23
ZPA = .53

NOTE: ACCELERATION VALUES FOR 38'-6" AND 54'-6" ARE FROM REFERENCE # 4.

CALCULATION SHEET

BECHTEL

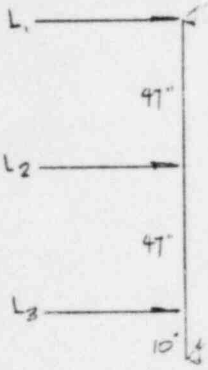
CALC. NO. 8011 - 10.5|10.12 REV. NO. 0

ORIGINATOR J E W[illegible] DATE 7 14 81 CHECKED Q Mam DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867 - 020

SUBJECT I/E BULLETIN 8011 SHEET NO. 8

LOADS AT EACH SUPPORT



$$L_1 = \left[\frac{47"}{2(12"/')} + \frac{8'}{2}\right]\left(17+17+13\ \#/ft.\right) + \left[\frac{47"}{2(12"/')} + \frac{16'}{2}\right]\left(7\ \#/ft\right) = .350\ k$$

$$L_2 = \left[\frac{47"}{12"/'}\right]\left(17+17+13+7\ \#/ft.\right) = .212\ k$$

$$L_3 = .350\ k$$

AXIAL LOAD

DUE TO WALL

$$WT / LENGTH = \left[\frac{45.75"}{15.625"}\right]\left(.0076\ k/in\right) = .0223\ k/in$$

$$℄\ DEAD\ WEIGHT = .0223\ k/in \left(\frac{128"}{2}\right) = 1.43\ k$$

$$AXIAL\ FORCE = 1.43\ k\left[1 + 2/3(.53)\right] = 1.93\ k$$

DUE TO LOADS

$$LOAD = 2(.350) + .212 = .912\ k$$

$$AXIAL\ FORCE = .912\ k\left[1 + 2/3(.53)\right] = 1.23\ k$$

$$TOTAL\ AXIAL\ FORCE = 1.93\ k + 1.23\ k = \underline{3.16\ k}$$

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-0.5/10.12 REV. NO. 0

ORIGINATOR J. E. West DATE 7.14.81 CHECKED R. Mann DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 9

SHEAR & MOMENT

ACCELERATED LOADS

$L_1 = .350^k(4.38) = 1.533^k$

$L_2 = .212^k(4.38) = .929^k$

$L_3 = 1.533^k$

$$R_{TOP} = \frac{1.53^k(10") + .93^k(57") + 1.53^k(104")}{128"} = 1.78^k$$

$R_{BOTTOM} = 2.22^k$

$\therefore$ MAX SHEAR = 2.22 k

$M_{\mathbb{C}} = 2.22^k(64") - 1.533^k(54") - .929^k(7") = 52.80$ k·in

$M_{\mathbb{C}} = 52.80$ k·in

COMPUTER RUN ATTACHED

AXIAL LOAD = 3.16 k

SHEAR = 2.22 k

MOMENT = 52.80 k

$\therefore$ SECTION FAILS (D + L + E')

Calc. No. 8011-10.5/10.17 Rev. No. -
Millstone Unit 2 Job No. [illegible]
Reevaluation of Concrete Masonry Walls
In Response to NRC I&E Bulletin 80-11
Vol. No. ______ Sht. No. 10

Orig. [signature] Date 7·17·81
Chk. [signature] Date 21 JULY 81

```
@ADD,P 10-5S.
EOF:13 SCAN:10
0:>EXIT
LINES:13 FIELDATA
>@ADD,P DEMAND.


@ADD,P DEMAND.
FURPUR 28R1-01 U1 E35 SL74R1 07/17/81 05:46:40
READY
READY
CASE UPPER ASSUMED
ED 15R2-4 FRI-07/17/81-05:46:49-(0,1)
EDIT
@ASG,A PROJECT*BLOCK.
 ************************************************************************


                    **** DATA FROM INTERNAL STORAGE****


                    ****BLOCK WALLS PROGRAM***
                    **** VERSION 6  2/12/81


                    ****QUESTIONS SHOULD BE ADDRESSED TO****
                    **** E. AKKOUSH   GPD X 3196        ****
                    **** S. CLOSE   GPD X 3196          ****
                    **** T. JOSEPH   GPD X 3192         ****


                    **********************
                    *                    *
                    *  UNITS KIPS INCHES *
                    *                    *
                    **********************


                    **** PROB. TITLE: 10-5(SSE)  ****
```

Calc. No. 8011-105/10.12 [illegible]
Millstone Unit 2 [illegible]
Reevaluation of Concrete Masonry Walls
In Response to NRC I&E Bulletin 80-11
Vol. No. ________ Sht. No. 11

Orig. [signature] Date 7.17.81
Chk. [signature] Date 21 July 81

**** SECTION PROPERTIES ****

AS= .62	ASP= .00	DS= 2.56	DP= .00
H= 7.6	L=128.0	B= 45.7	D= 3.8

INPUT FOR STRESS CALCULATION

IUCR=UNCRACKED INERTIA= 770.40
ICR=CRACKED INERTIA= 118.10
YCU=DIST. TO EXTREME FIBER IN COMP.(UNCRACKED)= 3.060
YTU=DIST. TO EXTREME FIBER IN TENSION(UNCRACKED)= 3.320
YCCR=DIST. TO EXTREME FIBER IN COMP.(CRACKED)= 1.250
YTCR=DIST. TO EXTREME FIBER IN TENSION(CRACKED)= 5.130
AAXIAL=EFFECTIVE AXIAL AREA= 324.80
ASHEAR=EFFECTIVE SHEAR AREA= 156.10
AC=TRANSFORMED COMPRESSIVE AREA OF SECTION= 55.80

**** MATERIAL PROPERTIES ****

YOUNG MODULUS= 1350.00
AVERAGE WT. PER UNIT LENGTH= .02170000
MODULAR RATIOS= 21.9 1.8
COMPRESSIVE STRENGTH OF MASONRY= 1.3
COMPRESSIVE STRENGTH OF GROUT= 1.8
YIELD OF REINFORCING STEEL = 60.0

** SSE SEISMIC CONSIDERATION FOR THIS PROBLEM **

FLOOR RESPONSE SPECTRUM DEFINITION

ADDITIONAL WEIGHTS AT MASS PTS. ARE:

ADDW1= .000 ADDW2= .000 ADDW3= .000

BEAM MODEL IS S.S AT BOTH ENDS

INPUT ALLOWABLE STRESSES

ALLOWABLE MASONRY AXIAL COMP. STRESS= .48400
ALLOWABLE MASONRY COMP.BENDING STRESS= 1.11400
ALLOWABLE GROUT TENSION STRESS= .17700
ALLOWABLE GROUT TENSION FOR FREQENCY CALCS.= .25500
ALLOWABLE STEEL STRESS IN TENSION=54.00000
ALLOWABLE STEEL STRESS IN COMPRESSION=54.00000
ALLOWABLE SHEAR STRESS IN MASONRY= .08400

*** FREQENCIES ARE *** 13.050 51.836 110.059

MODAL PARTICIPATION FACTORS ARE .07 .00 .01

*** ACCELERATIONS ARE *** 3.703 .530 .530

***SEISMIC MOMENT= 169.6KIPS.IN

SECTION IS CRACKED, IS A NEW RESPONSE
SPECTRA FOR THE CRACKED SECTION TO BE INPUT ?
(1=YES,0=NO)
INPUT FLOOR RESPONSE SPECTRUM
SPECTRUM INPUT IS A 2-D ARRAY DEFINING FREQUENCY INCPS VS ACCELERATION IN G
TYPE *N* NUMBER OF POINT USED TO DESCRIBE THE CURVE ?
INPUT 6 SET OF FREQUENCY VS ACCELERATIONS ENTRIES EACH ON A NEW LINE

.02	2.23
4.00	2.23
8.07	2.23
36.30	.53
75.00	.53
1000.00	.53

Calc. No. 8011-10.5/10.12 Rev. No. 0
Millstone Unit 2 Job No. 11867-020
Reevaluation of Concrete Masonry Walls
In Response to NRC I&E Bulletin 80-11
Vol. No. ______ Sht. No. 13

Orig. [signature] Date 7.17.81
Chk. R. Man. Date 21 JULY 81

*** FREQENCIES ARE *** 5.355 21.270 45.160

MODAL PARTICIPATION FACTORS ARE .07 .00 .01

*** ACCELERATIONS ARE *** 2.230 1.435 .530

***SEISMIC MOMENT= 102.2KIPS.IN
** COMP. BENDING MASONRY STRESS GREATER THAN ALLOWABLE**
TENSION STEEL OVERSTRESSED

*****RESULTS OF ANALYSIS*****

MASONRY COMPRESSIVE BENDING STRESS= 1.6405KSI	ALLOWABLE = 1.114KSI
MASONRY AXIAL COMPRESSIVE STRESS= .0097KSI	ALLOWABLE = .484KSI
TENSILE STEEL STRESS= 73.8663KSI	ALLOWABLE = 54.000KSI
COMPRESSIVE STEEL STRESS= .0000 KSI	ALLOWABLE = 54.000KSI
MASONRY SHEAR STRESS= .0287KSI	ALLOWABLE = .084KSI

MAXIMUM DEFLECTION = 1.433706 IN.
INTERACTION VALUE=1.5

DO YOU WANT TO RUN BLOCK WALL AGAIN YES TYPE 1 NO TYPE 0
@BRKPT PRINT$
EOF:178
LINES:178 FIELDATA
>@@SKIP 63

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J E Web DATE 7.14.81 CHECKED R. M DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 14

D + L + E

ACCELERATIONS - OBE

REFERENCES: 1. DWG 51006 SEC B
2. SHEET 3, REF 4

USE ELEV. 54'-6" E-W ACCELERATIONS

UNCRACKED (β = 2%) PEAK = 3.36
EPA = .291

CRACKED (β = 4%) PEAK = 2.17
EPA = .292

LOADS AT EACH SUPPORT

LOADS AT SUPPORTS WILL BE THE SAME AS D+L+E'

∴ FROM PAGE 8 of CALCULATIONS

L_1 = .350 K
L_2 = .212 K
L_3 = .350 K

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J.E. W[illegible] DATE 7.14 81 CHECKED R. Man. DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 15

AXIAL LOAD

FROM SHEET No. 8

AXIAL LOAD = 1.43 k + .912 k = 2.34 k

AXIAL FORCE = 2.34 [1 + 2/3 (.29)] = 2.79 k

SHEAR & MOMENT

ACCELERATED LOADS

L_1 = .350 (3.36) = 1.176 k
L_2 = .212 (3.36) = .712 k
L_3 = 1.176 k

$$R_{TOP} = \frac{1.176^k(10") + .712^k(57") + 1.176^k(104")}{128"} = 1.36\ k$$

R_{BOTTOM} = 1.704 k

$M_{℄}$ = 1.704ᵏ(64") − 1.176 (54") − .712ᵏ (7") = 40.57 k·in

COMPUTER RUN # ATTACHED

∴ SECTION FAILS (D + L + E)

Calc. No. [illegible] Rev. No. 0
Millstone Unit 2 Job No. [illegible]
Reevaluation of Concrete Masonry Walls
In Response to NRC I&E Bulletin 80-11
Vol. No. ______ Sht. No. 16

Orig. [signature] Date 7.17.81
Chk. [signature] Date 21 JULY 81

```
@ADD,P DEMAND.
FURPUR 28R1-01 U1 E35 SL74R1 07/17/81 06:12:25
READY
READY
CASE UPPER ASSUMED
ED 15R2-4 FRI-07/17/81-06:12:40-(0,1)
EDIT
@ASG,A PROJE'T*BLOCK.
 *********************************************************************

                    **** DATA FROM INTERNAL STORAGE****

                    ****BLOCK WALLS PROGRAM***
                    **** VERSION G  2/12/81

                    ****QUESTIONS SHOULD BE ADDRESSED TO****
                    **** E. AKKOUSH   GPD X 3196        ****
                    **** S. CLOSE   GPD X 3196          ****
                    **** T. JOSEPH   GPD X 3192         ****

                    **********************
                    *                    *
                    *  UNITS KIPS INCHES *
                    *                    *
                    **********************

                    **** PROB. TITLE: 10-5(OBE)  ****

                    EXTERNAL LOADS
     AXIAL LOAD(KIPS)    BENDING MOMENT(KIP.IN)    SHEAR FORCE(KIPS)
             2.79                   40.57                 1.70
```

Calc. No. 600-WS [illegible] Rev. No. 0
Millstone Unit 2 Job No. 11867-020
Reevaluation of Concrete Masonry Walls
In Response to NRC I&E Bulletin 80-11
Vol. No. ________ Sht. No. 17

Orig. [signature] Date 7.17.81
Chk. [signature] Date 21 JULY 81

INPUT FOR STRESS CALCULATION

IUCR=UNCRACKED INERTIA= 770.40
ICR=CRACKED INERTIA= 118.10
YCU=DIST. TO EXTREME FIBER IN COMP.(UNCRACKED)= 3.060
YTU=DIST. TO EXTREME FIBER IN TENSION(UNCRACKED)= 3.320
YCCR=DIST. TO EXTREME FIBER IN COMP.(CRACKED)= 1.250
YTCR=DIST. TO EXTREME FIBER IN TENSION(CRACKED)= 5.130
AAXIAL=EFFECTIVE AXIAL AREA= 324.80
ASHEAR=EFFECTIVE SHEAR AREA= 156.10
AC=TRANSFORMED COMPRESSIVE AREA OF SECTION= 55.80

**** MATERIAL PROPERTIES ****

YOUNG MODULUS= 1350.00
AVERAGE WT. PER UNIT LENGTH= .02170000
MODULAR RATIOS= 21.9 1.8
COMPRESSIVE STRENGTH OF MASONRY= 1.3
COMPRESSIVE STRENGTH OF GROUT= 1.8
YIELD OF REINFORCING STEEL = 60.0

** OBE SEISMIC CONSIDERATION FOR THIS PROBLEM **

FLOOR RESPONSE SPECTRUM DEFINITION

F	G
.02	3.36
4.00	3.36
8.07	3.36
36.30	.29
75.00	.29
1000.00	.29

Orig. [signature] Date 7.17.81
Chk. [signature] Date 21 JULY 81

```
**BEAM MODEL IS S.S AT BOTH ENDS**

                    ***INPUT ALLOWABLE STRESSES***
ALLOWABLE MASONRY AXIAL COMP. STRESS=  .32300
ALLOWABLE MASONRY COMP.BENDING STRESS=  .59250
ALLOWABLE GROUT TENSION STRESS=  .14100
ALLOWABLE GROUT TENSION FOR FREQENCY CALCS.=  .25460
ALLOWABLE STEEL STRESS IN TENSION=31.90000
ALLOWABLE STEEL STRESS IN COMPRESSION=31.90000
ALLOWABLE SHEAR STRESS IN MASONRY=  .06650


 *** FREQENCIES ARE ***    13.050    51.836   110.059

***MODAL PARTICIPATION FACTORS ARE***       .07       .00       .01

*** ACCELERATIONS ARE ***      2.818      .291      .291


***SEISMIC MOMENT=    129.1KIPS.IN

SECTION IS CRACKED, IS A NEW RESPONSE
SPECTRA FOR THE CRACKED SECTION TO BE INPUT ?
(1=YES,0=NO)
 INPUT FLOOR RESPONSE SPECTRUM
SPECTRUM INPUT IS A 2-D ARRAY DEFINING FREQUENCY INCPS VS ACCELERATION IN G
TYPE *N* NUMBER OF POINT USED TO DESCRIBE THE CURVE ?
 INPUT   6 SET OF FREQUENCY VS ACCELERATIONS ENTRIES EACH ON A NEW LINE


                    * * C R A C K E D   S E C T I O N * *

                    FLOOR RESPONSE SPECTRUM DEFINITION
                         F                  G
                        .02                2.17
```

