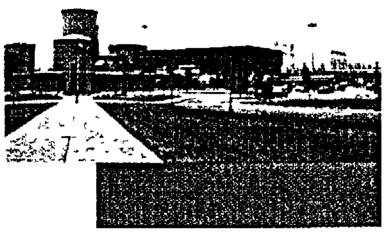
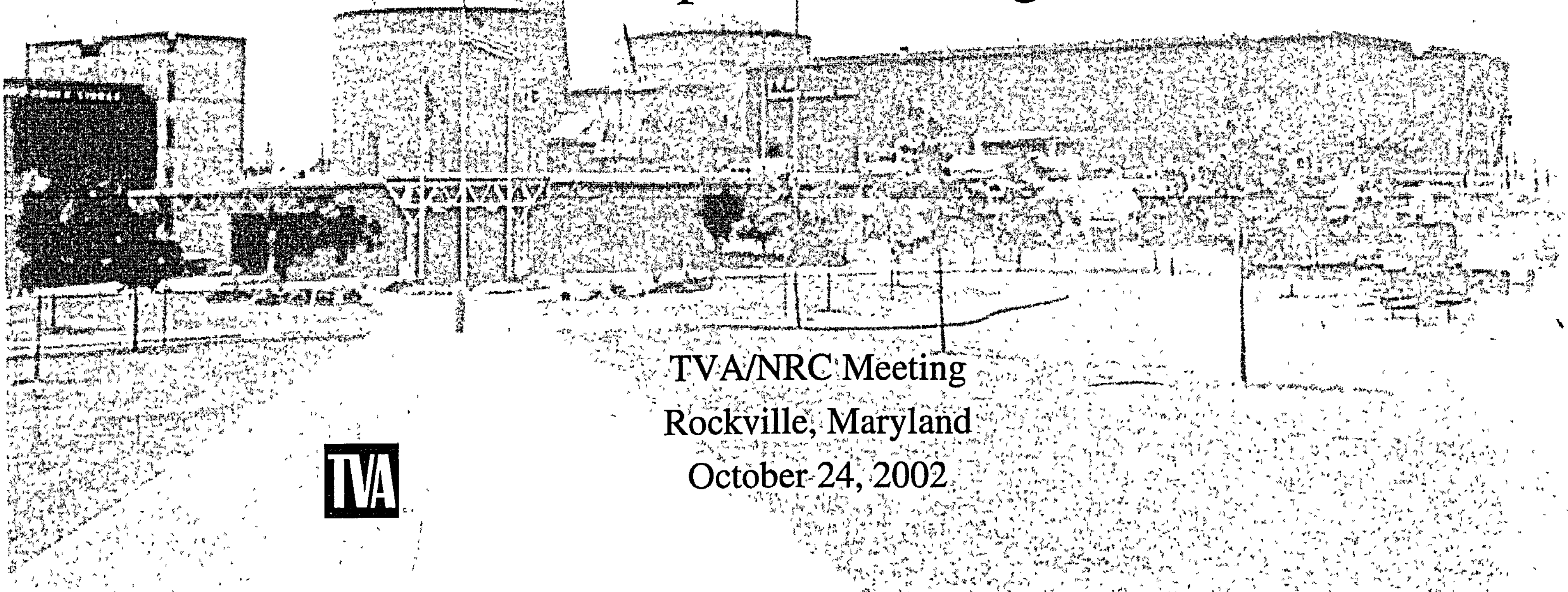
Sequoyah Nuclear Plant



Steam Generator Replacement Project
Topical Meeting



TVA/NRC Meeting
Rockville, Maryland
October 24, 2002

Agenda

Introduction	Pedro Salas
Rigging and Handling Program Presentation	Myron Anderson
Discuss Rigging and Handling RAI questions	Myron Anderson et al
Roof Modification (Splice Plate Design) Presentation	Bill Johnson
Discuss Roof Modification RAI Questions	Bill Johnson Myron Anderson
Discuss Bar-Lock Mechanical Splice RAI Questions	J. V. Smith Paul Trudel
Conclusion	Pedro Salas

Introduction

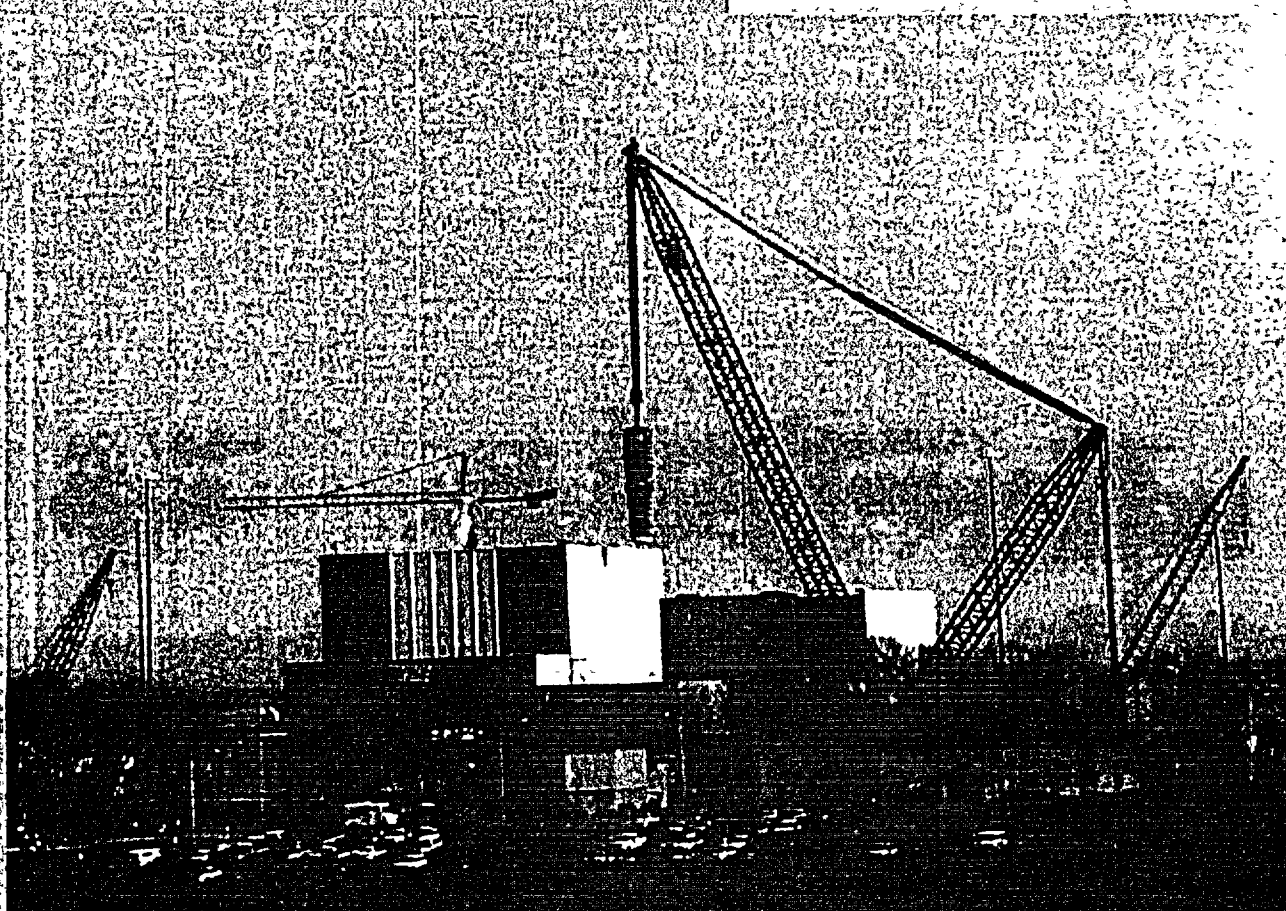
- Topics for Today's Meeting
  - Discuss TVA's Response to 39 RAI Questions
  - Two Presentations
    - Rigging and Handling Program (followed by discussion of RAI questions)
    - Compartment Roof Modification - Splice Plate Design (followed by discussion of RAI questions)
  - Discuss Balance of RAI Questions for Bar-Lock Mechanical Splice

