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RECIP.NAME RECIPIENT AFFILIATION
VASSALLO,D.B. Operating Reactors Branch 2

SUBJECT: Forwards response to 830726 request for addl info re proposed expansion of spent fuel pool.Cask drop protection sys within pool consists of dashpot supported vertically by pool floor.

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NIAGARA MOHAWK

NIAGARA MOHAWK POWER CORPORATION/300 ERIE BOULEVARD WEST, SYRACUSE, N.Y. 13202/TELEPHONE (315) 474-1511

August 5, 1983

Director of Nuclear Reactor Regulation
Attention: Mr. Domenic B. Vassallo, Chief
Operating Reactors Branch No. 2
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Re: Nine Mile Point Unit 1
Docket No. 50-220
DPR-63

Gentlemen:

Your letter of July 26, 1983 requested additional information regarding the proposed expansion of the spent fuel pool at Nine Mile Point Unit 1. In addition, item 14 and the preliminary calculations, addresses the question raised in the meeting on July 25, 1983 with members of your staff. The attachment to this letter responds to your request.

Very truly yours,

C. V. Mangan
Vice President
Nuclear Engineering and Licensing

CVM/MGM:bd

A001

1/1

NRC REQUEST FOR ADDITIONAL INFORMATION
NINE MILE POINT SPENT FUEL POOL
EXPANSION - STRUCTURAL ASPECTS

1. Question

Referring to paragraph 4.2 (Base Input Records), page 13 of the enclosure to your letter of June 24, 1983 (entitled "Supplemental Submittal"), do the time histories used for the design of the racks and the analysis of the pool produce response spectra which envelope the design floor response spectra for the Nine Mile Point 1 spent fuel pool at the proper elevation?

Response

The time histories and floor response spectra as used in the rack seismic analysis are for the appropriate floor elevations. The design floor response spectra was taken at the Reactor Building floor Elevation 298. The base input time history was taken from site design acceleration records for Nine Mile Point Unit #2 which envelope site design spectra recommended in Regulatory Guide 1.61 for a base acceleration of 0.15g. The site design base acceleration for Unit #1 is 0.11g.

Synthetic time histories were developed that envelope the broadened floor response spectra at Elevation 298 for a base acceleration of 0.11g. The comparison of these results to the above are shown in U.S. Tool & Die Report #8202-00-0108 Appendix "G".

2. Question

Referring to the same document as noted in item 1 above, paragraph 5.2, sub-paragraph 2, page 18, the justification statement for not using the load factors of the SRP is not a justification at all. All that is stated is that if the factors are not reduced, the structures will be overstressed. Provide a proper justification or increase the loads.

Response

ACI 349-76 specifies load factors and load combinations for nuclear service. Therefore, it is a consistent and logical extension to the original design code, ACI 318-63. The current finite element simulation and extensive post-processing represent significant enhancements to the spent fuel pool structural evaluation. In addition, the spent fuel pool evaluation was performed for conservative seismic loading conditions. For example, the vertical and horizontal seismic loads from all spent fuel racks were assumed to reach their maximum values at the same time, resulting in a worst case loading for the spent fuel pool floor. These calculation methods justify use of the lower load factors in ACI 349-76 as compared to the load factors in the Standard Review Plan. Also, see the response to question #13.

3. Question

Referring to the same document as noted in item 1 above, paragraph 5.3, page 20, provide a justification for using level D acceptance criteria for the pool liner. By definition, level D service limits allow gross deformations. Such deformations could result in failure of the liner which is a key component of the seismic Category I structure. What is the basis for the statement that the "liner integrity will remain" since buckling is postulated?

Response

There are no rules in ACI 349-76 for pool liners. Therefore, Section III of the ASME Boiler and Pressure Vessel code was used. The loads in the pool liner due to spent fuel rack horizontal seismic loads applied to the existing swing bolt devices and new support lugs welded to the pool liner are considered primary loads. The stresses in the pool liner due to these loads are within the primary stress limits (Sm) of the ASME code. The loads (or strain) in the pool liner due to gross bending of the spent fuel pool floor slabs and differential thermal expansion between the pool liner and the concrete floor are considered secondary loads. The stresses in the pool liner due to these loads, when combined with the stresses due to primary loads, are within the primary plus secondary stress limits (3Sm) of the ASME code. Furthermore, it should be noted that secondary loads result in bi-axial compressive stresses in the liner, virtually precluding the possibility of tensile rupture. As a consequence of this compressive loading, the liner could warp out-of-plane or "buckle" locally. Warping would relieve the secondary stresses in the liner. The resulting out-of-plane warping of the liner would be small (0.50") and not affect local cooling flow.