Orig. J.E. Webs Date 7.17.81
Chk. H. Mum Date 21 JULY 81

```
*** FREQENCIES ARE ***     5.354    21.265    45.151

***MODAL PARTICIPATION FACTORS ARE***        .07        .00        .01

*** ACCELERATIONS ARE ***      2.170     1.292      .292

***SEISMIC MOMENT=      99.4KIPS.IN
** COMP. BENDING MASONRY STRESS GREATER THAN ALLOWABLE**
**TENSION STEEL OVERSTRESSED**

                         *****RESULTS OF ANALYSIS*****
MASONRY COMPRESSIVE BENDING STRESS=    1.4814KSI          ALLOWABLE =        .593KSI
MASONRY AXIAL COMPRESSIVE STRESS=      .0086KSI           ALLOWABLE =        .323KSI
TENSILE STEEL STRESS=   66.7007KSI                        ALLOWABLE =     31.900KSI
COMPRESSIVE STEEL STRESS=      .0000 KSI                  ALLOWABLE =     31.900KSI
MASONRY SHEAR STRESS=      .0250KSI                       ALLOWABLE =       .066KSI
MAXIMUM DEFLECTION =  1.289819 IN.
INTERACTION VALUE=2.5

 DO YOU WANT TO RUN BLOCK WALL AGAIN     YES TYPE 1    NO  TYPE 0
@BRKPT PRINT$
EOF:178
LINES:178 FIELDATA
>@@SKIP 63
>@FIN
Bechtel Univac T/S: 36-847-127  (RSI)*

Siteid : U29S14

Enter Userid,Charge-to,Program Number
>@@TERM
```

CALCULATION SHEET

BECHTEL

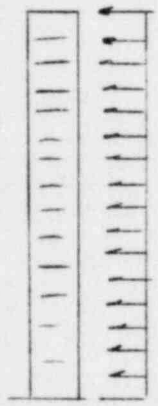
CALC. NO. 8011 - 10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Walo DATE 7.14.81 CHECKED R. Mam DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 20

D + L + W_T



3 psi UNIFORM LOAD

ANALIZE AS UNIFORM LOAD

3 psi (45.75 in) = 137 #/in = .137 K/in

MAX. SPAN = 128"

$$M_{MAX} = \frac{wL^2}{8} = \frac{.137(128)^2}{8} = 280.58 \text{ K}\cdot\text{in}$$

$$V_{MAX} = \frac{wL}{2} = \frac{.137(128)}{2} = 8.77 \text{ K}$$

$$f_{cr} = \frac{Mcn}{I} = \frac{280.58(3.32)(1.8)}{770.4} = 2.18 \text{ ksi} > .255 \text{ ksi}$$

∴ SECTION IS CRACKED

CALCULATION SHEET

BECHTEL

CALC. NO. 8011 - 10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Wek DATE 7·14·81 CHECKED D. Mann DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 21

CHECK ALLOWABLES (CRACKED SECTION)

$$f_{cbm} = \frac{280.58\text{ K·IN}(1.25\text{ IN})}{118.1} = 2.970\text{ ksi} > 1.114\text{ ksi} \quad NG$$

$$f_{ST} = \frac{280.58(5.13 - 2.56)(21.9)}{118.1} = 133.72\text{ ksi} > 54\text{ ksi} \quad NG$$

$$f_{MS} = \frac{V}{A} = \frac{8.77^{K}}{156.1\text{ IN}^2} = 0.056\text{ ksi} < .084\text{ ksi} \quad OK$$

∴ SECTION FAILS $(D + L + W_T)$

NOTE: WORST STRESSES CAUSED BY $D + L + W_T$
REPAIR SHOULD BE MADE FOR THIS
LOAD CASE.

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.R REV. NO. ―

ORIGINATOR [signature] DATE 7.14.81 CHECKED R. Man. DATE 21 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT I/E BULLETIN 8011 SHEET NO. 22

BOUNDARY CONNECTIONS

REFERENCE INFORMATION RESPONSE No. 302

FIELD VERIFICATION THAT TOP ANGLES DO NOT EXIST

∴ TOP BOUNDARY CONNECTION MUST BE REPAIRED

CONCLUSION

WALL NEEDS TO BE REPAIRED. WALL SHOULD BE REPAIRED FOR D+L+WT AND TOP BOUNDARY SHOULD BE REPAIRED.

OPERABILITY CHECK - REF. DWG 11079 & 11075

WALL CANNOT BE EVALUATED FOR ARCHING AS CONCRETE SLAB ABOVE & BELOW WALL ARE ONLY 6½ INCHES THICK.

THEREFORE, "SNAP THROUGH" WILL OCCUR BY INSPECTION, AND ARCHING IS NOT APPLICABLE.

CONCLUSION - WALL FAILS OPERABILITY

CHECK IN-PLANE SHEAR STRAIN

USE AUX. BUILDING N-5 (REF. 9 MILLSTONE UNIT 2, SEISMIC REPORT)

ELEV. 54'-6"

OBE = 0.0045' SSE = 0.0062'

ELEV. 38'-6"

OBE = 0.0036' SSE = 0.0049'

$$\text{OBE STRAIN} = \frac{0.0045 - 0.0036}{54.5 - 38.5} = 5.625(10)^{-5} < 0.0008$$

OK

$$\text{SSE STRAIN} = \frac{0.0062 - 0.0049}{54.5 - 38.5} = 8.125(10)^{-5} < 0.0013 \quad \text{OK}$$

CONCLUSION - IN-PLANE SHEAR STRAIN OK

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Weeks DATE 7·22·81 CHECKED R. Brown DATE 24 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 24

CHECK WALL WITH REPAIR

THERE ARE 5 STRIPS TO BE CHECKED (SEE SHEETS 27 & 28)

STRIP 1 FROM 'STRUDL' RUN SNUMB: T20193-2
STRIP 1 PAGE 27

$M_{MAX} = 50.76$ K·IN

$V_{MAX} = 9.29$ K

$f_{cr} = \frac{Mcn}{I} = \frac{50.76(3.32)(1.8)}{770.4} = .394$ ksi > .255 ksi

∴ SECTION IS CRACKED, USE CRACKED SECTION PROPERTIES

$f_{cbm} = \frac{Mc}{I} = \frac{50.76(1.25)}{118.1} = .537$ ksi < 1.114 ksi OK

$f_{st} = \frac{Mcn}{I} = \frac{50.76(5.13-2.56)(21.9)}{118.1} = 24.2$ ksi < 54 ksi OK

$f_{ms} = \frac{V}{A} = \frac{9.29}{156.1} = .060$ ksi < .084 ksi OK

CONCLUSION WALL REPAIR IS OK FOR THIS STRIP

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Ue[illegible] DATE 7.22.81 CHECKED R. Mani DATE 24JUL'81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

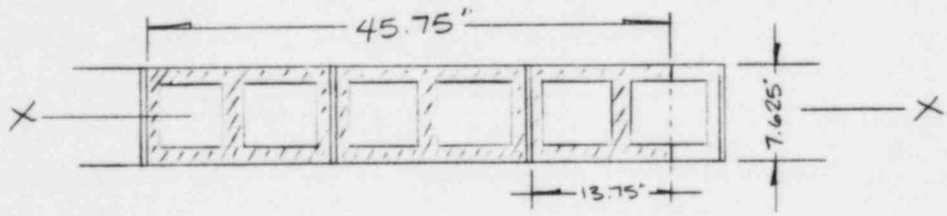
SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 25

CHECK WALL WITH REPAIR

STRIPS 2, 3 & 4 HAD SMALLER SHEAR & MOMENT
SEE RUN #'S T20193-2, T20574-1, & T20883-1

CONCLUSION - STRESSES WILL BE SMALLER THAN STRIP 1 ∴ WALL OK FOR THESE STRIPS.

STRIP 5 - THIS STRIP IS THE HOLLOW BLOCK SECTION ABOVE THE CHANNELS SINCE IT IS ASSUMED THERE IS NO REBAR IN THIS SECTION, NEW SECTION PROPERTIES MUST BE COMPUTED.



$$6t = 6(7.625) = 45.75''$$

$$\text{PARTIAL BLOCK} = 45.75'' - 2\left(\tfrac{3}{8}''\right) - 2(15.625'') = 13.75''$$

$$\text{GROSS BLOCK LENGTH} = 45.75 - 2(3/8'') = 45''$$

$$I_{xx} = \frac{45(7.625)^3}{12} - 5\left[\frac{6.3125(5.125)^3}{12}\right] - \frac{5.4375(5.125)^3}{12}$$

$$I_{xx} = 1247.41 \text{ IN}^4$$

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. -

ORIGINATOR J.E. W[illegible] DATE 7·22·81 CHECKED A. [illegible] DATE 24JUL81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 26

CHECK WALL WITH REPAIR

$$AREA = 2(45")(1.25") + 8(1")(5.125") = 153.5 \text{ in}^2$$

CHECK STRIP 5 AS 20" SIMPLE SUPPORTED BEAM

PRESSURE LOAD = 3 psi = .003 ksi

UNIFORM LOAD ON BEAM = .003 ksi (45.75") = .137 k/in

$$M_{MAX} = \frac{wL^2}{8} = \frac{.137(20)^2}{8} = 6.85 \text{ k·in}$$

$$V_{MAX} = \frac{wL}{2} = \frac{.137(20)}{2} = 1.37 \text{ k}$$

$$f_{cbm} = \frac{Mc}{I} = \frac{6.85\left(\frac{7.625}{2}\right)}{1247.41} = 0.0209 \lesssim 0.020 \text{ ksi} \quad OK$$

$$f_{ms} = \frac{V}{A} = \frac{1.37}{153.5} = .009 \text{ ksi} < 0.020 \text{ ksi} \quad OK$$

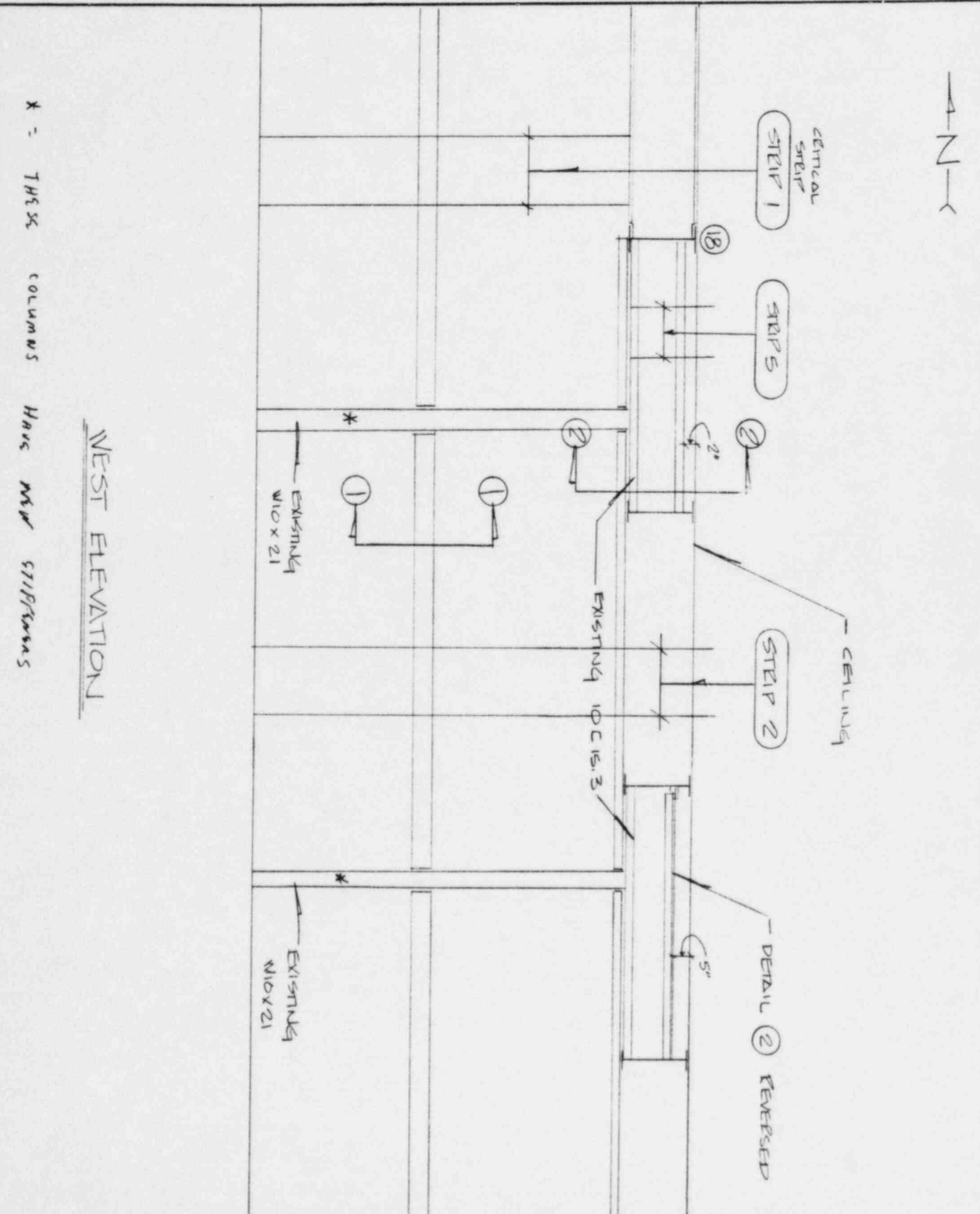
CONCLUSION: WALL OK FOR STRIP 5

∴ WALL OK FOR ALL STRIPS, REPAIR OK

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.E/10.12	REV. NO. 0
ORIGINATOR [signature] DATE 7.27.81	CHECKED R. [signature] DATE 4 AUG. 81
PROJECT MILLSTONE UNIT 2	JOB NO. 11867-020
SUBJECT 8011 BULLETIN - REPAIR	SHEET NO. 27



CALCULATION SHEET

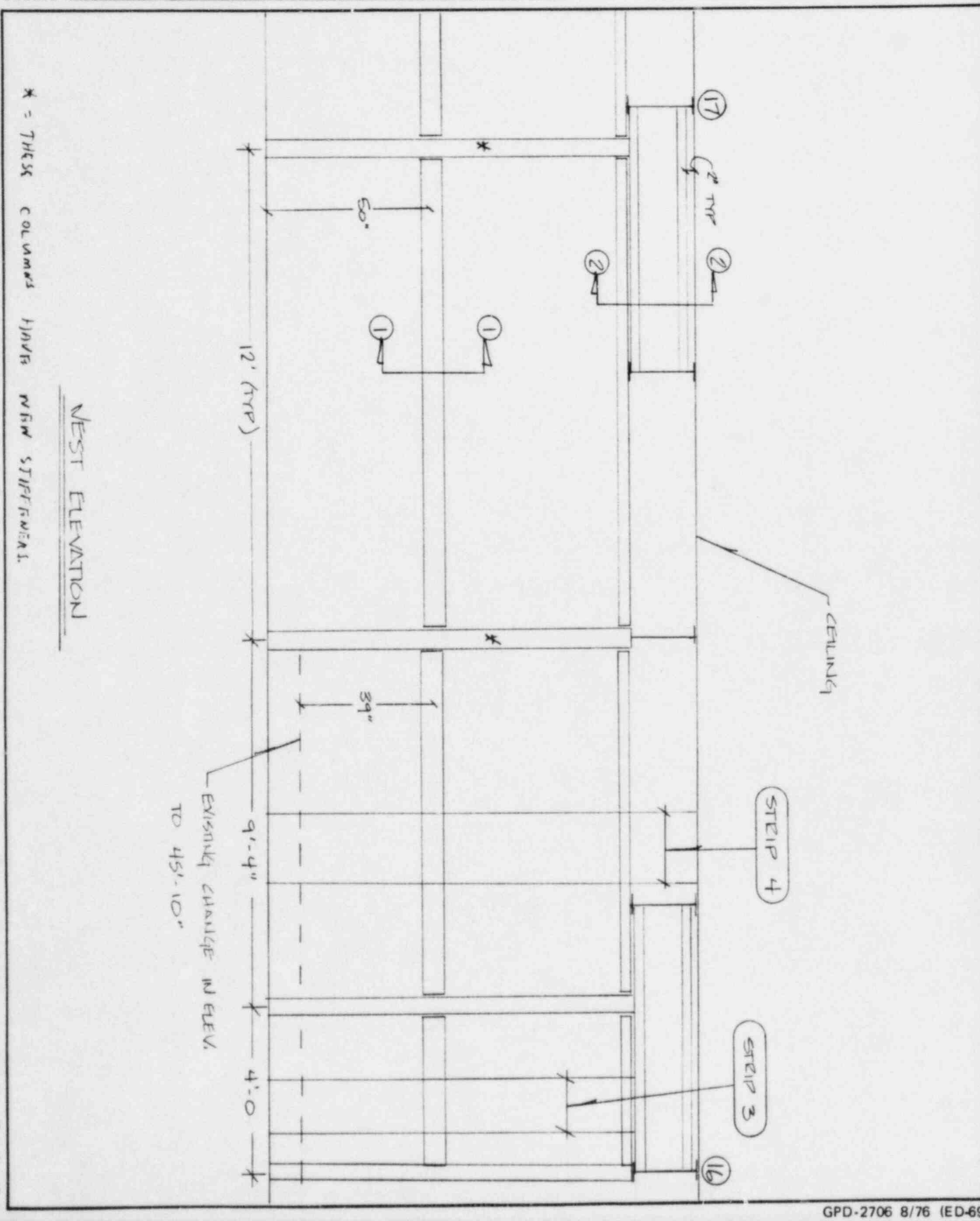
BECHTEL

CALC. NO. 8011-10.5/10.2 REV. NO. 0

ORIGINATOR J. E. Web DATE 7-27-81 CHECKED R. Mani DATE 4 Aug. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 28