Rigging and Handling Program

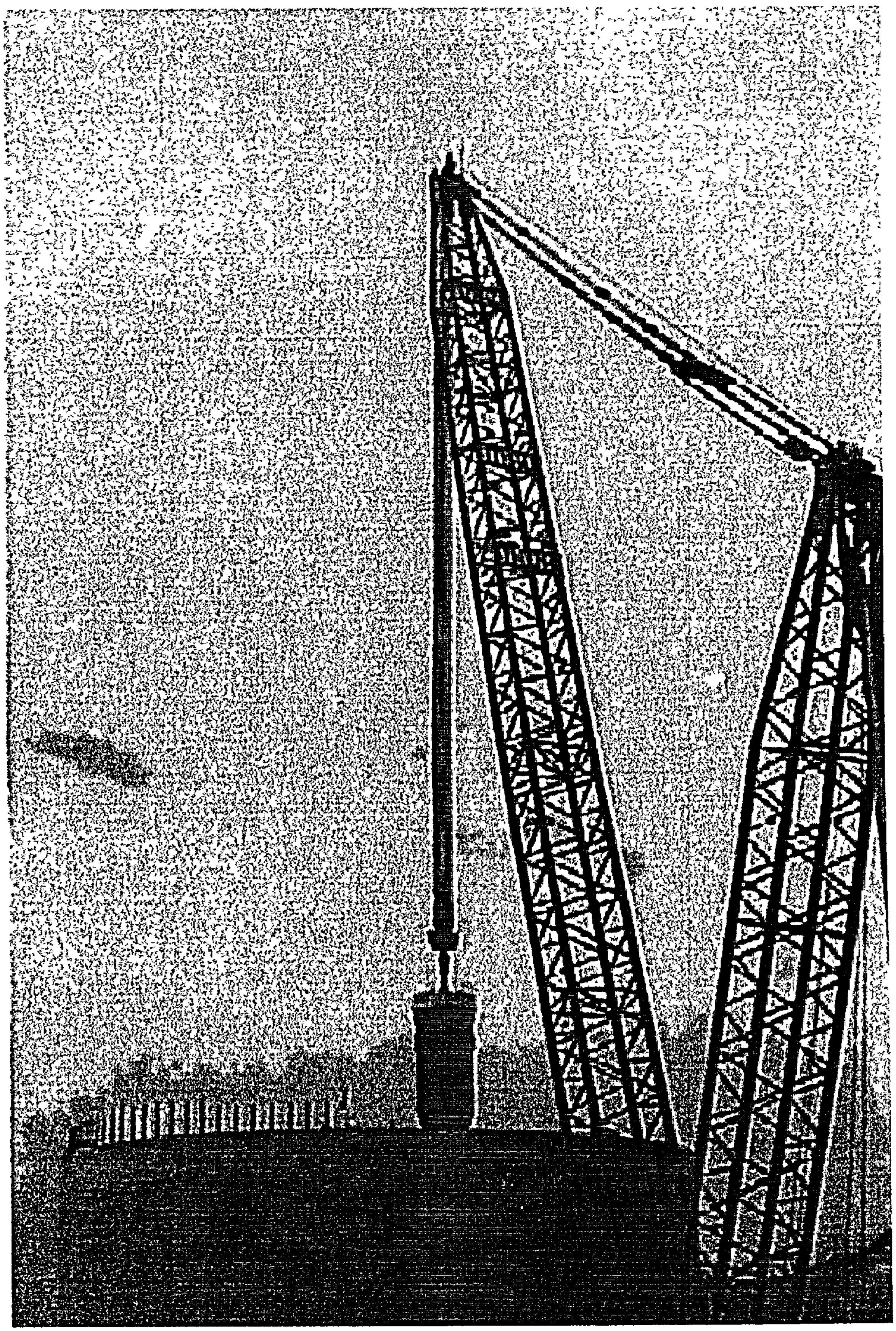
Rigging Program

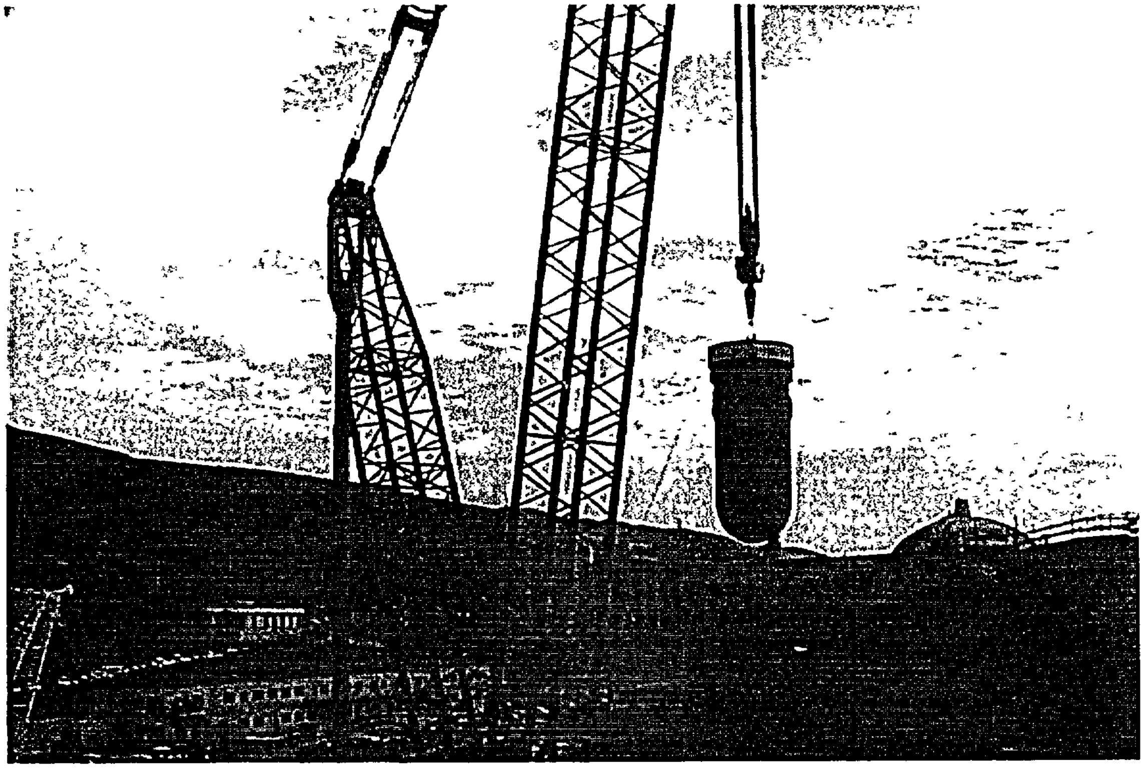
- Engineering Analysis
  - Codes and standards
  - Design products
  - Engineered packages
  - Equipment / Foundation
- Implementation
  - Personnel
  - Testing
  - Inspections / Calibrations
  - Work packages