4. Question

Referring to the same documents as noted in item 2 above, paragraph 5.2, page 17, 3rd paragraph, the cask drop protection system is classified as both dead load and live load. Please clarify, or correct this apparent discrepancy.

Response

The Spent Fuel Pool analysis specification classifies the Cask Drop Protection System as a dead load and the spent fuel shipping cask as a live load. Both the Cask Drop Protection System and cask were treated as live loads in the Spent Fuel Pool analysis. This is conservative, since the load factor on live load is 1.7 versus 1.4 for dead load.

5. Question

Referring to U.S. Tool and Die Co. report numbered 8202-00-0109 dated May 20, 1983, revision 2, section 5.2, pages 5 and 6, the analysis of base plate loads is questioned because:

a. It is understood that the racks were considered "rigid" for the purpose of determining a seismic load. This practice should be followed consistently and not abandoned in an effort to show reduced floor loads. The analysis on page 5 appears to be inconsistent and additional explanation is required.

b. Although the bearing area on page 6 is very conservatively arrived at, the bearing stress in the concrete is considered unacceptable by any reasonable code interpretation. Additional explanation and justification must be provided.

Response

The calculations on pages 5 and 6 indicate an order of magnitude estimate of the stresses at the pedestal-liner and liner-concrete interfaces. The concrete stress so calculated is not directly comparable to ACI Code allowables.

The analysis assumes that all parts of the structure (the pedestal, base plates and floor slab) are "rigid," i.e., no deformation occurs. Knife-edge loading, theoretically infinite, causes deformation and load redistribution over larger areas. Localized crushing in the concrete and liner deformation reduces the loads, whereas liner strain and concrete compressive stresses would not be excessive.

6. Question

Referring to the same document as noted in item 5 above, table 1.2, Stress Summary, what are the tabulated values for "Pool Floor Load at Pedestal" due to SSE?

Response

Pool floor bearing stresses are compared to ACI 349-76 allowables. ACI 349-76 defines the loading combinations to be considered. The loading combinations from the rack pedestals in bearing can be reduced to two combinations. Refer to U.S. Tool & Die Report 8202-00-0019 Rack Mechanical Analysis Section 5.2 and attached Supplement 1 to Report dated 7/23/83.

7. Question

Referring to U.S. Tool and Die Co. report numbered 8202-00-0215 dated May 1983, Volume 1 of 3, sheet 7, buckling of the pool liner is postulated as a worst case condition. Will such buckling damage the liner and cause it to leak? Provide the analytical basis for your answer.

Response

See response to Item #3.

8. Question

Referring to the same document as noted in item 7 above, sheet 33, provide an explanation for the use of two factors 1.7 and 1.15 for OBE loads in the table. Where and why is each factor used?

Response

The OBE load factors, 1.7 and 1.15, are as specified in ACI 349-76. These factors are used as shown on sheets 138 thru 140 of U.S. Tool & Die Report #8202-00-0215 dated May 1983 Vol. I.

9. Question

Referring to the same document as noted in item 7 above, sheet 98, the analysis indicates that the factor-of-safety against cask tipping is very close to one. NRC staff criteria is that the factor of safety against tipping should be at least 1.1 for this type of analysis. Justify the discrepancy.

Response

The Cask Drop Protection System within the pool consists of a dashpot which is supported vertically by the pool floor and horizontally by struts embedded in the pool walls. While a cask is contained within the dashpot, overturning is prevented by the dashpot supports.

10. Question

Referring to the same document as noted in item 7 above, page 71, it is stated that the reaction of the rack impacting the pool floor is less than the static reaction of the reaction on the other side of the rack as it tips up. Shouldn't the impact load and the seismic reaction be directly added? Please furnish a detailed explanation including a numerical summary of results of the nonlinear analysis.