CALCULATION SHEET

BECHTEL

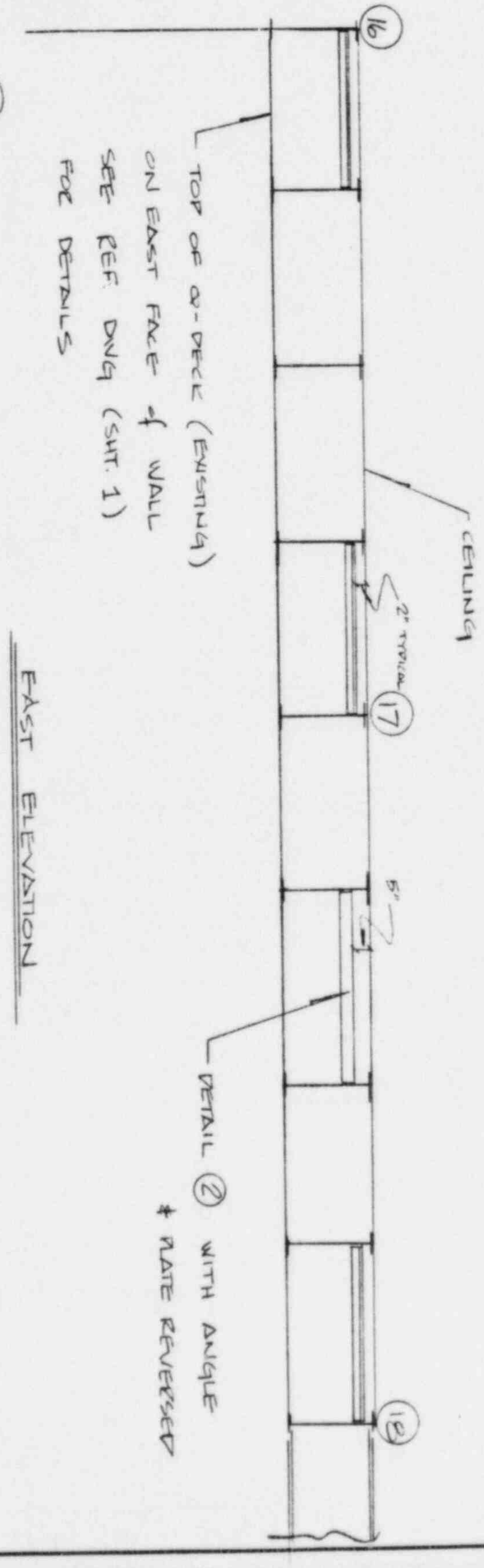
ORIGINATOR J. E. West DATE 7.22.81 CALC. NO. 8011-10.5/10.12 REV. NO. 0

CHECKED R. Mann DATE 4 AUG. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 29

REPAIR of TOP
BOUNDARY (EAST ELEVATION)



CALCULATION SHEET

BECHTEL

CALC. NO. 8011-105/10.2 REV. NO. 0

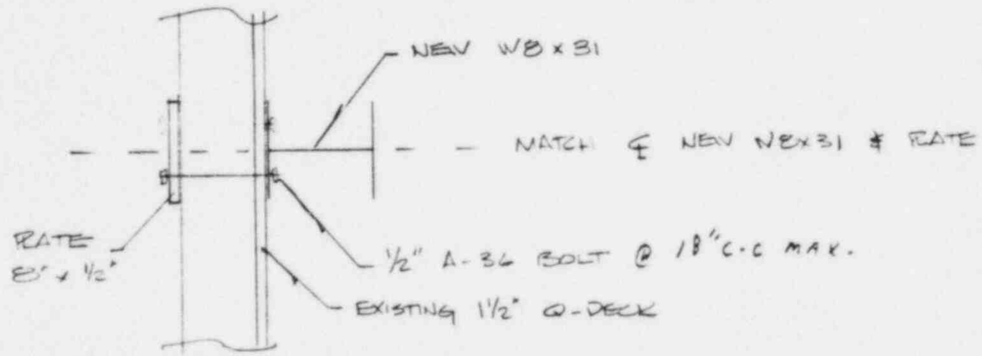
ORIGINATOR [signature] DATE 7.27.81 CHECKED [signature] DATE 4 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 30

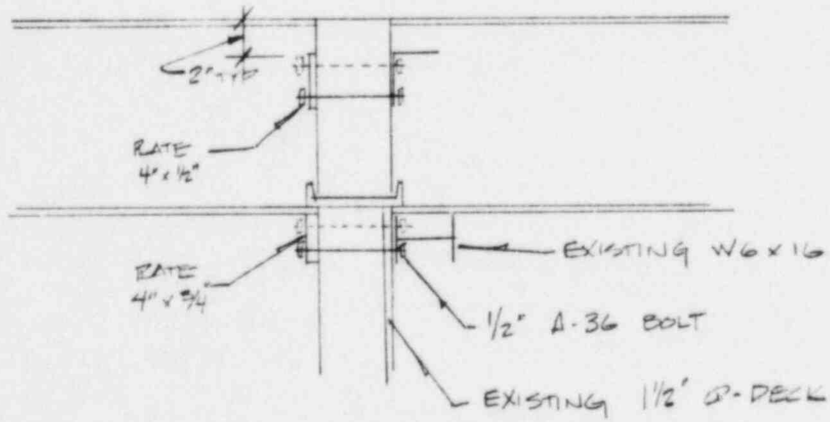
DESIGN REPAIR

DETAILS



DETAIL (1)
N.T.S.

LOOKING NORTH



DETAIL (2)
N.T.S.

LOOKING NORTH

NOTE - AT THE SECTION WHERE THE PLATE & ANGLE ARE SWITCHED, THE 2" TYPICAL MEASUREMENT BECOMES 5".

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5|10.12 REV. NO. 0

ORIGINATOR J.E. Walo DATE 7.22.81 CHECKED R. Marr DATE 24 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 31

DESIGN SECTIONS FOR REPAIR

DESIGN WF BEAM (MIDDLE SUPPORT) DET. #1 SHT. 30

MAX REACTION AT MIDDLE SUPPORT FROM 'STRUDL' RUN SNUMB: T20193-2

REACTION = 9.29 K LABELED STRIP 1 PG. 27

UNIFORM LOAD ON SUPPORT = $\frac{9.29^k}{45.75''}$ = .203 K/IN

DESIGN BEAM FOR MAX. SPAN (12')

$$M_{MAX} = \frac{\omega L^2}{8} = \frac{.203(144)^2}{8} = 526.33 \text{ K·IN}$$

$$V_{MAX} = \frac{\omega L}{2} = \frac{.203(144)}{2} = 14.62 \text{ K}$$

try W8 x 31 $S_x = 27.5 \text{ in}^3$

$$f_b = \frac{M}{S} = \frac{526}{27.5} = 19.1 \text{ KSI} < 21.6 \text{ KSI}$$ OK

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10E|10.12 REV. NO. 0

ORIGINATOR J. E. Wal DATE 7-24-81 CHECKED [illegible] DATE 24 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 32

DESIGN REPAIR

NOTE - SINCE 3 PSL (D+L+W_T) IS IN WESTERLY DIRECTION SOMETHING IS NEEDED TO TRANSFER LOAD TO NEW W8x31.

try PL 8" x 1/2"

$$S_Y = \frac{I}{c} = \frac{bh^2}{6} = \frac{8(.5)^2}{6} = .33 \text{ in}^3$$

$f_{b_{all}}$ = 27 ksi (MINOR AXIS BENDING)

$$M_{all} = f_b(S_Y) = 27(.33) = 8.9 \text{ k-in}$$

MAX. SPACING of BOLTS

$$L = \left(\frac{M(8)}{\omega}\right)^{1/2} = \left(\frac{8.9(8)}{.203}\right)^{1/2} = 18.72"$$ USE 18" MAX. SPACING

CHECK TENSILE STRESS IN BOLTS

MAX. LOAD ON BOLTS = .203 k/in (18") = 3.65 k

AREA of 1/2" BOLT = .196 in²

$$f_T = \frac{3.65^k}{.196 \text{ in}^2} = 18.6 \text{ ksi} < 19.1 \text{ ksi}$$ OK

GPD-2706 8/76 (ED-69)

CALCULATION SHEET

BECHTEL

CALC. NO. 8011 - 10.5 | 10.12 REV. NO. 0

ORIGINATOR J. E. Wilo DATE 7·24·81 CHECKED R. Mann DATE 24 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867 - 020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 33

DESIGN REPAIR

CONCLUSION - REPLACE EXISTING W6 x 25 ON EAST SIDE WITH W8 x 31. THROUGH BOLT 8" x 1/2" PLATE TO NEW W8 x 31 WITH 1/2" A-36 BOLTS. USE 16" C-C SPACING FOR THROUGH BOLTS WITH MAXIMUM SPACING of 18".

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. W. DATE 7.22.81 CHECKED R. Mun DATE 24 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

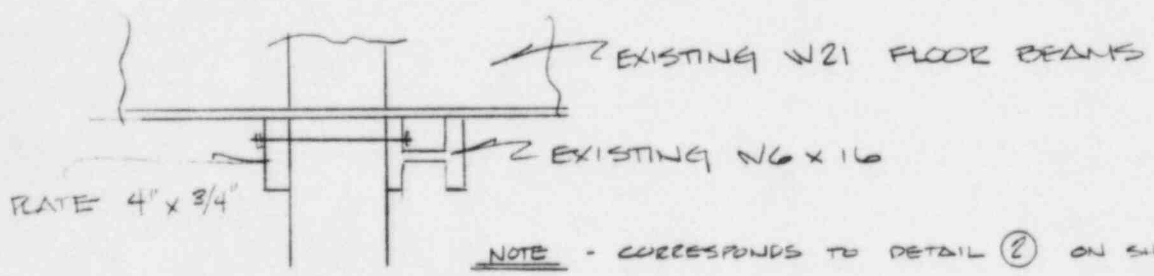
SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 34

DESIGN REPAIR

TOP BOUNDARY CONNECTIONS

PLATES WILL BE USED AS TOP BOUNDARY CONNECTIONS IN ORDER TO AVOID INTERFERENCES WITH VERTICAL CONDUITS

① THE FIRST DETAIL IS FOR ALL SECTIONS of WALL BETWEEN COLUMNS ⑱ & ⑯

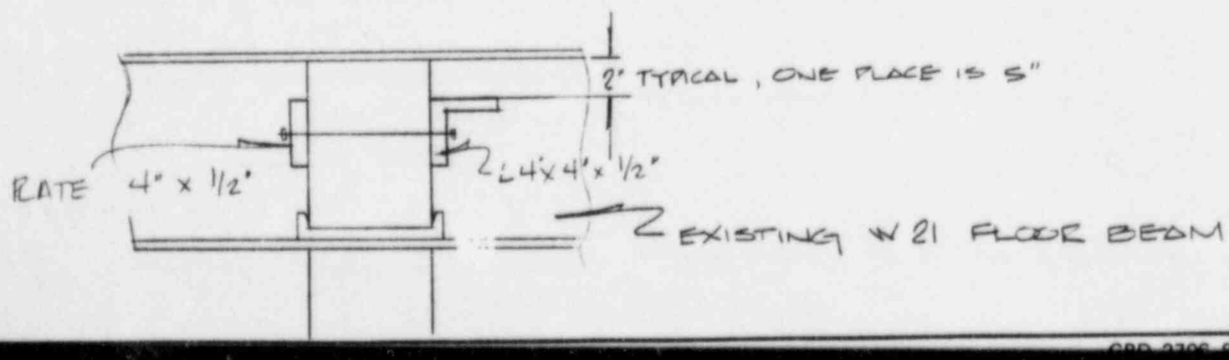


NOTE - CORRESPONDS TO DETAIL ② ON SHEET 30

LOOKING NORTH

② THE SECOND DETAIL IS FOR SECTIONS OF WALL WITH IMBEDDED 10 [15.3 CHANNELS

NOTE - CORRESPONDS TO DETAIL ② ON SHEET 30



BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Wah DATE 7.22.81 CHECKED R. Mani DATE 24JUL81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 35

DESIGN REPAIR

DESIGN PLATES (TOP BOUNDARY) SEE SECTION ① OR DETAIL ②

FROM 'STRUDL' RUNS T20883-1 (STRIP 4 ON PG. 28)

MAX. SHEAR AT TOP = 6.54 K

UNIT LOAD AT TOP = $\frac{6.54\ K}{45.75\ IN} = .143\ K/IN$

MAX. SPAN AT TOP = 6'-8" (80")

$M_{MAX} = \frac{\omega L^2}{8} = \frac{.143(80)^2}{8} = 114.36\ K \cdot IN$

$V_{MAX} = \frac{\omega L}{2} = \frac{.143(80)}{2} = 5.72\ K$

CHECK W6 X 16 (EXISTING) $S_x = 10.2\ IN^3$

$f_b = \frac{M}{S} = \frac{114.4}{10.2} = 11.2\ ksi < 21.6\ ksi$ OK

NOTE - SOMETHING IS NEEDED TO TRANSFER WESTERLY LOAD TO EXISTING W6 X 16

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR [signature] DATE 7-22-81 CHECKED R. Mau DATE 24 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 36

DESIGN REPAIR

try PLATE 4" x 3/4"

$$S_Y = \frac{bh^2}{6} = \frac{4(.75)^2}{6} = .375 \text{ in}^3$$

$f_{b\,all}$ = 27 ksi (MINOR AXIS BENDING)

$$M_{all} = f_b(S_Y) = 27(.375) = 10.13$$

MAX. SPACING of BOLTS

$$L = \left(\frac{M(8)}{w}\right)^{1/2} = \left(\frac{10.13(8)}{.143}\right)^{1/2} = 23.8"$$ USE 23" MAX SPACING

CHECK TENSILE STRESS IN BOLTS

MAX. LOAD ON BOLTS = .143 k/in (23") = 3.29 k

AREA of 1/2" BOLT = .196 in²

$$f_T = \frac{3.29^k}{.196 \text{ in}^2} = 16.8 \text{ ksi} < 19.1 \text{ ksi}$$ OK

CONCLUSION - FOR TOP BOUNDARY REPAIR BOLT PL 4" x 3/4" TO EXISTING W6x16 WITH 1/2" A-36 BOLTS SPACED 16" C-C WITH MAX. SPACING = 23"

CALCULATION SHEET

BECHTEL

ORIGINATOR J. L. W. DATE 7 22 81 CALC. NO. 8011-10.5/10.12 REV. NO. 0
CHECKED R. Mann DATE 24 JULY 81
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020
SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 37

DESIGN REPAIR

DESIGN TOP PLATE & ANGLE SEE SECTION ② & DETAIL ②

FROM HOLLOW PORTION CALCULATIONS (SHEETS 25 & 26)

$V_{MAX} = 1.37^k$

UNIT LOAD ON PLATES $= \frac{1.37^k}{45.75"} = .030\ k/in$

$M_{MAX} = \frac{.030(80)^2}{8} = 24.0\ k \cdot in$ $V_{MAX.} = \frac{.030(80)}{2} = 1.2^k$

try L 4" x 4" x 1/2" ANGLE $S_x = 1.97\ in^3$

$f_b = \frac{M}{S} = \frac{24.0}{1.97} = 12.2\ ksi < 21.6\ ksi$ OK

$f_v = \frac{V}{A} = \frac{1.2}{3.75} = 0.32\ ksi < 14.4\ ksi$ OK

A PLATE MUST BE USED ON WEST SIDE of WALL TO TRANSFER LOAD TO ANGLE

try PL 4" x 1/2"

$S_{YY} = \frac{bh^2}{6} = \frac{4(1/2)^2}{6} = .17\ in^3$

$f_{b\,all} = 27\ ksi$ (MINOR AXIS BENDING)

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J.E. Wuk DATE 7.22.81 CHECKED R. Mumi DATE 24JULY81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN REPAIR SHEET NO. 38

DESIGN REPAIR

DESIGN ANGLE & TOP PLATE

$M_{all} = f_b (S_y) = 27 (.17) = 4.6$ K·IN

MAX. SPACING of BOLTS

$L = \left(\frac{M(8)}{\omega}\right)^{1/2} = \left(\frac{4.6(8)}{.03}\right)^{1/2} = 35"$ USE MAX. SPACING 32"

CHECK TENSILE STRESS IN BOLTS

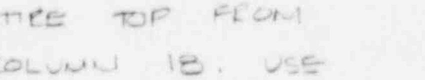
MAX LOAD ON BOLTS = $.030 (32") = .96^K$

AREA of 1/2" BOLT = .196

$f_T = \frac{.96^K}{.196 \text{ IN}^2} = 4.9$ KSI < 20 KSI   OK

CONCLUSION — FOR TOP CONNECTION, THROUGH BOLT A PL 4" x 1/2" TO AN L 4" x 4" x 1/2" ANGLE AT TOP PARTS of WALL WITH EXISTING 10 C 15.3 CHANNELS. USE 32" SPACING C-C FOR BOLTING PLATE TO ANGLE. ALSO, THROUGH BOLT 4" x 3/4" PLATES TO EXISTING TOP W6x16'S ALONG ENTIRE TOP FROM COLUMN 16 NORTH TO COLUMN 18. USE 16" SPACING FOR BOLTING PLATE TO EXISTING W6x16.