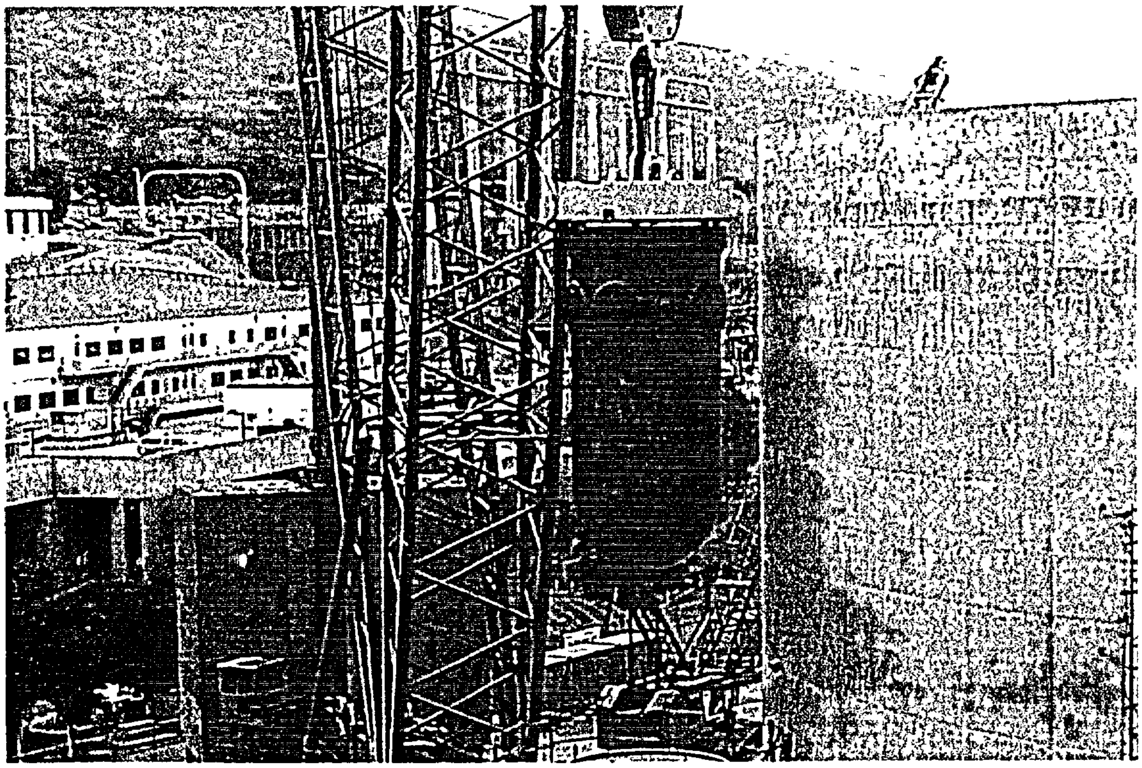
Steam Generator Replacement Project

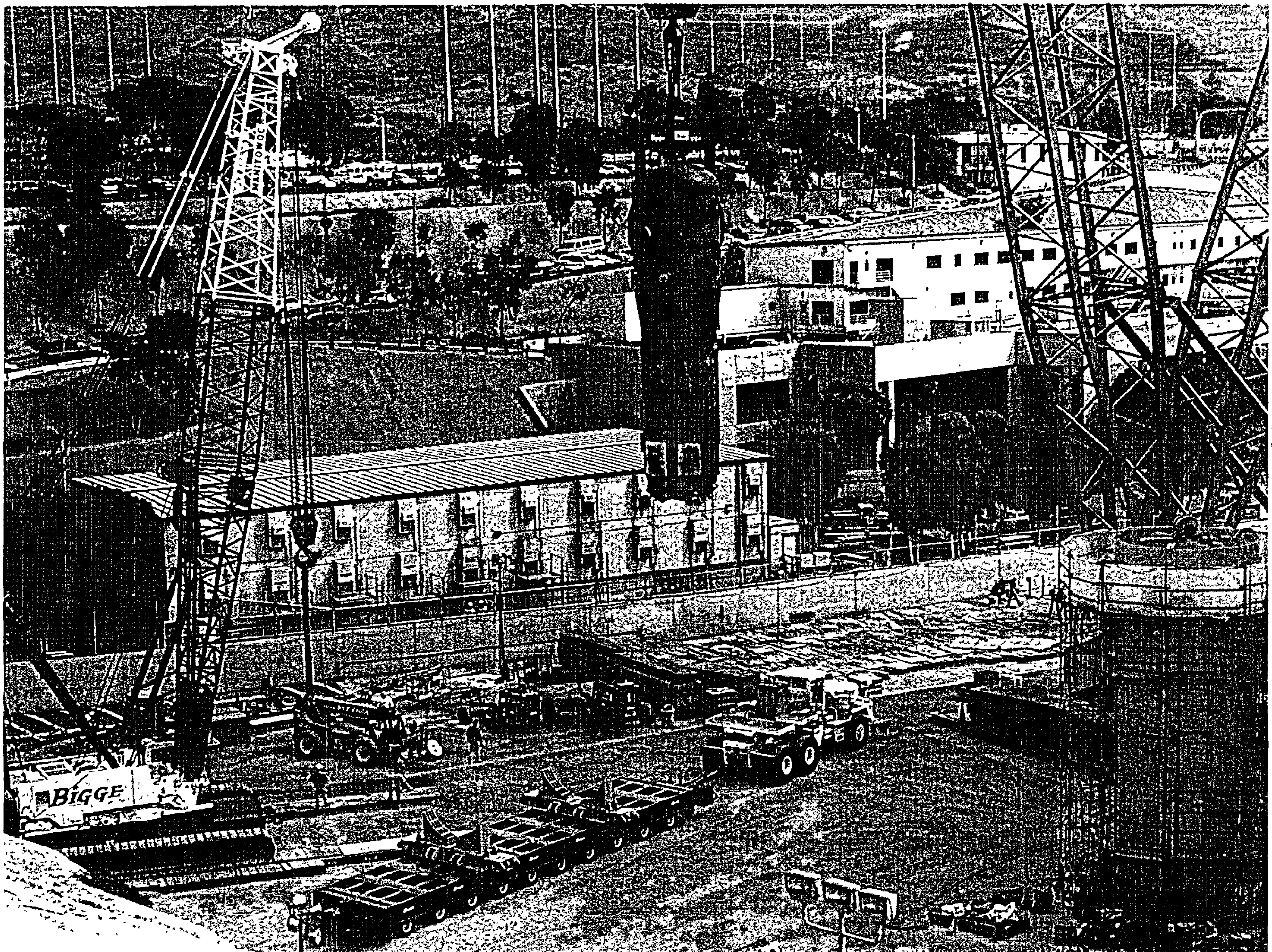