Response

The non-linear seismic analysis of the racks show the racks lift-off in all cases including Operating Basis Earthquake. The maximum lift-off occurs simultaneously with the maximum vertical reaction on opposite pedestals. Generally, the maximum horizontal reaction for the cases considered occurs about the same time (within 0.20 seconds). Therefore, the seismic design loads are taken with the maximum vertical reaction acting concurrently with the maximum horizontal reaction in either direction over the whole time history. Pedestal impacting causes maximum vertical loading on the pool floor. Refer to the U.S. Tool & Die Reports 8202-00-0108 "Seismic Analysis - Spent Fuel Racks, 8202-00-0109, "Mechanical Analysis," 8202-00-0215, "Spent Fuel Pool Structural Analysis" see pages 5 thru 9 of the Seismic Analysis for a numerical summary of the results of the nonlinear analysis.

11. Question

Referring to the same document as noted in item 7 above, page 136, the statement at the bottom of the page is considered to be unacceptable. Essentially, the author is saying that a certain criteria will be used until it cannot be met and then the criteria will be changed. Also, it is not understood why the SRSS result was assigned a sign, i.e., for purposes of determining load the sign is positive and directly additive to other effective loads. Additionally, the statements conflict with those made on sheet 5. Provide a complete justification for the procedure used and include an assessment of how it has affected the results of the analysis.

Response

The conventional square root sum of the squares (SRSS) method assures a linear structure for which the responses to seismic loads of opposite direction are equal in magnitude but of opposite sign. Due to the non-linear method of support for the spent fuel racks, loads on the spent fuel pool floor due to horizontal seismic loads of opposite direction are not equal in magnitude. The question becomes which component, east, west, north or south, should be used in the SRSS? The absolute maximum east or west, north or south were used, making the resulting evaluations conservative and less complex. This required executing the three dimensional finite element model of the spent fuel pool five times for seismic loads once in each direction (four horizontal and one vertical). However, if the resulting SRSS proved too conservative at any location, the logically more appropriate component was used in formulating the SRSS at such locations. It was only necessary to do this once, as shown on sheets 103-109 of DDI-TR-82-120-2 Volume II. In all cases, the SRSS was taken to be both plus or minus when forming the load combination, since it was not possible to judge prior which sign, plus or minus, would give the worst case.

12. Question

Referring to U.S. Tool and Die Report numbered DDI-TR-82-120-2, dated May 1983, "Volume II, Component Structural Evaluations, page 102, provide an expanded explanation of why only positive components were used in the SRSS.

Response

See response to question #11.

13. Question

Why was both ACI 349-76 and ACI 318-71 used for the analysis of the pool? Where did your criteria deviate from the requirements of ACI 349-76? Provide a detailed discussion.

Response

ACI 349-76 was used for the load factors and load combinations. Since ACI 349-76 code requirements are similar to the ACI 318-77 code, specification of ACI 318-77 is somewhat redundant. There are no deviations from the requirements within ACI 349-76. Also, see the response given in Question No. 2.

14. Question

NRC request to provide additional analysis on maintaining liner integrity under global loading (buckling) and rack local loading conditions.

Response

Refer to attached preliminary calculations dated July 27, 1983, Project No. 8202 sheets 1-10.

BY Di Pirale DATE 7/27/83 SUBJECT NMP 1 - PHASE II SHEET ____ OF ____
CHKD. BY ____ DATE ____ SUPPLEMENT TO REPORT 8202-00-0109 PROJ. NO. 8202

NRC Question 6

The "Pool Floor Loading at Pedestal" due to SSE was not listed in Table 1.2 of Mechanical Report #8202-00-0109. Following are calculations for both SSE and OBE based on ACI 349-76 load combinations and allowables. Seismic loads are from Table 4-3, Report 8202-00-0109.

(OBE) $P_u = 1.4D + 1.7L + 1.7E$
(SSE) $P_u = D + L + E'$

$$E = \sqrt{(E_{N-S})^2 + (E_{E-W})^2 + (E_{VT})^2}$$

$$= \sqrt{(77)^2 + (67)^2 + (7)^2} = 102.3 \text{ KIPS}$$

$$E' = \sqrt{(E'_{N-S})^2 + (E'_{E-W})^2 + (E'_{VT})^2}$$

$$= \sqrt{(87)^2 + (101)^2 + (13)^2} = 133.9 \text{ KIPS}$$

The rocks are conservatively assumed to be live loads, therefore $D = 0$

(OBE) $P_u = 1.7(78) + 1.7(102.3) = 306.5$ KIPS

$$\sigma_u = \frac{P_u}{A} = \frac{306.5}{11.75 \times 14.5} = \underline{\underline{1.80 \text{ KSI}}}$$