NOTE - STAGGER ALL BOLTS IE



CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Wut DATE 7-22-81 CHECKED R. Mam DATE 24 JULY 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT IEB011 BULLETIN - REPAIR SHEET NO. 39

DESIGN REPAIR

CHECKS

(1) CHECK EXISTING 10[15.3 — 'STRUDL' RUN T20574-1 STRIP # 3 PG 25

MAX. LOADS ON CHANNEL

FROM 'STRUDL' (SUPPORT 3) = 3.19^k

FROM HOLLOW BLOCK CALCS = 1.37^k (SHEET # 26)

TOTAL MAX LOAD = 3.19 + 1.37 = 4.56^k

MAX. UNIFORM LOAD = $\frac{4.56'}{45.75''}$ = .100 k/in

MAX SPAN = 80'

$M_{MAX} = \frac{wL^2}{8} = \frac{.100(80)^2}{8} = 79.74$ k·in $V_{MAX} = 3.99^k$

$S_{xx} = 13.5$ in³ (FOR CHANNEL 10C15.3)

$f_b = \frac{M}{S_x} = \frac{79.74}{13.5} = 5.91$ ksi < 21.6 ksi OK

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. C. W. DATE 7.24.81 CHECKED R. Mann DATE 24 JULY 81

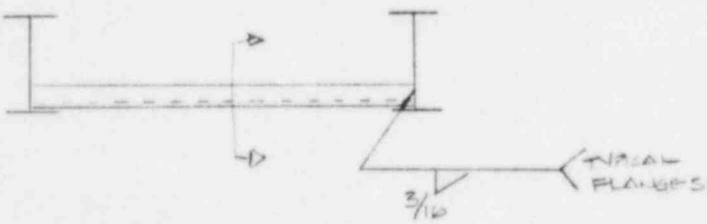
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

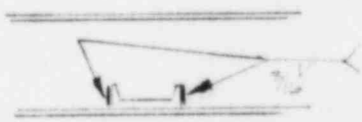
SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 40

DESIGN REPAIR

CHECKS ① CHECK EXISTING 10C15.3

END CONNECTIONS (REF. DWG 51006 - SECTION 5)





ASSUMED WELD

$V_{MAX} = \frac{.100(80)}{2} = 4.0^k$

ASSUME WELD IS ON ONE SIDE of EACH FLANGE

LENGTH of FLANGE = 2.600 (SAY 2½")

ALLOWABLE FORCE ON 3/16" WELD = 2.78 k/IN

ALLOWABLE FORCE ON WELD = 2.78k(2.5)(2) = 13.9k > 4.0k <u>OK</u>

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.6/1.2.12 REV. NO. 0

ORIGINATOR [signature] DATE 7 22 81 CHECKED [signature] DATE 7 AUG 81

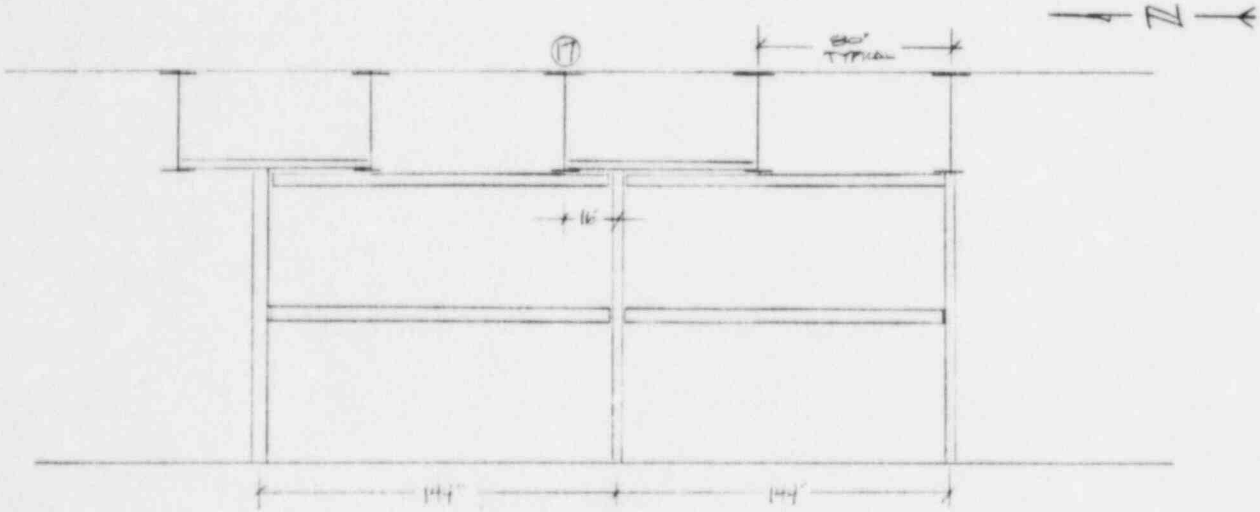
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 41

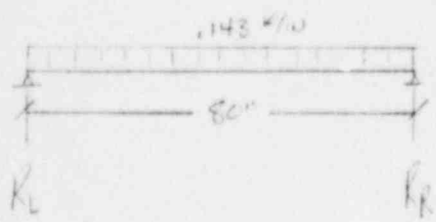
DESIGN REPAIR

CHECKS (3) EXISTING IMBEDDED W10 x 21

DETERMINE WORST LOADED CASE (W10x21 JUST SOUTH of COLUMN (17))



REACTION FROM TOP SUPPORT (DUE TO PLATES ATTACHED TO EXISTING W6x16)



$R_L = R_R = \frac{.143\,(80)}{2} = 5.72^K$

$R_{TOTAL} = 5.72^K(2) = 11.4^K$

BECHTEL

CALCULATION SHEET

ORIGINATOR J. E. Wule DATE 7-22-81 CALC. NO. 8011-10.5/10.12 REV. NO. 0
CHECKED R. Mm DATE 7 AUG 6-81
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020
SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 42

DESIGN REPAIR

CHECKS (3) EXISTING IMBEDDED W10 x 21

REACTION FROM MIDDLE SUPPORT

- DUE TO NEW PLATE BOLTED TO NEW W8 x 31

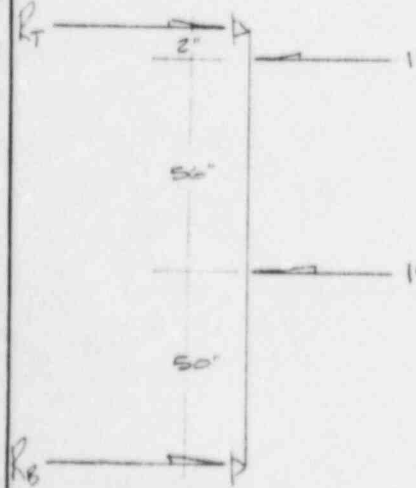
MAX. REACTION AT 50" SUPPORT (FROM 'STRUDL' RUN)

$R = 9.29^k$

$w = \frac{9.29^k}{45.7 \text{ in}} = .203 \text{ k/in}$

$V_{MAX} = \frac{wL}{2} = \frac{.203(144)}{2} = 14.6^k$ (FROM EACH W8 x 31)

DETERMINE MAX. MOMENT FOR W10 x 21



$R_T = \frac{11.4(106)}{108} + \frac{14.6(2)(50)}{108} = 24.7^k$

$R_B = 15.9^k$

$M_{MAX} = 15.9^k(50") = 796 \text{ k-in}$

S_x (W10x21) = 21.5 in³

$f_b = \frac{M}{S_x} = \frac{796}{21.5} = 37.0 \text{ ksi} > 22 \text{ ksi}$ NG

∴ EXISTING IMBEDDED W10x21 MUST BE STIFFENED

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR [signature] DATE 7.22.81 CHECKED [signature] DATE 7 Aug 81

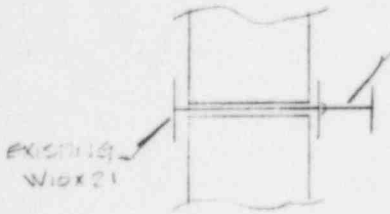
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 43

DESIGN REPAIR

CHECKS ⟨3⟩ EXISTING IMBEDDED W10 x 21

DESIGN WT WHERE NEEDED AS STIFFENER



try WT 5x15 $d = 5.235''$

$A = 4.42 \text{ in}^2$ $\bar{y} = 9.9 + 5.235 - 1.1$

$I_x = 9.28 \text{ in}^4$ $= 14.035''$

SECTION PROPERTIES

NOTE - STIFFNESS WILL BE ADDED TO WEST SIDE OF W10'S EXCEPT FOR EXISTING IMBEDDED W10x21 JUST SOUTH of COLUMN 17 (WHICH WILL BE STIFFENED WITH A WT10.5 x 22 ON EAST SIDE)

EXISTING W10x21

$A = 6.2 \text{ in}^2$

$I_x = 107$ $\bar{y} = 4.95$

$$\bar{y} = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{6.2(4.95) + 4.42(14.035)}{[6.2 + 4.42]} = 8.73'' = c$$

$$I_x = 107 + 6.2[8.73 - 4.95]^2 + 9.28 + 4.42[14.035 - 8.73]^2$$

$$I_x = 329.26 \text{ in}^4$$

$$f_b = \frac{Mc}{I} = \frac{796(8.73)}{329.26} = 21.1 \text{ ksi} < 21.6 \text{ ksi} \quad \underline{OK}$$

CONCLUSION - STIFFEN IMBEDDED W10x21 WITH A WT 5x15

NOTE - USING WT10.5 x 22 IN PLACE of WT5x15 WILL ONLY INCREASE SECTION PROPERTIES ∴ WT10.5 x 22 OK BY INSPECTION

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5 | 10.12 REV. NO. 0

ORIGINATOR [signature] DATE 7·22·81 CHECKED R. [signature] DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

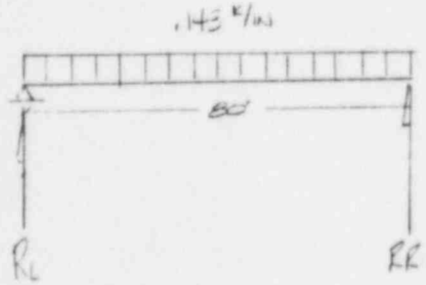
SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 44

DESIGN REPAIR

CHECKS ③ EXISTING IMBEDDED W10×21

CHECK W10×21 COLUMN JUST NORTH of COLUMN ⑯

REACTION FROM TOP SUPPORT



$R_L = \frac{.143(80)}{2} = 5.72^K$

$R_{TOP} = 5.72^K$

REACTION AT MIDDLE SUPPORT

REACTION AT MIDDLE SUPPORT COMES FROM 2 W6×25's of DIFFERENT SPANS (1 - 112" SPAN, 1 - 48" SPAN)

$R_{112} = \frac{wL}{2} = \frac{.203(112)}{2} = 11.37^K$

$R_{48} = \frac{wL}{2} = \frac{.203(48)}{2} = 4.87^K$

$R_{TOTAL} = 11.37 + 4.87 = 16.24^K$

CALCULATION SHEET

CALC. NO. 8011-10.5|10.12 REV. NO. 0

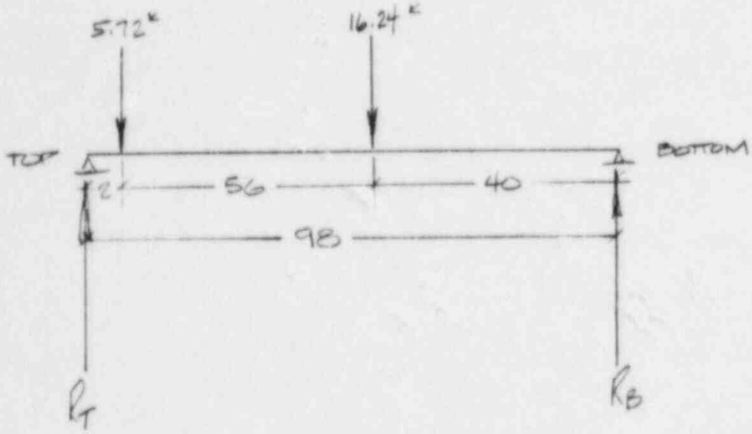
ORIGINATOR J. E. Wu DATE 7·22·81 CHECKED R. Mann DATE 7 AUG. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 45

DESIGN REPAIR

CHECKS ③ EXISTING IMBEDDED W10×21



$$R_B = \frac{5.72(2)}{98} + \frac{16.24(58)}{98} = 9.72 \text{ k}$$

$$M_{MAX} = 9.72(40) = 388.95 \text{ k·in}$$

$$f_b = \frac{M}{S_x} = \frac{389}{21.5} = 18.1 \text{ ksi} < 21.6 \text{ ksi} \quad \underline{\underline{OK}}$$

CONCLUSION – THIS IMBEDDED W10×21 DOES NOT NEED TO BE STIFFENED. HOWEVER, THE REST OF THE IMBEDDED W10×21'S FROM COLUMN 16 TO COLUMN 18 WILL HAVE TO BE STIFFENED (THERE WILL BE FOUR)

CALCULATION SHEET

CALC NO. [illegible] REV. NO. 0

ORIGINATOR [illegible] DATE 7-22-81 CHECKED R. [illegible] DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT [illegible] SHEET NO. 46

DESIGN BEAMS

CHECK (B) EXISTING IMBEDDED W10X21

END CONNECTIONS

REF. DWG 51006 SECTION S & T AND DETAIL 5

FROM SHEET 42

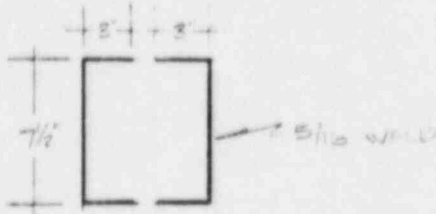
REACTION @ TOP = 24.7 K RBOTTOM = 15.9 K

REACTION 3" FROM TOP = 13.33 K MMAX = 796 K-IN

CONNECTIONS

TOP - ANGLES WELDED TO 10 [15.3

WELD PATTERN



$f_v = \frac{V}{L_w}$ WHERE L_w = LENGTH of WELD

$f_v = \frac{24.7}{2(7.5)+4.0} = 0.9148$ K/in < 4.64 K/in (ALLOWABLE for 5/16" WELD) OK

BOLTS (2-3/4" A-325 BOLTS DOUBLE SHEAR) BOLTS ARE LOCATED 3" FROM TOP

V_{BOLTS} = 13.33 K (SHEET 42)