1996 Ginna Station





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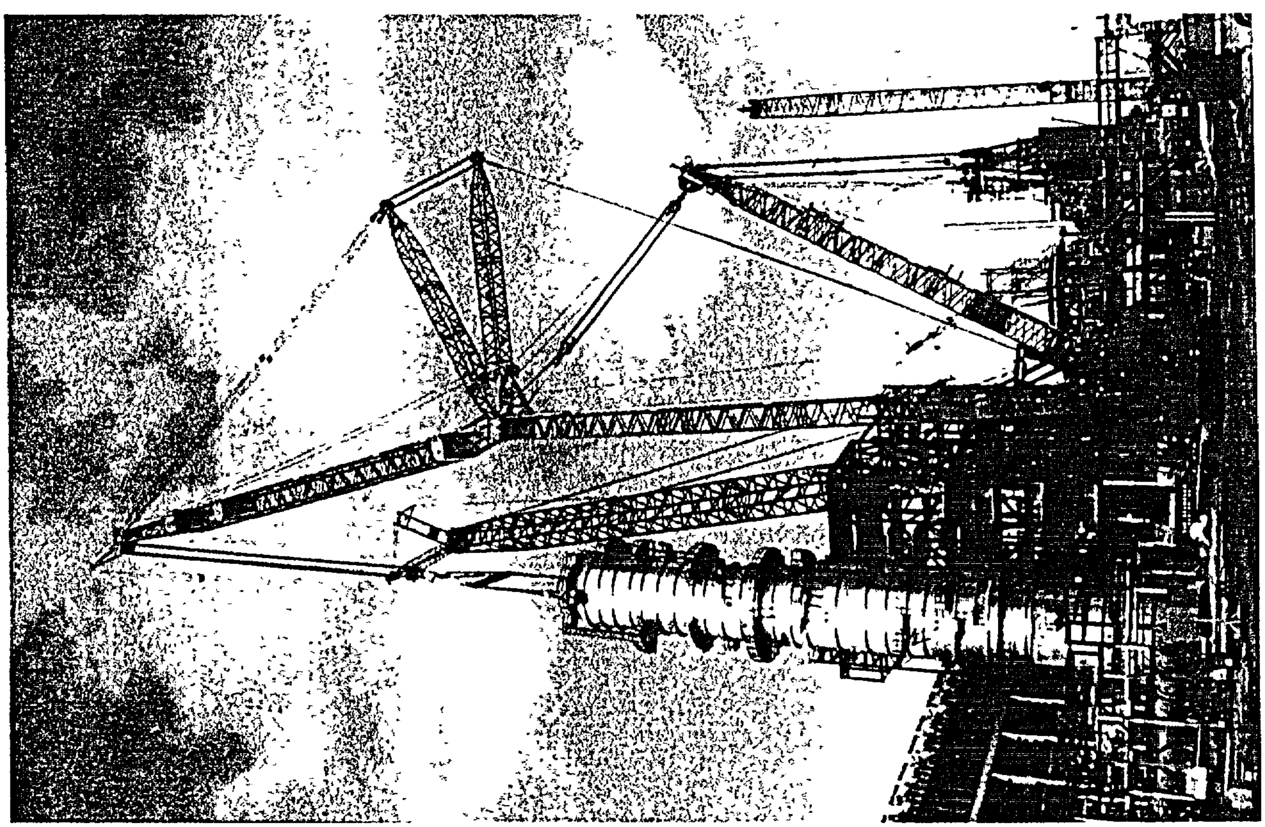