(SSE) $P_u = 78 + 133.9 = 211.9$ KIPS

$$\sigma_u = \frac{P_u}{A} = \frac{211.9}{11.75 \times 14.5} = \underline{\underline{1.24 \text{ KSI}}}$$

Allowable Bearing: $0.85 \phi f'_c$ (ACI 349-76, Sect. 10.14)

$$= 0.85(0.7)(3500) = \underline{\underline{2.08 \text{ KSI}}}$$

(OBE) $F.S. = \frac{2.08}{1.80} = \underline{\underline{1.16}}$

(SSE) $F.S. = \frac{2.08}{1.24} = \underline{\underline{1.68}}$

RD 7/29/83

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BY [illegible] DATE 7/27/83 SUBJECT NMP1-II SHEET NO. 1 OF 10

CHKD. BY _____ DATE _____ QUEST 14 PROJ. NO. 8202

RESPONSE TO QUESTION BROUGHT UP BY NRC IN JULY 25, 1983 MEETING TO ADDRESS LINER STRESSES DUE TO BOTH GLOBAL LOADING (FROM POOL ANALYSIS) AND LOCAL LOADING (FROM RACKS) - CALL THIS QUESTION #14

REFERENCES: USTAD REPORT 8202-00-0109 "MECHANICAL ANALYSIS" SECTION 5.7

USTAD REPORT 8202-00-0215 "SPENT FUEL POOL STRUCTURAL ANALYSIS - VOLUME 3

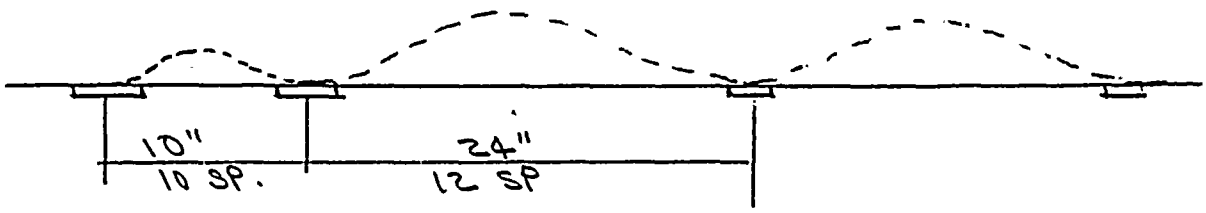
THE SPENT FUEL POOL STRUCTURAL ANALYSIS COMPUTES LINER STRESSES DUE TO THERMAL & GLOBAL DWT & SEISMIC LOADING.

THIS CONCLUDES THAT THE LINER IS STRESSED TO - 32,700 PSI, MOSTLY FROM THERMAL LOADINGS (T_a = 160°F). STRAINS IN BOTH E-W & N-S DIRECTIONS ARE APPROXIMATELY EQUAL TO 8.00×10^{-4} IN/IN.

BUT THE LINER IS A FLEXIBLE ELEMENT IN COMPRESSION & CAN THEREFORE BUCKLE. BUCKLING IN THIS INSTANCE DOES NOT CONSTITUTE "FAILURE" SINCE THE LINER IS NOT A STRUCTURAL LOAD CARRYING ELEMENT.

THE LINER IS A 1/4" THICK PLATE WHICH IS WELDED TO 5"x3/4" PLATES RUNNING N-S IN THE POOL, WHICH ARE IN TURN ATTACHED TO THE POOL FLOOR CONCRETE SLAB BY 5/8" - Φ ANCHOR BOLTS.

IN THE E-W DIRECTION THE MIN LINER SPAN IS $10\frac{1}{2}$" & MAX IS 24 9/16" WITH AN AVERAGE OF 23 13/16". SINCE THE PLATE IS BACKED UP BY CONCRETE, IT WILL BUCKLE IN THE FOLLOWING CONFIGURATION:



EULER BUCKLING STRESS:

$$\sigma_{CR} = \frac{\pi^2 E}{(KL/r)^2}$$

SINCE ENDS ARE ROTATIONALLY RESTRAINED, $K = 0.5$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{bt^3/12}{bt}} = \sqrt{\frac{t^2}{12}} = \sqrt{\frac{(0.25)^2}{12}}$$

$r = 0.072$ IN.