FROM SEC 4-4 1970 AISC CODE

F_{ALL} = 2[13.25] = 26.5 K > 13.33 K OK

CALCULATION SHEET

CALC. NO. Bul-10-5 R2 REV. NO. 0

ORIGINATOR J. E. [illegible] DATE 7-22-81 CHECKED R. [illegible] DATE 7 AUG. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-MO

SUBJECT BUL BULLETIN - REPAIR SHEET NO. 47

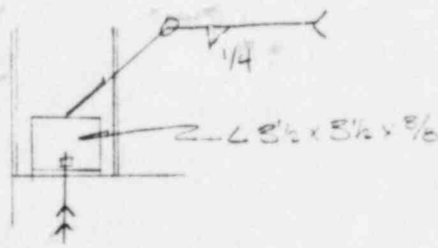
DESIGN REPAIR

CHECK (3) EXISTING IMBEDDED W10 x 21

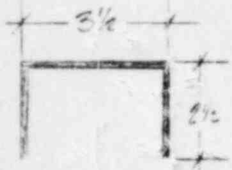
BOTTOM CONNECTIONS - REFERENCE DWG 51006 SECTIONS S & T

- IMBEDDED W10x21 IS WELDED TO CLIP ∠'S WHICH ARE BOLTED TO 3/4" ¢ WEJ - IT'S IMBEDDED 5"

CHECK WELDS



DWG 51006 SECTION T



$A_w = 2\left[3\tfrac{1}{2} + 2(2\tfrac{1}{2})\right] = 17$ IN

ASSUMED WELD PATTERN EACH SIDE of W10x21

$f_v = \frac{V}{A_w} = \frac{15.9^k}{17} = .94\ ^k/_{in} < 3.71\ ^k/_{in}$ OK

(ALLOWABLE FOR 1/4" FILLET WELD)

GPD-2706 8/76 (ED-69)

CALCULATION SHEET

BECHTEL

CALC. NO. E011-10.5/10.6 REV. NO. 0

ORIGINATOR J.E. Wech DATE 7.22.81 CHECKED R. Mann DATE 7 Aug. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT E011 BULLETIN - REPAIR SHEET NO. 48

DESIGN REPAIR

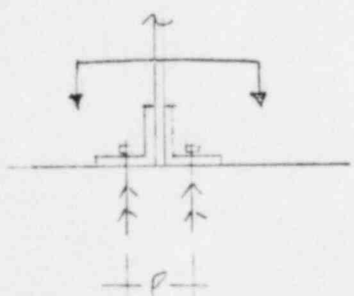
CHECKS (3) EXISTING IMBEDDED W10x21

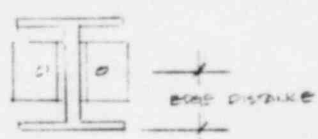
CHECK 3/4" ϕ WELD-ITS

DWG 51006 NOTE - DRAWING DOESN'T GIVE DIMENSION P

SECTION 5 CONSERVATIVELY ASSUME P =

ASSUMED EDGE DISTANCE = $\frac{9.9}{2}$ = 4.95





CONE OVERLAP

MIN $P = 1/4" + 2[3/4"] + 2[3/8"] = 2.5"$

$L_e = 5"$

$\therefore \frac{P}{L_e} = \frac{2.5}{5} = .5 \Rightarrow K = .5$

$R_{Fc} = K + \left(.3\frac{P}{L_e}\right) = .5 + (.3(.5)) = .65$ (DUE TO CONE OVERLAP)

EDGE DISTANCE

FROM PROCEDURE

$P = 2$(Actual Edge Distance) $= 2(4.95) = 9.9$

$L_e = 10$(Anchor ϕ) $= 10(.75) = 7.5$

$\therefore P/L_e = 9.9/7.5 = 1.32 \Rightarrow K = .49$

$R_F = .49 + .3(1.32) = .92$ (DUE TO EDGE DISTANCE)

GPD-2706 8/76 (ED-69)

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5|10.12 REV. NO. 0

ORIGINATOR J. E. Wal[illegible] DATE 7.22.81 CHECKED R. [illegible] DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 49

DESIGN REPAIR

CHECKS (3) EXISTING IMBEDDED W10 x 21

FROM MANUFACTURERS DATA [6.29.79] JOB NO. 11867-018
W/ MIN. EMBEDMENT = 3.25"

$V_{MAX} = 15.875$ K/BOLT F.S. = 4

$\text{MAX ALLOWABLE} = \frac{V_{MAX}}{F.S.} = \frac{15.875}{4} = 3.97$ K/BOLT

REDUCE MAXIMUM FOR CONE OVERLAP & EDGE DISTANCE

$\text{ALLOWABLE} = 3.97(R_{F1})(R_{F2}) = 3.97(.65)(.92) = 2.37$ K/BOLT

2.37 K/BOLT (2 BOLTS) = $4.75^K < 15.9^K$ NG

BOTTOM BOUNDARY CONNECTION of W10 x 21
MUST BE REPAIRED

CALCULATION SHEET

BECHTEL

CALC. NO. 801-10.5|10.12 REV. NO. 0

ORIGINATOR J.E. West DATE 8.7.81 CHECKED R. [illegible] DATE 7 AUG. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 50

REPAIR BOTTOM CONNECTION of EXISTING W10×21

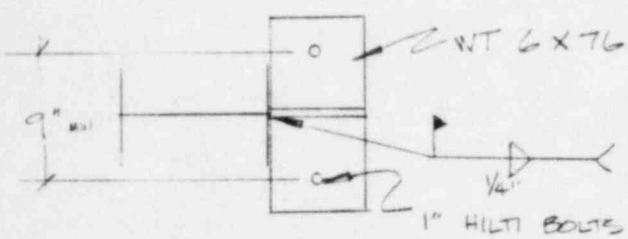
REACTION @ BOTTOM W/ RIGID SUPPORT ASSUMPTION = 15.9 K
THIS WILL BE LOWERED DUE TO SUPPORT
ACTING AS SPRING.

FACTOR REACTION

.113 = UNIFORM LOAD DUE TO SPRING SUPPORT
.203 = UNIFORM LOAD DUE TO RIGID SUPPORT

$$15.9^{K}\left(\frac{.113}{.203}\right) = 10.18^{K}$$

SAY 11^{K}



TYPICAL 3 COLUMNS

DETERMINE BOLT SIZE

try 2 - 1" HILTI ULT. SHEAR = 27.5 K FS = 4

$$V_{ALL} = \frac{27.5}{4} = 6.875\ ^{K}/_{BOLT}$$

$$2\ \text{BOLTS} = 2(6.875) = 13.75^{K} > 11^{K}$$ OK

USE 2 - 1" HILTI BOLTS

MINIMUM EMBEDMENT = 4 1/2"

GPD-2706 8/76 (ED-69)

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-105/10.2 REV. NO. 0

ORIGINATOR J. F. Wells DATE 8-7-81 CHECKED A. Mm DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 80-11 BULLETIN - REPAIR SHEET NO. 51

CHECK 1/4" WELD

WELD WEB of WT to EXISTING FLANGE of W10×21

LENGTH of WEB = 5½"

MINIMUM WELD LENGTH = 10"

$$\text{LOAD} = \frac{11^k}{10"} = 1.1^k/\text{IN} < 3.71^k/\text{IN} \qquad \underline{OK}$$

USE 1/4 WELD EACH SIDE

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. Weeks DATE 7-23-81 CHECKED R. Am DATE 7 AUG. 81

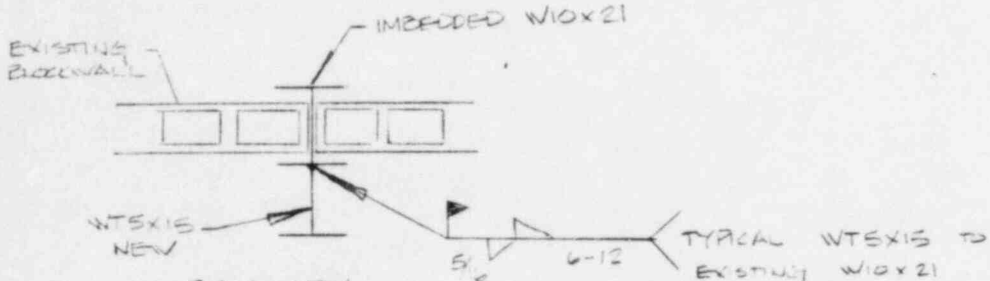
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO 52

DESIGN REPAIR

DESIGN WELDS

① WELD WT5x15 TO EXISTING W10x21 (IMBEDDED)



PLAN VIEW

CHECK SHEAR FLOW ACROSS WEB OF WT5x15

$f_v = \frac{VQ}{I}$ $V_{max.} = 24.7$ (REF SHEET 42)

$\bar{y} = 5.235 - 1.10 + 1.17 = 5.307''$ (SHEET 43)

$f_v = \frac{24.7(23.46)}{329.26}$ $A = 4.42\ in^2$

$Q = A\bar{y} = 4.42(5.307) = 23.46$

$f_v = 1.76\ ^K/_{in} < 4.64\ ^K/_{in}$ (ALLOWABLE FOR 5/16" WELD) OK

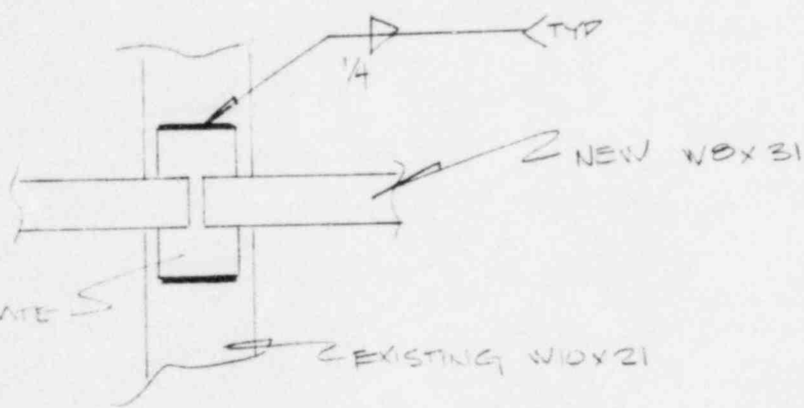
CALCULATION SHEET

CALC. NO.	[illegible]	REV. NO.	0
ORIGINATOR	[signature]	DATE	7.27.81
CHECKED	R. Man	DATE	7 AUG.81
PROJECT	MILLSTONE UNIT 2	JOB NO.	11867-020
SUBJECT	BOX BULLETIN - REPAIR	SHEET NO.	53

DESIGN REPAIR

DESIGN WELDS

② WELD 5½" x 1" x 1'-0 PLATE TO EXISTING W10 x 21



V = 29.2ᴷ (FROM BOTH W8x31's) [COMPRESSION, INTO PLANE of PAGE]

$f_v = \frac{V}{l_w} = \frac{29.2}{2(5\frac{1}{2})} = 2.65\ ^{k}/_{in} < 3.71\ ^{k}/_{in}$ (ALLOWABLE FOR 1/4" WELD)

OK

CALCULATION SHEET

BECHTEL

ORIGINATOR ______ DATE 7-27-81 CALC. NO. 8011-10.5/10.12 REV. NO. 0

CHECKED D. Man DATE 7 AUG. 81

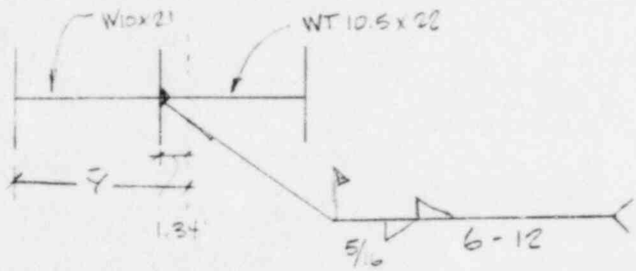
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

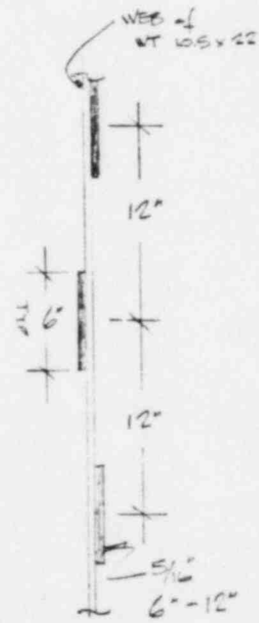
SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 54

DESIGN REPAIR

DESIGN WELDS

③ WELD WT 10.5 x 22 TO EXISTING W10 x 21





COMBINED SECTION PROPERTIES

$$\bar{y} = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{6.2\left(\frac{9.9}{2}\right) + 6.49\,(9.9 + 10.33 - 2.98)}{(6.2 + 6.49)} = 11.24''$$

CHECK SHEAR FLOW ACROSS WEB of WT 10.5 x 22

$f_v = \frac{VQ}{I}$ $V_{MAX} = 24.7^k$ (REF. SHEET 42)

$\bar{y} = 10.33 - 2.98 - 1.34 = 6.01''$

$f_v = \frac{24.7(39.0)}{657.82}$ $A = 6.49\ in^2$

$Q = A\bar{y} = 39.0$

$f_v = 1.46\ ^k/_{in}$ $I_x = 107 + 6.2\left[\frac{9.9}{2} + 1.34\right]^2 + 71.1 + 6.49\,[6.01]^2$

$I_x = 657.82$

$1.46\ ^k/_{in} < \frac{4.64\ ^k/_{in}}{2}$ (ALLOWABLE for 5/16" STAGGERED 6" @ 12" c-c WELD)

OK

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR [signature] DATE 1-27-81 CHECKED R. Mar DATE 7 AUG. 81

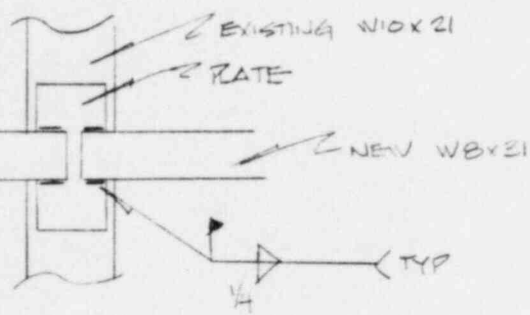
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 55

DESIGN REPAIR

DESIGN WELDS

(4) WELD NEW W8x31 TO NEW 5½" x 1" x 1'-0 LG PLATE



FROM SHEET 31, LOAD AT END of W8x31 = 14.62^K

WELD LENGTH = 2"

$f_v = \frac{V}{A_w} = \frac{14.62}{2(2")} = 3.66\ ^K/_{IN} < 3.71\ ^K/_{IN}$ (ALLOWABLE 1/4" WELD)

OK

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

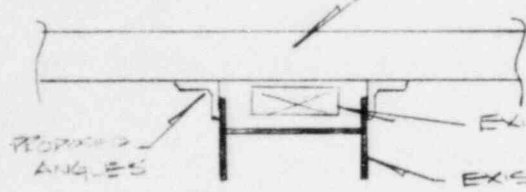
ORIGINATOR J.W.L DATE 7-23-81 CHECKED R. Man DATE JA 26 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 56

DESIGN REPAIR - TO PROTECT CABLE TRAY

SUPPORT WALL AROUND COLUMN (19) CABLE TRAY Z22DA10
& CONDUITS Z1T765
& Z1T790

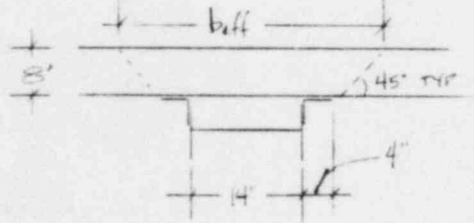


REFERENCE -
DWG 5127
SECTION E

LOAD FROM WALL WILL BE TRANSFERED TO W14 x 78 BY ADDITION of ANGLES

CALCULATE LOAD TRANSFERED TO BEAM

CRITICAL LOAD = D+L+Wt (3 psi)



ASSUME LEG of ANGLE = 4"

$b_{eff} = 14 + 2(4) + 2(8) = 38"$

LOAD ON BEAM = 38"(0.003 ksi) = 0.114 k/in

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-105 [illegible] REV. NO. 0

ORIGINATOR [signature] DATE 7-23-81 CHECKED R. Moran DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 57

<u>DESIGN REPAIR</u>

<u>CHECK MINOR AXIS BENDING IN W14×78</u>

$S_{YY} = 34.5 \text{ in}^3$

$$M_{MAX} = \frac{wL^2}{8} = \frac{0.114\,(128'')^2}{8} = 233 \text{ k·in}$$

$$V_{MAX} = \frac{wL}{2} = \frac{0.114\,(128)}{2} = 7.30^{k}$$

$$f_b = \frac{M}{S} = \frac{233}{34.5} = 6.75 \text{ ksi} < 21.6 \text{ ksi} \qquad \underline{OK}$$

EXISTING W14×78 IS ADEQUATE TO CARRY LOAD TRANSFERED FROM WALL.