VAN SEUMEREN GROUP

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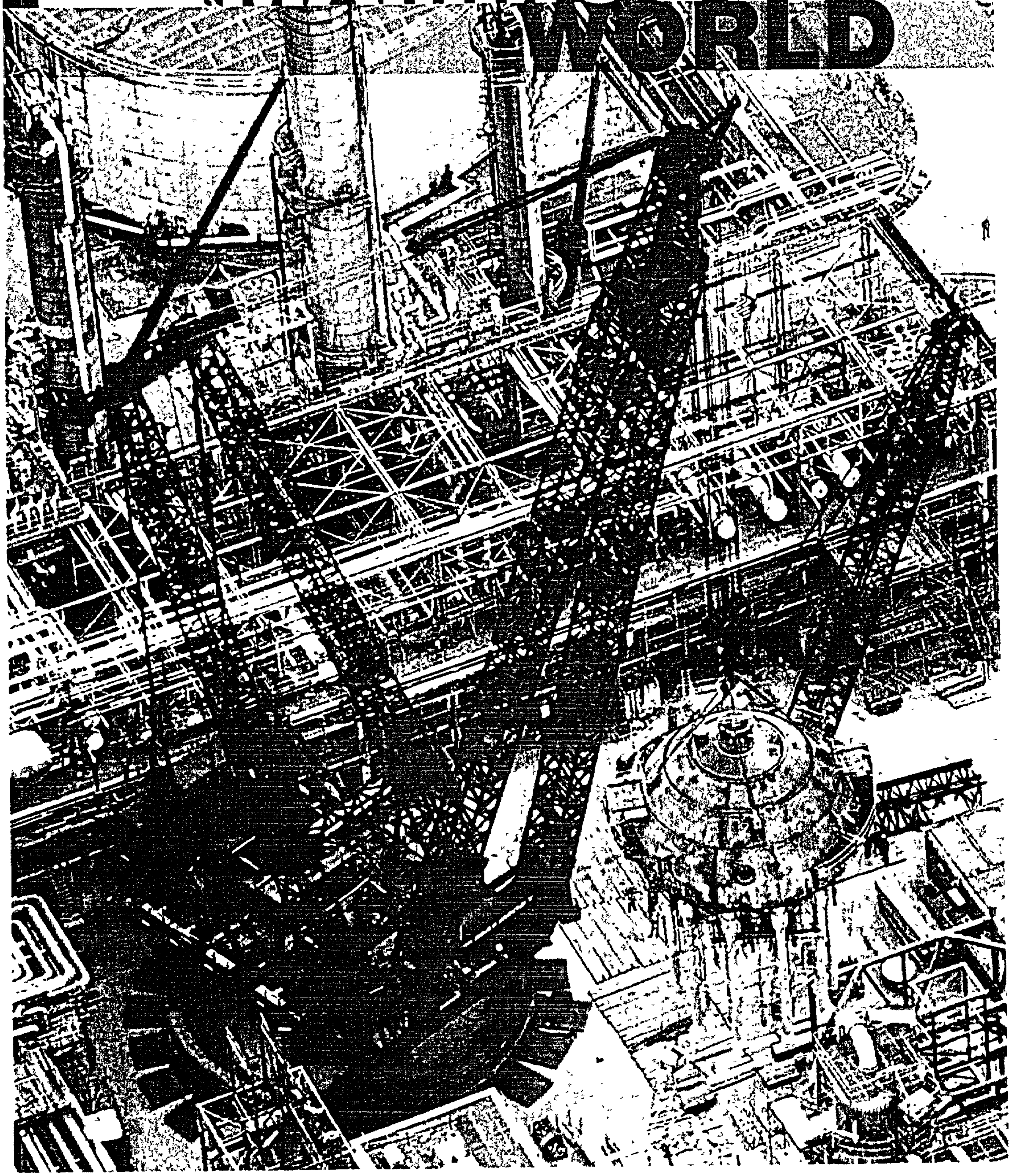
MAMMOET WORLD



Engineering Analysis

- Codes and standards
- Design products
- Engineered package
- Rugged Equipment / Foundation
- Work performed by registered PE's

Codes and Standards

- DIN (Deutsches Institut fur Normung)
- ASME NQA-1 Subpart 2.15
- ASME B30.5
- ACI 318
- AISC manual of steel construction
- Civil/structural design criteria

Design Documents

Detailed engineering analysis consisting of the following:

- Calculations
  - Generator weight
  - Load drop
  - OLS foundation
  - Up/downending equipment and foundation
  - Containment impact response
  - Haul route load drop
  - OLS seismic evaluation
  - Rigging devices
  - Geotechnical engineering properties
- Drawings
  - Load path
  - Equipment
  - Foundation