FOR 10" SPAN:

$$\sigma_{CR} = \frac{\pi^2 (28\times10^6)}{[0.5(10)/.072]^2} = 57.300 \text{ PSI}$$

SINCE $F_y = 25{,}000$ PSI THE LINER WILL YIELD PRIOR TO BUCKLING.

FOR 24" SPAN $\sigma_{CR} = \frac{\pi^2(28\times10^6)}{[0.5(24)/.072]^2} = 9950$ PSI

∴ THE 24" SPANS WILL BUCKLE PRIOR REACHING THE FULL COMPRESSIVE STRESS OF −32,700 PSI. HOWEVER THE DISTANCE BETWEEN CHANNELS MUST STILL BE REDUCED BY $\epsilon L = .0008(24) = -.0194$ IN.

THE PLATE WILL SHORTEN BY: $\Delta = \frac{\sigma L}{E} = \frac{9950(24)}{28\times10^6}$
$= -.0085$"

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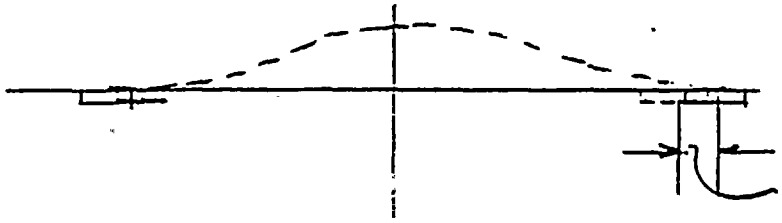
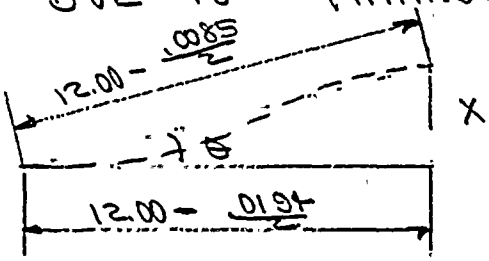
BY D. Franklin DATE 7/27/83 SUBJECT NMP1-II SHEET NO. 3 OF
CHKD. BY DATE QUEST 14 PROJ. NO. 8202.

THIS DIFFERENCE ACCOUNTED FOR BY BUCKLED SHAPE:



$D = \frac{1}{2}(.0194)$
$= .0097''$

THIS DUE TO "TRIANGULATION" OF DEFLECTED R:



$$\cos\theta = \frac{11.990}{11.9958}$$

$$\theta = 0.0301 \text{ RAD.}$$

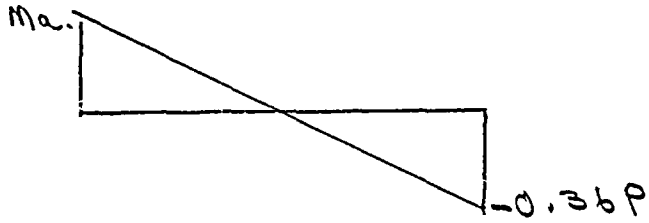
$$\tan\theta = \frac{X}{11.990}$$

$$X = 11.990(.03015) = 0.362 \text{ IN.}$$

ASSUME BUCKLED SHAPE IS IDENTICAL TO DEFLECTED SHAPE OR A BEAM LOADED BY A COMPRESSIVE LOAD & MOMENT EQUAL TO PY WHERE Y IS DEFLECTION.