CONNECTIONS ARE MADE WITH 2-3/4" ϕ A327 BOLTS WHICH ARE ADEQUATE TO CARRY WALL LOAD REFERENCE DWG. 5127 SECTION E

<u>CHECK CONNECTIONS</u>

2-3/4 ϕ A-325 BOLTS - SINGLE SHEAR

$V_{MAX} = 7.30^{k} < 6.63^{k}(2)$ <u>OK</u>

FROM MANUAL of STEEL CONSTRUCTION SECTION 4 4, 7th EDITION

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.2 REV. NO. 0

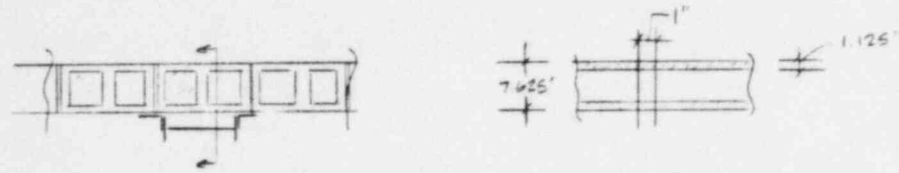
ORIGINATOR [signature] DATE 7.23.81 CHECKED [signature] DATE 7 AUG. 8[illegible]

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 58

DESIGN REPAIR

CHECK WALL BETWEEN ANGLES



L = 14" = DEPTH of W14 x 78

PER 1" STRIP

$$I = \frac{1"(7.625)^3}{12} - \frac{1"(5.125)^3}{12} = 25.7 \text{ in}^4$$

$$M = \frac{.003(14)^2}{8} = .0735 \text{ k-in} \qquad V = \frac{.003(14)}{2} = .021 \text{ k}$$

$$f_b = \frac{Mc}{I} = \frac{.0735\left(\frac{7.625}{2}\right)}{25.7} = .0109 \text{ ksi} < 0.020 \text{ ksi} \qquad \underline{OK}$$

$$f_v = \frac{V}{A} = \frac{.021}{2[1(1.125)]} = .0093 \text{ ksi} < 0.020 \text{ ksi} \qquad \underline{OK}$$

DESIGN ANGLES & WELDS

ASSUME EACH ∠ CARRIES 1/2 (bell) LOAD & MAX. VERTICAL SPAN of ANGLE = 50" (CONSERVATIVE)

w = .114/2 = 0.057 k/in

$$M_{MAX} = \frac{.057(50)^2}{8} = 17.8 \text{ k-in}$$

$$V_{MAX} = \frac{0.057(50)}{2} = 1.43 \text{ k}$$

CALCULATION SHEET

BECHTEL

ORIGINATOR [signature] DATE 7·23·81 CHECKED R. Mam DATE 7 Aug 81

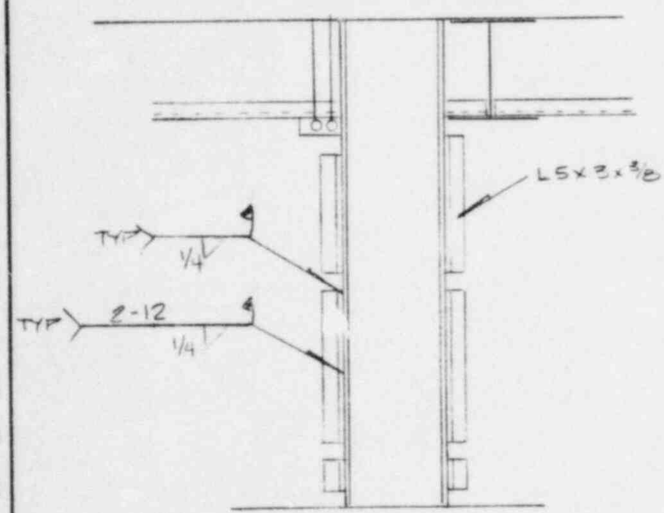
CALC. NO. 8011-10.5/10.12 REV. NO. 0

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-003

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 59

DESIGN REPAIR

DESIGN ANGLES & WELDS



SEE DWG SHEET 58

try L 5 × 3 × 3/8 $S_{xx} = 2.24\ in^3$

$f_b = \frac{M}{S} = \frac{17.8}{2.24} = 7.95$ ksi < 21.6 ksi OK

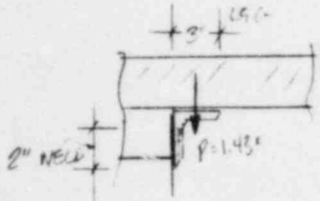
$\frac{b}{t} = \frac{3}{3/8} = 8 < 12.67 = \frac{76}{\sqrt{36}}$ OK

$L_u = 50" < \frac{76(5)}{\sqrt{36}} = 63"$ OK

SHEAR ON ANGLE OK BY INSPECTION

CHECK WELDS

$t_f = 11/16 \Rightarrow$ MIN WELD = 1/4"   TRY 2" @ EACH END



V = 1.43k

M = 1.5"(1.43k) = 2.15 k·in

L = 2"

$S_w = \frac{2(2)^2}{6} = 1.33\ in^2$

FORCE ON WELD

SHEAR $\frac{1.43}{2} = 0.72$ k/in

BENDING $\frac{2.15}{1.33} = 1.62$ k/in

$F_R = \sqrt{(0.72)^2 + 1.62^2} = 1.77$ k/in < 3.71 k/in

OK

NOTE: ADD STITCH WELD ON SIDE of ANGLE CONSERVATIVE

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-105/10.12 REV. NO. 0

ORIGINATOR J. E. Weeks DATE 7-23-81 CHECKED R. Mm DATE 7 Aug '81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 60

DESIGN REPAIR

ASSUME SECTION ABOVE EXISTING IMBEDDED 10C 15.3 CHANNEL IS HOLLOW & REPAIR WITH PL 4" x 1/2" THROUGH BOLTED TO AN L 4" x 4" x 1/2" ANGLE. USE THE 1/2" PLATE ON WEST SIDE TO AVOID INTERFERENCES.

REFERENCE: STRIP (5) SHEET 27

DRAWING — DETAIL (2) SHEET 30
CALCULATIONS — SHEETS 25 & 37

CALCULATION SHEET

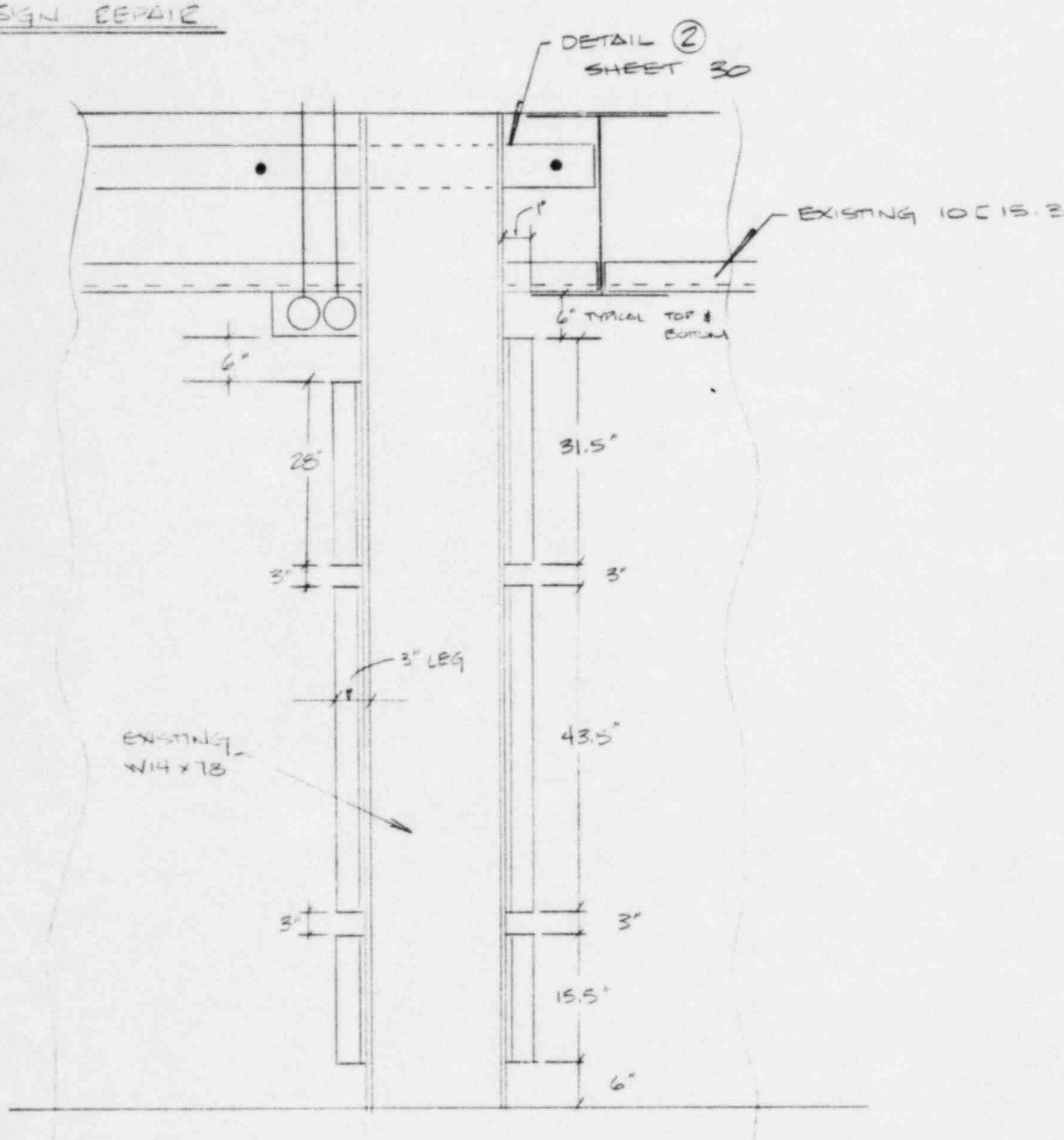
CALC. NO. 8011-10.5|10.2 REV. NO. 0

ORIGINATOR J E Wil DATE 7.23.81 CHECKED A. Mum DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 61

DESIGN REPAIR



ELEVATION VIEW LOOKING EAST

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR [signature] DATE 8-3-81 CHECKED [signature] DATE 7 AUG. 81

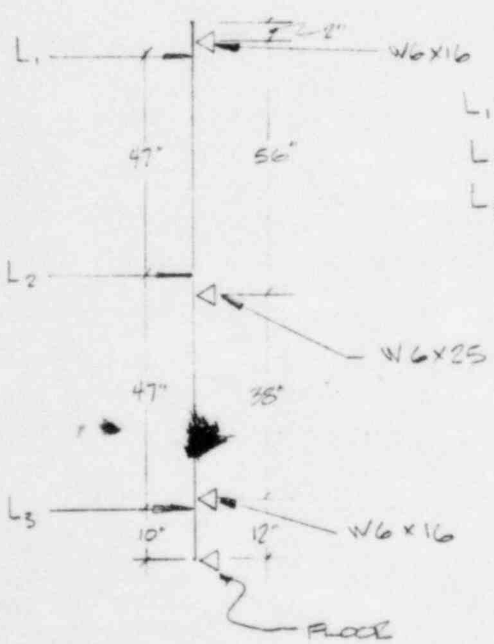
PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 62

<u>CONCLUSION</u> - THIS REPAIR HAS BEEN SHOWN TO PROTECT THE S/R ITEMS BOTH ATTACHED TO THE WALL & IN THE FALL PATH FOR D + L + W_T (3 PSI) LOAD CASE. IT NOW MUST BE SHOWN THAT THE WALL IS NOW SAFE FOR THE D+L+E & D+L+E' LOAD CASES.

<u>D+L+E'</u> - IT WAS PREVIOUSLY SHOWN THAT D+L+W_T WAS THE WORST CASE IN THE WESTERLY DIRECTION. THEREFORE SINCE WALL WAS REPAIRED FOR D+L+W_T IN WESTERLY DIRECTION IT WILL BE OK FOR D+L+E' & D+L+E IN WESTERLY DIRECTION. HOWEVER, IT MUST BE SHOWN WALL IS OK FOR D+L+E' IN EASTERLY DIRECTION

<u>MODEL of WALL</u> - WORST '6t' SECTION
LOADS FROM SHEET NOS 8 & 9



$L_1 = 1.53^K$

$L_2 = .93^K$

$L_3 = 1.53^K$

<u>NOTE</u> - THE W6's CAN BE ASSUMED TO ACT AS SUPPORTS & NOT ACT AS SPRINGS BECAUSE THEY ARE ATTACHED TO THE EXISTING W10x21 COLUMN IN THE '6L' SECTION ∴ DEFLECTIONS WILL BE MINIMAL

CALCULATION SHEET

BECHTEL

CALC. NO. 801-10.5/10.12 REV. NO. 0

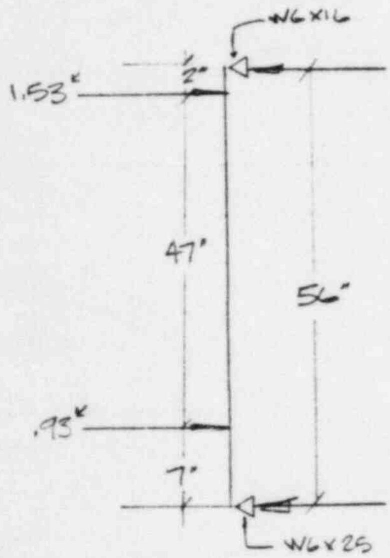
ORIGINATOR J. E. Wells DATE 8·3·81 CHECKED R. Mann DATE 7 AUG. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867 - 020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 63

CONCLUSION

MAXIMUM BENDING MOMENT WILL OCCUR BETWEEN TOP W6x16 & MIDDLE W6x25. CONSERVATIVELY ASSUME SIMPLE SUPPORTED BEAM



$$R_{16} = \frac{.93(7) + 1.53(54)}{56} = 1.59^k$$

$$R_{25} = \frac{1.53(2) + .93(49)}{56} = .87^k$$

$$M_{MAX} = .87^k(7") = \underline{6.09\ k \cdot in}$$

FROM SHEET No 13 -
CRACKED SEISMIC MOMENT = 102.2 K·IN

MAX. MOMENT on '6t' SECTION = 102.2 + 6.09 = 108.3 K·IN
BUT 128" LONG

CONVERT THIS TO PSI LOAD

$$M = \frac{wL^2}{8} \quad \therefore \quad w = \frac{M8}{L^2} = \frac{108.3\ (8)}{(108)^2} = .0743\ k/in$$

$$w = P(45.75) \quad \therefore \quad P = \frac{w}{45.75} = \frac{.0473}{45.75"} = .00162\ ksi = 1.62\ psi$$

$$P = 1.62\ psi < 2\ psi \qquad \underline{OK}$$

REFERENCE CALC #7604 - PC-M

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

ORIGINATOR J. E. W[illegible] DATE 8-3-81 CHECKED R. Mansi DATE 7 AUG. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 64

CONCLUSION

D+L+E' - IT WAS JUST SHOWN THAT THE MAX. PRESSURE LOAD IN THE EASTERLY DIRECTION DUE TO D+L+E' WILL BE 1.62 psi, THIS IS LESS THAN THE 2 psi* THE Q-FLOOR AND FRAMING WERE DESIGNED FOR. REFERENCE CALC 7604 PR-M.