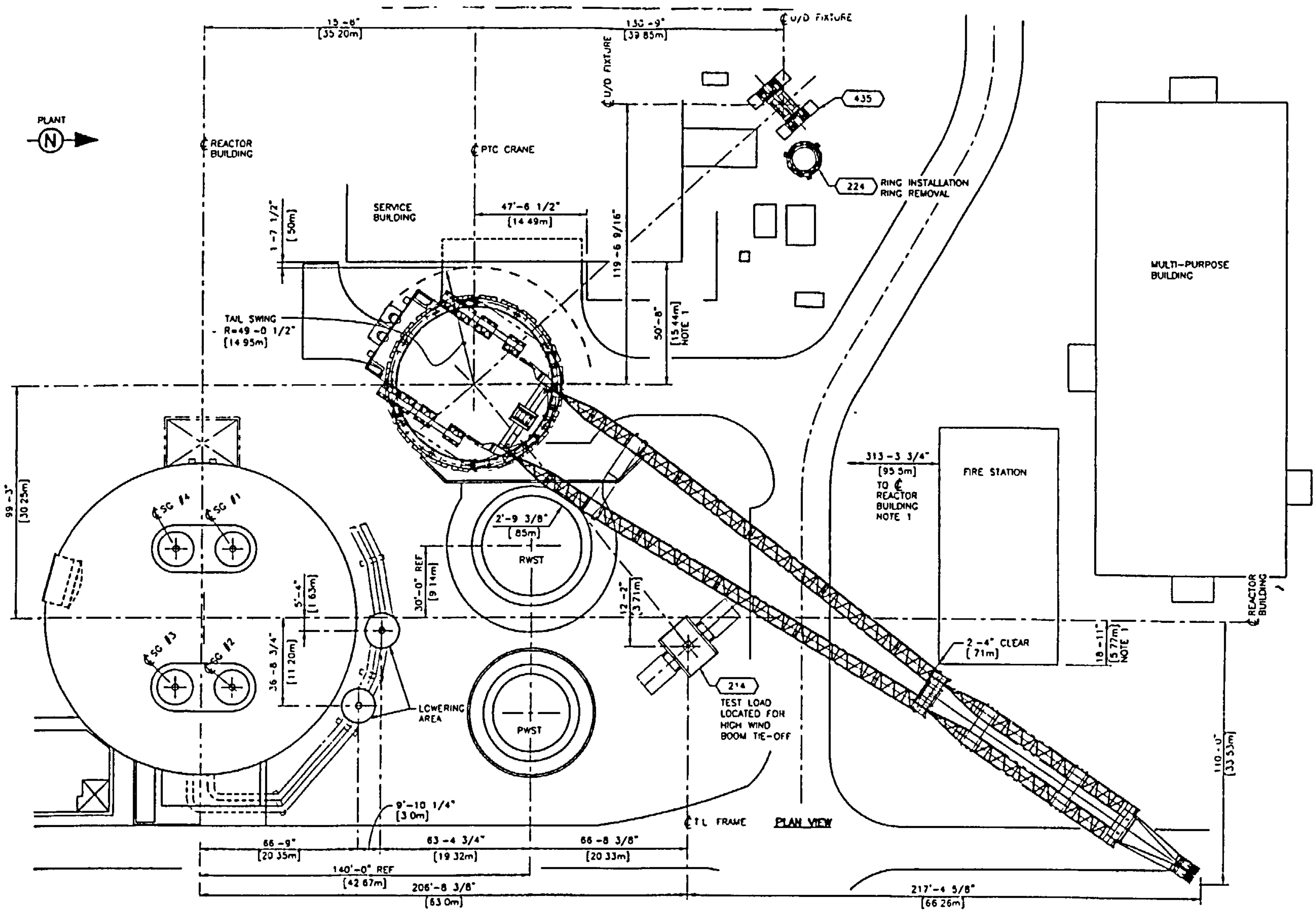
Engineering Package

The rigging operation is defined in packages which includes the following:

- Qualification of Rigging Crane (Outside Lift System [OLS])
  - Load
  - Wind
  - Seismic
  - Lift radius
  - Crane capacity
- Design for all associated rigging components
- Design foundation for maximum anticipated loads including seismic
- Documented load paths
- Load drop analysis
- Analysis of underground commodities
- Protection of SR underground commodities from a load drop
- Protection of commodities as required along haul route
- Compensatory measures for a postulated generator drop

Crane Lateral Load Capacity

- A-Frame main mast
- Lateral load on generator was calculated using a wind speed of 50 mph
- A comparison with this load and allowable OLS lateral loads indicates that even with a wind speed of 50 mph the wind force on the steam generator will be approximately 55% of the allowable wind force


PLANT
N
℄ REACTOR BUILDING
℄ PTC CRANE
℄ U/D FIXTURE
℄ U/D FIXTURE
15 -6"
[35 20m]
130 -9"
[39 85m]
435
224
RING INSTALLATION
RING REMOVAL
SERVICE BUILDING
47'-6 1/2"
[14 49m]
119 -6 9/16"
1 -7 1/2"
[50m]
50'-8"
[15 44m]
NOTE 1
TAIL SWING
R=49 -0 1/2"
[14 95m]
MULTI-PURPOSE BUILDING
FIRE STATION
313 -3 3/4"
[95 5m]
TO ℄
REACTOR BUILDING
NOTE 1
℄ REACTOR BUILDING
99 -3"
[30 25m]
℄ SG #4
℄ SG #1
℄ SG #3
℄ SG #2
2'-9 3/8"
[85m]
RWST
PWST
30'-0" REF
[9 14m]
12 -2"
[3 71m]
5'-4"
[1 63m]
36 -8 3/4"
[11 20m]
LOWERING AREA
214
TEST LOAD
LOCATED FOR
HIGH WIND
BOOM TIE-OFF
2 -4" CLEAR
[71m]
18 -11"
[5 77m]
NOTE 1
110 -0"
[33 53m]
℄ T L FRAME
PLAN VIEW
9'-10 1/4"
[3 0m]
66 -9"
[20 35m]
63 -4 3/4"
[19 32m]
66 -8 3/8"
[20 33m]
140'-0" REF
[42 67m]
206'-8 3/8"
[63 0m]
217'-4 5/8"
[66 26m]

Conservative Wind Speeds Used During Rigging Operations

- The OLS has been evaluated for wind effects up to 33 mph during the rigging operation
- Rigging operations conservatively limit to:
  - 22 mph when greater than 3 feet above ground
  - 33 mph when 3 feet or less above ground
- Rigging operations will be suspended for wind gusts above the stated maximum

Crane Evaluated So That Lifted Loads will Remain Within Chart Capacity

Important characteristics are:

- Maximum capacity: 1600 mt
- Maximum lifted load: 386 mt
- Maximum lift radius: 55 m
- Maximum chart capacity at above radius: 408 mt
- Maximum percentage of chart capacity: 94.3%

Crane Foundation

- Foundation for the crane has been designed for the imposed loads associated with the rigging operation including dead, live, wind, and seismic
- Ring foundation ~80 ft diameter, 8 ft wide, 4 ft thick pile cap on 71 piles into bedrock

Implementation

- Personnel
- Testing
- Inspections / Calibrations
- Work packages

A Person-In-Charge (PIC) is Designated to Oversee The Project Rigging

- PIC is designated by management and has proper supervisory experience
- Controls rigging activities
- Ensures that proper rigging procedures are being followed
- Ensures that good handling practices are being followed

Operator Qualification

- Operators will be qualified per Chapter 5-3, "Operation" of B30.5 for health and fitness requirements
- Operators are provided by the manufacturer trained to operate the OLS
- Operators are trained to adhere with applicable site procedures