$$M_o = M_a - PY$$

FOR 4" WIDE SECTION:
$M_o = M_a - 9950\,Y.$



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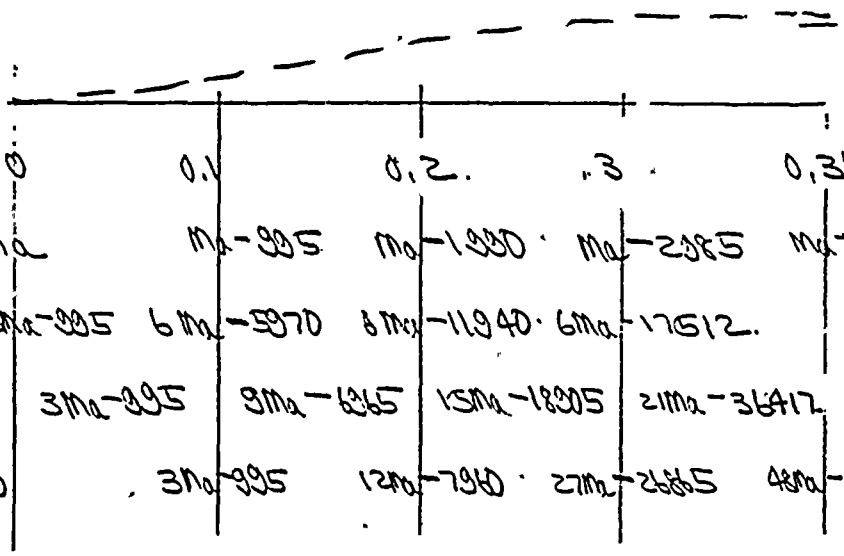
BY D. Paolo DATE 7/27/83 SUBJECT NMP1-II SHEET NO. 4 OF ___
CHKD. BY ___ DATE ___ QUEST 14 PROJ. NO. 7202.

USE THE NEWMARK METHOD TO COMPUTE Ma: WITH ASSUMED DEFLECTIONS:



ASSUMED Y	0	0.1	0.2	.3	0.36
M/EI	Ma	Ma-995	Ma-1990	Ma-2985	Ma-3582
EQUIV. CONC LOADS	3Ma-995	6Ma-5970	8Ma-11940	6Ma-17512	
θ	0	3Ma-995	9Ma-6965	15Ma-18905	21Ma-36417
$\Delta\left[\frac{1}{EI}\left(\frac{L}{4}\right)^2\left(\frac{1}{6}\right)\right]$	0	3Ma-995	12Ma-7960	27Ma-25865	48Ma-63282

$$\Delta = 0.36'' = (48Ma - 63282)\left(\frac{1}{EI}\right)\left(\frac{L^2}{16}\right)\left(\frac{1}{6}\right)$$

$$\frac{EI(16)(6)}{(12)^2}(.36) = 48Ma - 63282$$

$$(9.71\times10^4)(.36) + 63282 = 48Ma$$

$$Ma = 2046$$

$$Y_1 = \frac{3Ma - 995}{9.71\times10^4} = .052''$$

$$Y_2 = \frac{12Ma - 7960}{9.71\times10^4} = 0.17''$$

$$Y_3 = 0.29''$$

REPEAT CALC. USING NEW Y:

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BY DiPrinzio DATE 7/27/83 SUBJECT NMPI-II SHEET NO. 5 OF ___
CHKD. BY _____ DATE _____ QUEST 14 PROJ. NO. _____

Y	0	0.05	0.17	0.29	0.36
	M_a	$M_a - 517$	$M_a - 1692$	$M_a - 2886$	$M_a - 3582$
ECL	$3M_a - 517$	$6M_a - 3762$	$6M_a - 10171$	$M_a - 16818$	
θ	$3M_a - 517$	$9M_a - 4279$	14450	$21M_a - 31268$	
	0	$3M_a - 517$	$12M_a - 4796$	$27M_a - 19246$	$48M_a - 50514$

$(9.71 \times 10^4)(.36) = 48M_a - 50514$

$M_a \doteq 1780$

$Y_1 = .050$

$Y_2 = 0.17$ CLOSE ENOUGH.

$Y_3 = 0.30$

$\sigma_b = \frac{Mc}{I} = \frac{1780(.125)}{.052} = \pm 42800$ PSI

$\sigma = -9950 \pm 42800 = +32850, -52750$ PSI

THIS IS A CASE OF CONSTRAINT OF FREE END DISPLACEMENT. SUBSECTION NF DEFINES THE ALLOWABLES FOR THIS SITUATION AS BEING EQUAL TO 3 TIMES NORMAL LIMITS:

PLATE IN BENDING: $\sigma_A = 0.75\ S_y = 18750$ PSI

$\sigma_A = 3(18750) = 56250$ PSI

F.S. = 1.07.