D+L+E - FROM SHEET No's 15 & 19 THE EXTERNAL LOADS & CRACKED SEISMIC MOMENT WILL BE LESS FOR THE D+L+E CASE THAN THEY WERE FOR THE D+L+E' CASE ∴ THE D+L+E LOAD CASE WILL PASS ALSO

*NOTE THE ORIGINAL Q-DECK & FRAMING ON EAST SIDE WERE DESIGNED TO HANDLE A LOAD OF 2 psi IN THE EASTERLY DIRECTION. THIS FRAMEWORK WAS MODIFIED BY INCREASING SOME of THE EXISTING W6x25'S TO W8x31'S. BY INSPECTION, SINCE THE NEW W8x31's WILL HAVE LARGER SECTION PROPERTIES IT CAN BE SEEN THAT IT WILL ONLY IMPROVE RESISTANCE TO A 2 psi EASTERLY LOAD. THE SAME THING CAN ALSO BE SAID ABOUT THE WELDS, AS THEY WERE DESIGNED FOR A 3 psi LOAD CASE. HOWEVER, THE 5½" x 1" x 1'-0 PLATES WELDED TO EXISTING W10x21 COLUMNS MUST BE CHECKED FOR MINOR AXIS BENDING

BECHTEL

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

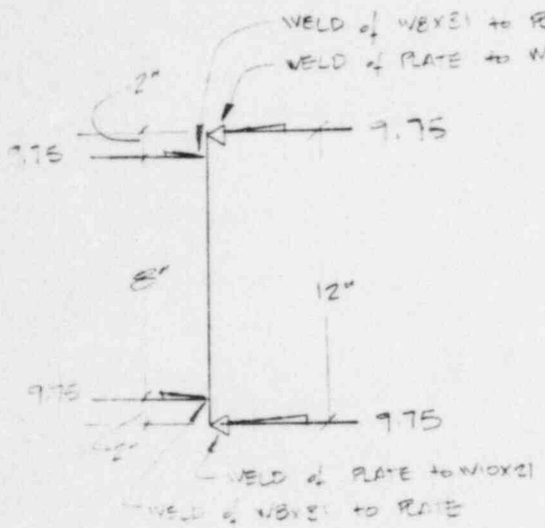
ORIGINATOR [illegible] DATE 8.4.81 CHECKED [illegible] DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 65

CONCLUSION

MODEL PLATE AS SIMPLE SUPPORTED BEAM WITH CONCENTRATED LOADS AT WELDS



FROM SHEET No 31, THE REACTION AT THE END of W8 x 31 = 14.62^k (FOR 3 psi LOAD)

PROPORTION 3 psi → 2 psi TO FIND LOAD AT END of W8x31

$$\frac{x}{2} = \frac{14.62^k}{3} \Rightarrow x = 9.75^k$$

FROM EACH W8x31

$$M_{PL} = 9.75^k (2\ in) = 19.5\ k\text{-}in$$

$$S_{PLATE} = \frac{b(h^2)}{6} = \frac{5\tfrac{1}{2}(1)^2}{6} = .917\ in^3$$

$$f_b = \frac{M}{S} = \frac{19.5}{.917} = 21.3 < 27\ ksi$$ OK

∴ PLATE IS OK IN EASTERLY AS WELL AS WESTERLY DIRECTIONS

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

TOR J. F. Web DATE 8-6-81 CHECKED R.I.Mm. DATE 7 AUG. 81

MILLSTONE UNIT 2 JOB NO. 11867-020

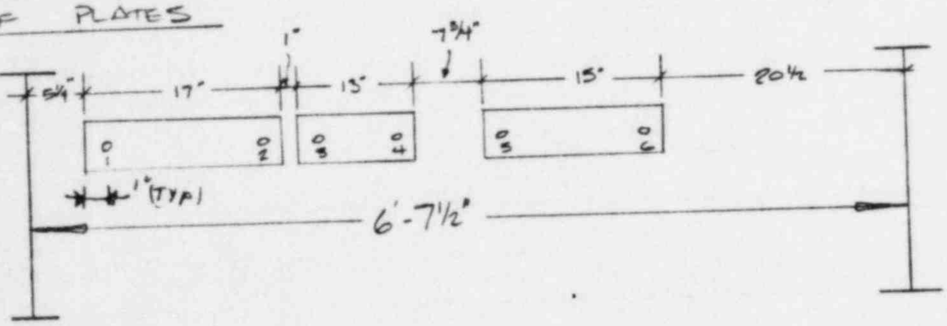
8011 BULLETIN - REPAIR SHEET NO. 66

REVISION A - DRAWING

[redacted]

ITEM ① - AN ANGLE AND PLATE MUST BE ADDED NORTH of COLUMN 19 TO PROTECT A Z CABLE TRAY ON WEST SIDE. SEVERAL PLATES MUST BE USED TO TRANSFER LOAD IN WESTERLY DIRECTION (3 psi) TO ANGLE, AS THERE ARE SEVERAL INTERFERENCES.

LAYOUT OF PLATES



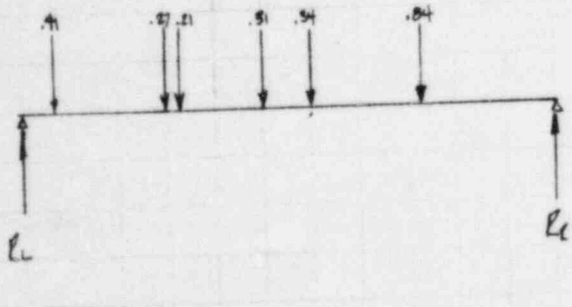
LOADS ON BOLTS FROM SHEET № 37 w = 0.030 k/in

1 - .03 (6¼ + 15/2) = .41 k
2 - .03 (15/2 + 3/2) = .27 k
3 - .03 (3/2 + 11/2) = .21 k
4 - .03 (11/2 + 9¾/2) = .31 k
5 - .03 (9¾/2 + 13/2) = .34
6 - .03 (13/2 + 21½/2) = .84 k

Total = 2.38

∴ BOLTS ½"Ø THRU BOLTS OK

LOADS ON ANGLE



$$R_L = \frac{(.41(73.25) + .27(58.25) + .21(55.25) + .31(44.25) + .34(34.5) + .84(21.5))}{79.5}$$

[redacted]

R_L = 1.27

R_R = 1.11

CALCULATION SHEET

BECHTEL

CALC. NO. 8011-10.5/10.12 REV. NO. -

ORIGINATOR J. F. Wa[illegible] DATE 8.6.81 CHECKED [illegible] DATE 7 AUG 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 67

REVISION A - DRAWING

$M_{MAX} = 1.11(34.5) - .84(13) = 27.4$ K·IN

CHECK ANGLE (BENDING) $S_x = 1.97$ IN^3 ($\angle 4 \times 4 \times \frac{1}{2}$)

$f_b = \frac{27.4}{1.97} = 13.9$ KSI < 21.6 KSI OK

ITEM (2) - AN ANGLE MUST BE ADDED TO WALL ON EAST SIDE ABOVE EXISTING IMBEDDED C-CHANNELS NORTH of COLUMN 19 TO PROTECT A S/R DUCT IN FALL PATH.

WORST LOAD CASE IN THIS DIRECTION WILL BE 2 psi - CONSERVATIVELY USE REACTION FROM 3 psi FROM SHEET No 37 $w = .030$ K/IN

CHECK $\angle 4 \times 4 \times 1/2$ MAX SPAN = 8'-6" → 102" $S_x = 1.97$

$M_{MAX} = \frac{.03(102)^2}{8} = 39.02$ K·IN $f_b = \frac{39.02}{1.97} = 19.8 < 21.6$

OK

$V_{MAX} = \frac{.03(102)}{2} = 1.53^k$ f_v OK BY INSPECTION

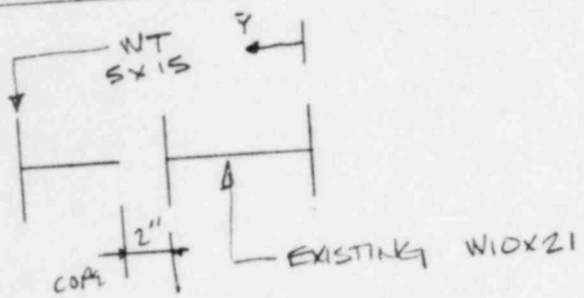
COPED SECTION ON DRAWING ALSO OK BY INSPECTION AS ARE 1/4 WELDS DESIGNATED ON DRAWING

CALCULATION SHEET

REVISION A - DRAWING

ITEM ③ - COPE WEB of WT STIFFNER

DETERMINE COPED SECTION PROPERTIES



FROM SHEET No 42

M_{MAX} = 796 K·IN

↑ DUE TO RIGID SUPPORT ASSUMPTION WOULD BE LESS IF SUPPORT ALLOWED TO DEFLECT

W10x21

$A = 6.2 \text{ in}^2$

$\bar{Y} = 9.9/2 = 4.95''$

$I_x = 107 \text{ in}^4$

COPED WT 5x15

$A_f = 5.81(.51) = 2.96 \text{ in}^2$ (FLANGE)

$\bar{Y} = 9.9 + 5.235 - .255 = 14.88 \text{ in}$

$A_w = 2.725(.3) = .8175 \text{ in}^2$

$\bar{Y} = 13.26 \text{ in}$ (WEB)

$$\bar{Y} = \frac{\Sigma A\bar{Y}}{\Sigma A} = \frac{6.2(4.95) + 2.96(14.88) + .8175(13.26)}{6.2 + 2.96 + .8175} = 8.58 \text{ in}$$

13.26 − 8.58 = 4.68

$$I = 107 + 6.2\left[8.58 - 4.95\right]^2 + \frac{.3(2.725)^3}{12} + .8175\left[4.68\right]^2 + \frac{5.81(.51)^3}{12} + 2.96\left[6.3\right]^2$$

I = 324.5

c = 8.58

$$f_b = \frac{Mc}{I} = \frac{796(8.58)}{324.5} = 21.0 \text{ Ksi} < 21.6$$

OK

CALCULATION SHEET

CALC. NO. 8011-10.5/10.12 REV. NO. 0

CHECKED R. Mmi DATE 7 AUG 81

ORIGINATOR J.E. Web DATE 8.7.81 JOB NO. 11867-020

PROJECT MILLSTONE UNIT 2 SHEET NO. 69

SUBJECT 8011 BULLETIN - REPAIR

CONCLUSION

THE REPAIR WAS DESIGNED WITH THE W8x31 BOLTED TO THE PLATE MODELED AS A RIGID SUPPORT. IN ACTUALITY THE STEEL SYSTEM of W8x31's & W10x21's WILL DEFLECT SOMEWHAT & MUST BE MODELED AS A SPRING SUPPORT.

DETERMINE MINIMUM STIFFNESS of SYSTEM

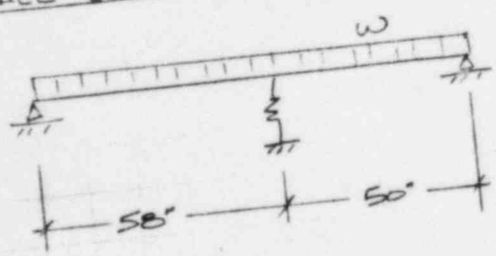
ASSUME - HORIZONTAL W8x31's UNIFORMLY LOADED
VERTICAL W10x21's CONCENTRATED LOAD @ CONNECTION

$$K = \left[\left[\frac{384\,(29{,}600)(110)}{5\,(144)^3}\right]^{-1} + \left[\frac{48\,(29{,}600)(107)}{(108)^3}\right]^{-1}\right]^{-1} = 49\ ^{k}/_{in}$$

$$K_{eff} = \frac{45.75''}{144''}\left[49\ ^{k}/_{in}\right] = \underline{15.6\ ^{k}/_{in}}$$

WALL BEAM MODEL

$w = .003\,(45.75) = .137\ ^{k}/_{in}$



GPD-2706 8/76 (ED-69)

CALCULATION SHEET

BECHTEL

CALC. NO. BOM-10.5/10.12 REV. NO. 0

ORIGINATOR J.E. Weck DATE 8.7.81 CHECKED R. Man DATE 7 AUG. 81

PROJECT MILLSTONE UNIT 1 JOB NO. 11867-020

SUBJECT BOM BULLETIN - REPAIR SHEET NO. 70

CONCLUSION

FIND EFFECTIVE I FOR CRACKED SECTION

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_t + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

SHT. 20

$M_{cr} = 32.9$ k-in

$I_{cr} = 118.1$ in⁴

$I_t = 770.4$ in⁴

try

$I_e = 168$

$M_a = 78.6$ k-in (REF 'STRUDL' RUN)

$$I_e = \left(\frac{32.9}{78.6}\right)^3 770.4 + \left[1 - \left(\frac{32.9}{78.6}\right)^3\right] 118.1$$

$I_e = 166 \cong 168$ OK

CHECK STRESSES IN THE WALL

MASONRY COMPRESSIVE BENDING

$$f_{cbm} = \frac{78.6(1.25)}{118.1} = 0.832 \text{ ksi} < 1.114 \text{ ksi}$$ OK

TENSILE STEEL

$$f_t = \frac{78.6(5.13-2.56)21.9}{118.1} = 37.5 \text{ ksi} < 54 \text{ ksi}$$ OK

MASONRY SHEAR

$$f_v = \frac{V}{A} = \frac{4.64}{156.1} = 0.0297 \text{ ksi} < 0.084 \text{ ksi}$$ OK

CALCULATION SHEET

BECHTEL

ORIGINATOR J. F. Weik DATE 8.7.81 CALC. NO. 8011-10.5/10.12 REV. NO. 0

CHECKED R. Mann DATE 7 AUG. 81

PROJECT MILLSTONE UNIT 2 JOB NO. 11867-020

SUBJECT 8011 BULLETIN - REPAIR SHEET NO. 71

CONCLUSION

REACTION @ MIDDLE SUPPORT = $2.65 + 3.30 = 5.95^k$

EQUIVALENT UNIFORM LOAD = $\frac{5.95^k}{45.75''} = .130\ ^k/_{in} < .203\ ^k/_{in}$

$.203\ ^k/_{in}$ WAS USED AS LOADING TO DETERMINE LOADS ON NEW MEMBERS

∴ LOADS USED IN CALCULATIONS WERE CONSERVATIVE <u>OK</u>

```
*******************************************
*                                         *
*            ICES STRUDL-II               *
*     THE STRUCTURAL DESIGN LANGUAGE      *
*                                         *
*   CIVIL ENGINEERING SYSTEMS LABORATORY  *
*  MASSACHUSETTS INSTITUTE OF TECHNOLOGY  *
*       CAMBRIDGE, MASSACHUSETTS          *
*                                         *
*      07.25.07         07 AUG 81         *
*                                         *
*      UNIVAC 1100 SERIES  EXEC 8         *
*             VERSION 2.8                 *
*                                         *
*******************************************
 2. TYPE PLANE FRAME
 3. UNITS KIPS INCHES
 4. CONSTANTS E 1350 ALL
 5. JOINT COORD
 6. 1 0.0 S
 7. 2 50.0 S
 8. 3 108.0 S
 9. MEMBER INCIDENCES
10. 1 1 2
11. 2 2 3
12. MEMBER 1 TO 2 PROP PRIS AX 156.1 IZ 168.
13. LOADING 1
14. MEMBER 1 TO 2 LOAD FORCE Y UNI .137
15. JOINT RELEASES
16. 1 TO 3 MOM Z
17. 2 KFY 15.6
18. STIFFNESS ANALYSIS
19. LOADING LIST 1
20. LIST DISP JOINT 2


 ****************************
 *RESULTS OF LATEST ANALYSIS*
 ****************************


  PROBLEM - REPAIR        TITLE - WALL 10-5


  ACTIVE UNITS  INCH   KIPS   RADIAN DEGF   SECOND

  ACTIVE STRUCTURE TYPE  PLANE FRAME
```

Orig. J. F. Date 8-7-81
Chk. R. Date 7 AUG 81

Millstone Unit 2 Job No. 1130 Rev. No. 0
Reevaluation of Concrete Masonry Walls
In Response to NRC I&E Bulletin 80-11
Vol. No. ______ Sht. No. 72

LOADING - 1

RESULTANT JOINT
SUPPORTS
DISPLACEMENTS -

JOINT		X DISPL	Y DISPL	Z DISPL	X ROT	Y ROT	Z ROT
2	GLO	.0000000	.3816448				.0016367

RESULTANT JOINT
FREE JOINTS
DISPLACEMENTS -

JOINT		X DISPL	Y DISPL	Z DISPL	X ROT	Y ROT	Z ROT

21. LIST SEC FOR ALL SECTION FR DS 0.0 0.1

Calc. No. [illegible] Rev. No. 0
Millstone Unit 2 Job No. 11867-020
Reevaluation of Concrete Masonry Walls
In Response to NRC I&E Bulletin 80-11
Vol. No. ______ Sht. No. 73