Crane Testing Per ASME NQA-1, Subpart 2.15 601.2(a)

- Test per NQA-1 600
- Perform tests per applicable consensus standards (use ASME B30.5-2000)
- Functional test of crane per ASME B30.5
  - Load lifting/lowering mechanisms
  - Boom lifting/lowering mechanisms
  - Swinging mechanisms
  - Safety devices

Load Test Per ASME NQA-1, Subpart 2.15 601.2(a)

- Perform tests per applicable consensus standards (ASME B30.5-2000)
- Load test with a 550 kip test load with the boom extended such that the crane is loaded to 110% of crane capacity at that radius (ASME B30.5, Section 5-2.2.2(a) (1))
- Note: Crane has been load tested to 125% of maximum capacity when manufactured (Spring 2001)

Rigging Equipment Inspected Per Section 600 of ASME NQA-1, Subpart 2.15

- Crane manufactured in 2001
- Yearly periodic inspection was performed 9/27/02 and 9/28/02
- Onsite inspection per manufacturers users manual
- Items to be inspected onsite include: bearings, gearboxes, bolts, shafts, wires, structural members, welds, etc., at the intervals specified per the manufacturer

Equipment Calibration/Maintenance

- OLS crane components will arrive at the site in good working condition. The manufacturer will provide current maintenance documentation for the crane
- Calibration records submitted for instrumentation such as load cell and wind anemometer records
- Load cell calibration will be confirmed using 550 kip test load

Generator Weights Will Be Measured Prior To Performing The Lifts

- Weights have been calculated but will be measured prior to movement

Rigging Clearances

- Potential interferences within cubicles (piping, supports, platforms, etc.)
- Concrete enclosure opening
- Steel Containment Vessel (SCV) Plate
- Shield building opening
- Support frame on top of Shield building

Work Packages Developed For Each Rigging Task

- Specific work plan written for each activity dictated by engineering packages
- Plan is prepared by construction personnel who are responsible for performing task
- Plan is reviewed by Design Engineering to confirm that activities conform to engineering evaluations and analyses
- Plans have applicable acceptance criteria for critical tasks documenting satisfactory compliance
- Plans have inspection check points for QC sign-off documenting proof of satisfactory completion of critical tasks

Conclusion

- Detailed engineering analysis
- Qualified professionals
- Inspections and testing IAW procedures
- Controlled rigging operations

Changes to Rigging Topical

- Enveloping SG weight increased from 355 tons (322.7 mt) to 424.6 tons (386 mt)
  - Old weight included only SG; new weight includes SG, rigging, attached upper lateral restraint and attached insulation
- SG downending/upending method using offset foot trunnions instead of ring
- Mobile crane operating area may expand beyond 60 ft from OLS as justified by engineering analyses
- Commitment to realign CCW source for spent fuel pool cooling not necessary

RAI Questions 8 thru 35

- Topical Report No. 24370-TR-C-002, "Rigging and Heavy Load Handling"

(See Attached Questions and Responses)

8. The topical report discusses the dose consequences of dropping an original steam generator outside the containment. For staff to complete review of your dose consequences analysis, additional information is needed on the referenced calculation (Reference 23 of the topical) Provide the assumptions, inputs and methodologies used to determine the dose consequences of dropping the original steam generator. This should include the source term (isotopes and activities), control room ventilation system operation assumptions and the atmospheric dispersion factors (X/Qs) used in the dose calculation. Additionally, if the X/Qs are newly calculated and have not been reviewed by the staff, provide the inputs (including meteorological data), assumptions (including the location of the drop) and methodologies used to calculate the X/Q values.

SUMMARY OF DOSE CALCULATION

The purpose of the old steam generator (OSG) drop dose analysis was to determine the doses at the exclusion area boundary (EAB), the low population zone (LPZ), and the control room (CR) due to the failure of an OSG during the steam generator replacement (SGR) effort. The scenario postulated to cause the failure was a drop of an OSG from the Outside Lift System (OLS) crane or from the transporter at the worst location along the haul route between the containment and the Old Steam Generator Storage Facility (OSGSF). The dose analysis was performed using the following inputs, assumptions and methodology.

INPUT

1. *Based on surveys taken between 2/25/2000 and 3/1/2000 (3 to 10 days following shutdown), with the primary side of the OSGs full of water and the secondary side drained, the dose rate at a radial distance of 10 ft from the outside surface of the shell in the vicinity of the tube region (at elevation 723 ft) is 85 mR/hr.*

   (Note: Surveys taken during the November 2001 outage show the maximum dose rate at 3 ft from steam generator (SG) No. 3 is 85 mR/hr at El. 722 ft with the primary side full and the secondary side drained. Thus, the dose rate used in this analysis of 85 mR/hr at 10 ft under the same fill conditions is conservative.)

2. *Isotopic surveys for a number of components in the reactor coolant cleanup and radwaste systems during full power operation were performed. CVCS resin and drain tank residue represent the worst case distributions since they have the highest fractions of Co-60, the most dominant contributor to organ and whole body doses. The CVCS distribution was selected as bounding because it has higher amounts of Cs-134 and Cs-137 than the drain tank residue; these isotopes are also important dose contributors. Since the isotopic surveys were taken while the plant was at power and the SG external dose rate survey was only a few days after shutdown, the two surveys are well matched in time and thus no adjustment was necessary.*

3. *Dimensions of the tubes within the OSGs and of the tube region of the OSG were as reported in UFSAR Table 5.5 2-1.*

4. *The maximum accident atmospheric dispersion factors (X/Qs) are as follows:*

Release Point	Dose Point	X/Q (sec/m^3)	Reference
Containment	EAB	1.64E-3	UFSAR, Table 15A-2
Containment	LPZ	1.96E-4	UFSAR, Table 15A-2
Containment	CR	1.59E-3	UFSAR, Table 15.5.3-6
Haul Route or OSGSF	EAB	2.71E-3	See X/Q Calc Summary Below
Haul Route or OSGSF	LPZ	4.51E-5	See X/Q Calc Summary Below

5. The maximum breathing rate of persons offsite and in the control room is 3.47E-4 m^3/sec [Regulatory Guide 1.4, Sheet 2]. This rate was conservatively assumed for the duration of the accident.