WELDS AT EACH END ACT TO PROVIDE A BRACE POINT:

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BY L. FINN DATE 7/27/83 SUBJECT NMP1-II SHEET NO. 6 OF ___

CHKD. BY ___ DATE ___ QUEST 14 PROJ. NO. F202

FROM: SALMON & JOHNSON, "STEEL STRUCTURES: DESIGN & BEHAVIOR" INTEXT, 1971 PP 466-470

THE FORCE REQUIRED TO PREVENT BUCKLING

$$F = d_o \frac{K_{ideal}}{\left(1 - \frac{K_{ideal}}{K_{act}}\right)} \quad - \text{ EQN } 9.10.3$$

WHERE $K_{ideal} = \frac{2P_{cr}}{L}$ $\quad P_{cr} = \frac{\pi^2 E A}{(KL/r)^2}$

K_{act} = ACTUAL STIFFNESS PROVIDED USUALLY MUCH GREATER THAN K_{ideal}
ASSUME $K_{act} = 10\ K_{ideal}$

d_o = INITIAL ECCENTRICITY

$$\therefore F = d_o \frac{K_{ideal}}{\left(1 - \frac{1}{10}\right)} = 1.1\ d_o \left(\frac{2P_{cr}}{L}\right)$$

SINCE $d_o \approx 0.36 \rightarrow$ USE 0.50 $P_{cr} = 9950(A)$, $L = 24''$

$$F = \frac{1.1\ (.50)\ 2\ (9950\ A)}{24}$$

2-WELDS ARE SPACED AT 6":

$A = 6(.25) = 1.5$

$F = 684$ LB WHICH MUST BE CARRIED BY 2 5/8" Ø SPOT WELDS ATTACHING LINER TO IMBEDDED PLATE.

$$\sigma = \frac{F}{A} = \frac{684}{.31} = 2240 \text{ PSI}$$

ALLOWABLE TENSILE STRESS = 15 PSI DUE TO CONSTRAINT OF FREE-END DISP.
$\sigma_A = 3(15) = 45000$ PSI

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ENGINEERING ANALYSIS SERVICES

BY D. Prado DATE 7/27/83 SUBJECT NMP1-II SHEET NO. 7 OF ___
CHKD. BY ___ DATE ___ QUEST 14 PROJ. NO. 8202

ANCHOR BOLTS SPACED AT : 24" (2-⅝"-Φ)

$$P = \frac{1.1(.50)(2)}{24}(9950)(24/4)$$

$P = 2736$ LBS.

$$\sigma = \frac{2736}{.61} = 4490 \text{ PSI}$$

$\sigma_a = 3(0.6)(50) = 90000$ PSI

FOR To CASE : TEMP = 125° ; ΔT = 50°

ASSUME COMP. STRESS IN PLATE DUE TO LOADING IS IDENTICAL.

THERMAL DEFORMATION $= \frac{50}{85}(.0194) = .0114$ IN

DUE TO COMPRESSION OF PL : .0085

TRIANGULATING : $\cos\theta = \frac{12 - .0114/2}{12 - .0085/2}$

$\theta = .0156$ RAD

$X = 11.994 \cdot \tan\theta$
$= 0.187''$

ASSUME Y IS LINEAR & USE MOMENT AREA THEORY TO CALC. Ma.

$EI(.19) = 7.2 M_a - 9950(.19)(24)$

$M_a = 1000$ IN-LB

$$\sigma_b = \frac{Mc}{I} = \frac{1000(.125)}{.0052} = \pm 24{,}100 \text{ PSI}$$

SAME ALLOWABLE AS FOR To CASE, To: 56250

$$F.S. = \frac{56250}{(9950 - 24100)} = 1.65$$

DESIGN DECISIONS, INC. PITTSBURGH, PA 15241
ENGINEERING ANALYSIS SERVICES

BY D. Pnoin DATE 7/27/83 SUBJECT NMP1-II SHEET NO. 8 OF ___
CHKD. BY _____ DATE _____ QUEST 14 PROJ. NO. 8202.