Orig. J.F. Wal Date 8-7-81
Chk. [illegible] Date 7 AUG-81

****
RESULTS OF LATEST ANALYSIS
****

PROBLEM - REPAIR TITLE - WALL 10-5

ACTIVE UNITS INCH KIPS RADIAN DEGF SECOND

ACTIVE STRUCTURE TYPE PLANE FRAME

ACTIVE COORDINATE AXES X Y

INTERNAL MEMBER RESULTS

MEMBER 1

LOADING 1

DISTANCE FROM START	FORCE AXIAL	FORCE Y SHEAR	FORCE Z SHEAR	MOMENT TORSION	MOMENT Y BENDING	MOMENT Z BENDING
.000 FR	.0000000	4.2006650	.0000000	.0000000	.0000000	.0000020
.100	.0000000	3.5156651	.0000000	.0000000	.0000000	-19.2908258
.200	.0000000	2.8306651	.0000000	.0000000	.0000000	-35.1566496
.300	.0000000	2.1456651	.0000000	.0000000	.0000000	-47.5974760
.400	.0000000	1.4606651	.0000000	.0000000	.0000000	-56.6132998
.500	.0000000	.7756652	.0000000	.0000000	.0000000	-62.2041259
.600	.0000000	.0906652	.0000000	.0000000	.0000000	-64.3699512
.700	.0000000	-.5943347	.0000000	.0000000	.0000000	-63.1107774
.800	.0000000	-1.2793347	.0000000	.0000000	.0000000	-58.4266043
.900	.0000000	-1.9643346	.0000000	.0000000	.0000000	-50.3174319
1.000	.0000000	-2.6493345	.0000000	.0000000	.0000000	-38.7832599

MEMBER 2

Orig. J.F. Wad Date 8.7.81
Chk. [illegible] Date 2 AUG. 81

LOADING 1

DISTANCE FROM START	FORCE AXIAL	FORCE Y SHEAR	FORCE Z SHEAR	MOMENT TORSION	MOMENT Y BENDING	MOMENT Z BENDING
.000 FR	.0000000	3.3043232	.0000000	.0000000	.0000000	-38.7832494
.100	.0000000	2.5097232	.0000000	.0000000	.0000000	-55.6439877
.200	.0000000	1.7151232	.0000000	.0000000	.0000000	-67.8960438
.300	.0000000	.9205232	.0000000	.0000000	.0000000	-75.5394182
.400	.0000000	.1259232	.0000000	.0000000	.0000000	-78.5741119
.500	.0000000	-.6686767	.0000000	.0000000	.0000000	-77.0001259
.600	.0000000	-1.4632767	.0000000	.0000000	.0000000	-70.8174610
.700	.0000000	-2.2578767	.0000000	.0000000	.0000000	-60.0261164
.800	.0000000	-3.0524766	.0000000	.0000000	.0000000	-44.6260924
.900	.0000000	-3.8470766	.0000000	.0000000	.0000000	-24.6173892
1.000	.0000000	-4.6416765	.0000000	.0000000	.0000000	-.0000066

22. FINISH

Docket No. 50-336

Millstone Unit No. 2

Final Safety Analysis Report

Appendix 5.D

Expanded Spectrum of Tornado Missiles

May, 1984

APPENDIX 5.D

EXPANDED SPECTRUM OF TORNADO MISSILES

The spectrum of tornado missiles is expanded to include the following:

1. Utility pole 13.5" dia. x 35 ft. long with density of 43 lbs/ft^3
2. 1" solid steel rod 3 ft. long with a density of 490 lbs/ft^3
3. 6", schedule 40 pipe, 15 ft. long with a density of 490 lbs/ft^3
4. 12", schedule 40 pipe, 15 ft. long with a density of 490 lbs/ft^3

For each of the above tornado-borne missiles, the following information is provided:

1. The maximum velocity and height attained. Assuming in the analyses that each of the missiles originate at ground level and at the highest structural elevation on the site capable of producing each missile.
2. The required thickness of a reinforced concrete missile barrier to stop the missiles without their penetrating the missile barrier. Discussing the adequacy of all tornado missile barriers protecting systems and components necessary for safe shutdown.
3. The required thickness of a reinforced concrete missile barrier to preclude the generation of secondary missiles within the structure.
4. The effects that secondary missiles could have on safety related equipment and systems in the event that they occur.

In developing the above information, the analytical approach presented in BC-TOP-9, Design of Structure for Missile Damage with the following exceptions is used, assuming the missiles do not tumble and are at all times oriented such as to have the maximum value of $\frac{CdA}{W}$ while in flight.

<u>THE TORNADO MODEL</u>: The tornado model will be patterned after the Dallas tornado of April 2, 1957, as studied by Hoecker (Ref. 1). The model is basically that given in WCAP-7897 (Ref. 2) but with a more rigorous extrapolation to the parameters desired for a design tornado, then given by Bates and Swanson (Ref. 3).

Hoecker summarized his findings by the use of a "pressure-time profile" for an average translational velocity of 27 mph and as a function of percentage of total pressure drop.

In attachment A, it is shown that when this time-pressure profile is used to solve the cyclostropic wind equation, the tangential wind velocities correspond with the experimental ones when a total pressure drop of 60 mb or 0.882 psi, and a translational velocity of 27 mph is substituted into the equation.

When a total pressure drop of 3 psi, and a 60-mph translational velocity (88 fps) is substituted into the same equation, a 304-mph maximum tangential velocity at a 300-foot radius is obtained. This corresponds closely to the assumptions which have been made in the past when describing the design tornado.

The two exponential equations used by Hoecker to determine the time-pressure profile cross each other at a radius of 1,240 feet instead of the 300 feet at which they cross when a translational velocity is 27 mph. Therefore, it is only necessary to use one equation since the starting tangential velocity corresponding to this distance is 66 mph, which is less than the minimum 75 mph considered by Bates and Swanson.

By incorporating these two assumptions, namely, that the vertical component is equal to one third of the tangential, and the radial component is a function of radial distances between minimum and maximum tangential components being considered, a complete windfield was defined by using the following equations:

$$V_t = \frac{249 \sqrt{\text{Exp}\ (-48.3\ V_1^3/R\ )\ V_1^3 D}}{R^3} \qquad (1)$$

$$V_r = - \frac{(1240-R)\ R}{(1240-300)} \qquad (2)$$

$$V_v = 1/3\ V_t \qquad (3)$$

Where:

V_t = tangential velocity (fps)

V_r = radial velocity (fps)

V_v = vertical velocity (fps)

V_1 = translational velocity (fps)

D = total pressure drop (psf)

R = radius (ft)

Equation (1) has been left in a general form for use in future models to predict different total pressure drops or translational velocities. However, at this time D is taken as 432 psf and V_1 = 88 fps.

The relative conservatism with reference to the actual Dallas tornado is shown in Figure 5.D-1.

Constant velocities from the ground to a height of 500 feet is a degree of conservatism which is justified by the expanded view of these velocities. This information has been published by Hoecker and is reproduced in Figure 5.D-2.

TYPES OF MISSILES: Previous studies had considered a car as a missile for low elevations and a wooden plank for high elevations. Later a small cross-section missile in the form of a pipe was added. Five missiles are now being considered.

All the missiles are intended as prototypes of the many missiles generated by tornados. Considering the present state of the art, a detailed physical description of a missile is of little value when designing the missile-proof target. Empirical formulations have to be used in areas where impactive energy and the impactive area are the points to be considered.

The more logical approach is to assume a generalized range of missiles with the required drag factors impacting at given elevations with the highest possible velocity. The impactive kinetic energy per square foot of impact area for each elevation would then be computed.

If a table is made with C_dA/W factors from 0.10 to 0.015, which is the smallest measurement for an airborne missile, it will be found that there is a drag factor that will give the highest velocity at each elevation. This is shown in Table 5.D-1.

It is interesting to show the small range and the gap left for the maximum drag factor proposed.

Wooden plank	0.06	Steel rod	0.031
Utility pole	0.026	6 inch pipe	0.029
		12 inch pipe	0.021

Impactive energy per unit area measured in lb/ft as shown on Table II is readily found as follows:

$$K = \frac{W\,V^2}{2g\,A} = \frac{V^2}{2g\,F}$$

Where:

K = impactive factor (lb/ft)

V = velocity at impact (fps)

A = area of impact (ft^2)

$F = C_dA/W$ for $C_d = 1$

As expected, this impactive factor is much higher at lower elevations: it varies from 49,640 lb/ft for a 10-foot elevation at 22,070 lb/ft for a 110-foot elevation.

Penetrations can be computed by the required empirical formulation which is workable in terms of these impactive factors.

METHODS OF INJECTION AND PROPULSION: Bates and Swanson propose three methods of injection:

a. Explosive injection
b. Aerodynamic injection
c. Ramp injection

These are intended to limit the height at which a given object may be injected into a tornado. So many considerations and assumptions have to be made that they become of no practical value when it is to be assumed that the object will reach the highest point of a structure even if the missile has to be held at a convenient elevation for injection to occur.

If an explosive injection occurs some distance away from a structure, it is concluded that the object could clear the structure, if such an injection could occur. Aerodynamic injection will require aerodynamic objects or else the injection is overestimated. Likewise, a ramp injection will depend on the given ramp, a factor that is hard to generalize.

All three methods of injection required many assumptions which make it difficult for generalization. A fourth method which would be called the "Uplift Injection" offers the advantages of simplicity and applicability.

In the uplift injection it is assumed that the wind finds its way beneath a surface and the object will become airborne at the time when the vertical component of the wind produces an upward forced equal to the weight of the object. While on the ground the object is assumed to be free to move on the horizontal plane in a frictionless manner as the tangential and radial components of the wind act on it.

When a missile flight is to be ascertained by applying the three components of the wind (tangential, radial and vertical) simultaneously, a random surface is assumed to be facing all three components. This random surface will produce what is called in WCAP-7897 an "effective drag factor" to be applied in all directions and which is computed as follows:

Cylinder:

$$C_e = \frac{0.389\ (h+0.66D)}{P_{obj}\ h\ D}$$

Parallelpiped:

$$C_e = \frac{0.483\ w(h+d)}{P_{obj}\ w\ h\ d}$$

Where:

C_e = effective drag factor

h = length, ft

D = diameter, ft

P_{obj} = density, lb/ft^3

w = width, ft

d = depth, ft

To date, this is the best method of computing an effective drag area for an object thrown into a tornado.

Using these effective drag areas in the computer program, DALLAS MISS GEN, the following results were obtained:

	Wooden Plank	Utility Pole	Steel Rod	6 in. Pipe	12 in. Pipe
	C_e = 0.03	C_e = 0.0082	C_e = 0.0097	C_e = 0.0015	C_e = 0.00078
Elevation					
10	223	182	192	98	---
20	264	---	---	--	---
30	279	---	---	--	---
40	270	---	---	--	---
50	261	---	---	--	---
60	258	---	---	--	---

These results show that only the wooden plank tank type missile could be sustained in the air. It supports WCAP-7897, Chapter 5: "Investigation of Some Specific Missiles" which clearly states: "The results of Figure 3 indicate that objects with a $C_d A/W$ less than 0.012 ft^2/lb will not be sustained by the vertical wind even if injected above immediate obstructions." (Figure 3 is contained in WCAP-7897.)

It appears then that instead of assuming impossible $C_d A/W$ factors for a given set of missiles, it is best to assume an infinite number of missiles, all with possible effective drag factors.

The required thickness of a concrete element that will just be perforated by a missile is given by:

$$T - \frac{427}{\sqrt{f'_c}} \frac{W}{D1.8} \left(\frac{V_s}{1000}\right)^{1.33}$$

where:

T = Thickness of concrete element to be just perforated (inches)

W = Weight of missiles (lb)

D = Diameter of missiles (inches)

Note: For irregularly shaped missiles, an equivalent diameter is used. The equivalent diameter is taken as the diameter of a circle with an area equal to the circumscribed contact, or projected frontal area, of the non-cylindrical missile.

V_s = Striking velocity of missile (ft/sec)

f'_c = Compressive strength of concrete (psi)

This formula is known as the Ballistic Research Laboratory, BRL, formula.

The thickness, t_p, of a concrete element required to prevent perforation must be greater than T. It is recommended to increase T by 25 percent, but not more than 10 inches, to obtain the t_p, required to prevent perforation.

$$t_p = 1.25T \leq T + 10 \text{ (in inches)}$$

The results obtained by using the above formula are presented in Table 5.D-5. The concrete barriers furnished to protect systems and components necessary for safe shutdown exceed the required thickness to prevent perforation by the missiles.

Spalling of concrete from the side opposite the impact surface of the element is considered as a secondary missile. For an estimate of the thickness that will just start spalling, it is recommended that the following equation be used:

$$T_s = 2T$$

where:

T_s = Concrete element thickness that will just start spalling (inches)

T = Concrete thickness to be just perforated (inches).

The thickness, t_s, of a concrete element required to prevent spalling must be greater than T_s. It is recommended to increase T_s by 25 percent, but not more than 10 inches, to prevent spalling.

$$t_s = 1.25\ T_s \leq T_s + 10 \text{ (in inches)}$$

The results obtained by using the above formula are presented in Table 5.D-5.

The Ballistic Research Laboratory (BRL) formula was selected after a thorough study of all available formulae in the literature for concrete perforation and spalling due to missile impact. As with all other available formulae the BRL formula represents an empirical expression based upon high velocity test data and was developed for use in the high velocity range (i.e., missile impact velocity in excess of 1,000 ft/sec). The range of missile velocities considered in a nuclear facility is generally below 500 ft/sec. In order to provide a confidence margin for the lower velocity range, and to assure that barrier thickness would exceed that at which perforation or spalling impends, the design thickness was increased.

Test data on the impact of a one-inch diameter steel rod having a velocity from 150 ft/sec to 320 ft/sec on a concrete barrier of 3 inches, 6 inches and 9 inches in thickness indicate that these formulae provide conservative results for both concrete perforation and spalling in the velocity range as stated. A summary of the test results is presented in Figure 5.D-6.

A procedure for determining thickness of spalling is presented in Reference 4. The spalling effects on concrete wall due to the impacts of wooden plank and utility pole were investigated and in both cases, no spalling of concrete wall were indicated. Therefore, the secondary missiles are not considered creditable.

The thickness, t_s, of a concrete element required to prevent spalling, is more than the thickness, T_m, of a concrete element furnished in the cases of wooden plank and utility pole, as indicated in Table 5.D-5. The thickness, t_s, provides a simplified approach of determining a thickness required for a concrete barrier to stop a missile. A margin of safety, an increase of 25% of the calculated values with an upper limit of 10 inches, is a logical safety factor against spalling or perforation and is further reinforced by the test data presented in Figure 5.D-3. The formula used to determine the thickness of spalling does not consider reinforcing steel which tends to reduce the amount of spalling. If t_s is less than T_m, as in the case of 3" steel rod and 6" pipe, no additional analysis is required.

To determine the thickness of spalling, the following formula, as presented in Reference 4.

$$X_{max} = \frac{282\ N\ W}{f'c\ d^{1.8}} \left(\frac{V}{1000}\right)^{1.8}$$

$$C_1 = \left(\frac{T}{X_{max}} + C_3 - 1\right)\left(\frac{C_s}{V_s}\right)^{1/3}$$

$$C_3 = \frac{0.877\ W^{1/3}}{X_{max}}$$

$$C_2 = \left(2.8 - C_3\right)\ C_1^{1.38} \Big/ 30$$

$$\Delta_t = C_2\ X_{max}\ \frac{V_s}{C_s}^{1/3}$$

where:

W = weight of missile, pounds

V = missile velocity, ft/sec

T = target thickness, inches

d = diameter of missile, inches

$f'c$ = concrete strength, psi

N = nose factor = 0.845 for hemi-spherical nose

C_s = dilational velocity in concrete = 9,800 ft/sec

By substituting the following values into the equations, the value of Δ_t, thickness of spalling, for the wooden plank and the utility pole was determined to be insignificant.

Wooden Plank	Utility Pole
W = 105 pounds	1,500 pounds
V = 280 fps	182 fps
T = 12 inches	24 inches
d = 7.82 inches	13.5 inches
$f'c$ = 3,000 psi	3,000 psi