IN THE N-S DIRECTION: PLUG WELDS ATTACH THE PLATE TO THE CHANNEL: 2 - 5/8"-Φ WELDS SPACED AT 3" OR 6". THE CHANNELS ARE IN TURN ATTACHED TO THE CONCRETE WITH 2 - 5/8"-Φ ANCHOR BOLTS SPACED AT 6" (18 SP) TO 2'-0 (14 SP)

USE SAME PROCEDURE:

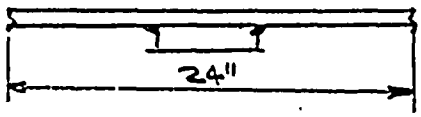
FOR PL IN 3" SPAN:

$$\frac{P_{CR}}{A} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (28 \times 10^6)}{(.5(3)/.072)^2}$$

$\sigma_{CR} = 600.000$ PSI ∴ MATERIAL WILL YIELD FIRST:

6" SPAN IS COMPARABLE:

HOWEVER THE PLATE & CHANNEL COMBINATION WITH A 24" SPAN CAN BUCKLE IN ITS OWN RIGHT:



$$\bar{Y} = \frac{5(.75)(.375) + 24(.25)(.875)}{8.75}$$

$\bar{Y} = 0.68"$

$$I = \frac{5}{12}(.75^3) + \frac{24}{12}(.25^3) + 3.75(.31)^2 + 6(.19)^2$$

$I = 0.78$ IN^4

$$r = \sqrt{\frac{I}{A}} = 0.28$$

$$\sigma_{CR} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 E}{(.5(24)/.28)^2} = 150.000 \text{ PSI}$$

∴ CONFIGURATION WILL ALSO YIELD,

HENCE WELDS & ANCHOR BOLTS ARE NOT STRESSED IN THIS DIRECTION.

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BY D. Parker DATE 7/27/83 SUBJECT MMF1 SHEET NO. 9 OF
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SWING BOLT CLEVIS LUGS ARE ALL ATTACHED TO THE LINER DIRECTLY ABOVE AN IMBEDDED PLATE.

SECTION 3.7 OF 8202-00-0103 SHOWS THAT STRESS IN LINER PLATE SURROUNDING CLEVISES IS NEGLIGIBLE & WILL NOT ADD TO VALUES CALCULATE IN THIS SECTION.
STRESS IN SPOT WELDS = 8.66 KSI SHR
2.77 KSI TENS.

WHEN THESE STRESSES ARE ADDED TO CALCULATED VALUE OF 2240 PSI, (TENS)

$\tau = 8.66 \qquad \sigma = 2.77 + 2.24 = 5.01.$

$\tau_{EQUIV} = \sqrt{\left(\frac{5.01}{2}\right)^2 + (8.66)^2} = 9.01.\ KSI$

$\tau_{ALL} = 3(10.0) = 30.0\ KSI$

TWO EASTERNMOST RACKS ARE SUPPORTED BY PLATES WELDED TO LINER WHICH IS IN TURN WELDED TO IMBEDDED PLATES. NORTHERNMOST OF THESE PLATES IS WELDED DIRECTLY ABOVE AN IMBEDDED PL WHICH DOES NOT AFFECT STRESSES PREVIOUSLY CALCULATED.

SOUTHERN PLATE IS WELDED TO LINER WHICH MUST TRANSMIT LOAD TO PLUG WELDS. LINER IS STRESSED IN SHEAR (2.71 KSI) WHEN THIS IS ADDED TO PREVIOUSLY CALCULATED LINER STRESSES (PAGE 5 OF THIS SECTION) DIFFERENCES ARE NEGLIGIBLE.

STRESSES IN PLUG WELDS: FROM SECT 3.7

$\tau \approx 10.0\ KSI.$

PAGE 6 OF THIS SECTION: $\sigma = 2.24\ KSI$

$\tau = \sqrt{\frac{2.24}{2} + 10.0} = 10.1\ KSI$

$\tau_{ALL} = 3(10.0) = 30.0\ KSI \quad \therefore O.K.$

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CHKD. BY _____ DATE _____ QUEST # 14 PROJ. NO. 8202

THE PLATE ALSO ACTS TO REINFORCE LINER SO THAT LINER PROBABLY WILL NOT BUCKLE IN THIS AREA. SO THAT PLATE WILL BE STRESSED TO ITS YIELD POINT IN BOTH E-W & N-S DIRECTIONS. - WITHOUT BENDING.

$\sigma \cong 30.000$ PSI.

$\sigma_{ALLOWABLE} = 3(15.0) = 45.0$ KSI (UNIF. COMP.).

SUPERIMPOSING TWO EFFECTS - GLOBAL & LOCAL DOES NOT CAUSE ANY OVERSTRESS IN LINER.