6. Doses are calculated using the inhalation, air submersion, and ground deposition dose conversion factors (DCFs) in Federal Guidance Reports 11 and 12 [Ref. EPA-520/1-88-020 and EPA-402-R-93-081].

7. Based on experimental data and NRC recommendation, the structural shielding factor used was 0.75 for submersion and 0.33 for ground deposition [Ref. NUREG/CR-5164, Sheet 18; NUREG/CR-4551, Volume 2, Part 7 Sheet 3-28]. These factors account for the shielding provided by buildings and other structures during normal activities.

8. Based on experimental data for various aerosol compositions and sizes and various deposition surfaces, the mean ground deposition velocity used was 0.3 cm/sec [NUREG/CR-4551, Volume 2, Part 7, Sheet 2-21].

ASSUMPTIONS

1. It was assumed that 90% of the total SG isotopic inventory was in the tube region and that this activity corresponds to the dose rates measured in the vicinity of the tube region. Of the 3 regions of the generator (steam dome, tube region, channel head), it was assumed that most of the activity was in the tube region because the channel head is much smaller than the tube region and the steam dome, by design, is expected to have negligible levels of activity. The isotopic inventory inside the tube region was calculated based on this dose rate and the known isotopic distribution and physical dimensions. The inventory for the entire SG was then estimated by dividing the tube region activity by 90%.

2. It was assumed that 10% of the SG activity was released due to the impact of the drop and that 1% of this release amount was in the form of particulates small enough to become airborne. Hence, the fraction of the total SG activity that gets released to the environment is 0.001. The use of the 0.1% of the isotopes for dose assessments has been used historically on other steam generator replacements (SGRs). The early SGRs were not performed under the requirements of 10 CFR 50.59. Instead, a repair report was prepared and submitted to the NRC for review and concurrence. As part of their review, the NRC performed confirmatory analyses that used this percentage of isotopes being released. Recent SGRs performed under 10 CFR 50.59 have continued to use this isotopic release percentage.

3. *All activity releases were assumed to occur within the first 2 hours of the accident. This is conservative as it minimizes the dispersion of activity released to the environment, thereby maximizing the doses*

4. *The inhalation dose to the control room operator was calculated using the atmospheric dispersion factor for the control room, but without taking credit for the control room structure.*

5. *LPZ doses due to ground deposition are conservatively calculated assuming no evacuation or remediation during the 30-day exposure period.*

6. *The control room is closer to the containment than to any point on the haul route. It was therefore assumed that the control room dose from a SG rupture at the containment bounds the dose from a rupture at any point along the haul route or at the OSGSF.*

METHODOLOGY

The dose analysis was performed using the following steps.

1. *Using the worst case measured isotopic distribution [Table 1] for the activity inside the SG, a characteristic energy spectrum was calculated in the units of MeV/sec by energy group.*

2. *The energy spectrum [Step 1] was used in a point-kernel computer program to calculate a dose rate 10 ft from the outside of the SG.*

   As the dose measurement was taken with the primary side of the generators filled with water and the secondary side drained, the internal medium of the tube region was modeled as a homogenized mixture of steel and water. The homogeneous density of the tube region was calculated by dividing the total mass of the steel and water by the volume of the region.

3 *By ratioing the calculated dose rate [Step 2] to the measured dose rate 10 ft from the outside of the SG, a source adjustment factor was determined. Dividing this ratio by 90% [Assumption 1] yielded an adjustment factor of 9.0. The assumed initial isotopic inventory [Input 2] was multiplied by 9.0 to obtain the estimated total activity inside the generator.*

4. *The isotopic distribution [Step 1] was multiplied by the source adjustment factor [Step 3] to obtain the isotopic activities inside the SG corresponding to the measured dose rate of 85 mR/hr at 10 ft from the outside of the SG.*

5. *It was assumed that a certain fraction [Assumption 2] of the isotopic activity in the SG [Step 4] is released to the environment as a result of the rupture. For a given isotope and organ, the inhalation, submersion, and deposition doses were calculated based on the guidance in NUREG/CR-5164. Doses were calculated for the containment to EAB pathway. The doses for the other pathways were obtained by applying X/Q ratios to the total EAB doses The doses from all isotopes and pathways were summed to arrive at the total dose.*

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The total activity in the SG was calculated to be 777 Ci, with approximately 8% being from Fe-55, 6% from Co-58, 18% from Co-60, 38% from Ni-63, 12% from Cs-134, and 15% from Cs-137. The rupture of each SG was postulated to release 0.777 Ci to the atmosphere.

Table 1 - Isotopic Survey Data from CVCS Resin Tank

Isotope	ìCi/g	Isotope	ìCi/g	Isotope	ìCi/g	Isotope	ìCi/g
H-3	3.99E-03	Ni-59	2.79E-01	Sn-113	2.90E-02	Pu-239	2.30E-04
C-14	6.72E-01	Ni-63	3.26E+01	Sb-125	4.94E-01	Pu-240	2.30E-04
Mn-54	1.11E+00	Zn-65	3.65E-02	I-129	5.60E-05	Pu-241	3 13E-02
Fe-55	6.71E+00	Sr-89	7.49E-03	Cs-134	1.01E+01	Am-241	1.00E-04
Co-57	1.94E-01	Sr-90	1.07E-01	Cs-137	1.30E+01	Cm-242	2.59E-04
Co-58	4.98E+00	Tc-99	1.08E-04	Ce-144	1.25E-02	Cm-243	2.66E-04
Co-60	1.59E+01	Ag-110m	7.87E-02	Pu-238	6.51E-04	Cm-244	2.66E-04
						TOTAL	8.63E+01

Table 2 - Summary of Doses from OSG Rupture

Event	Release Point	Dose Point	Dose (Rem)			
			Whole Body	Lung	Bone	Skin
Drop from Crane	Containment	EAB	2.94E-02	1.28E-01	9.55E-02	1.83E-04
		LPZ	4.63E-03	1.62E-02	1.27E-02	1.30E-03
		CR	3.76E-02	1.31E-01	1.03E-01	1.06E-02
Drop During Transport	Haul Route	EAB	4.86E-02	2.11E-01	1.58E-01	3.02E-04
		LPZ	1.07E-03	3.73E-03	2.93E-03	3.00E-04
		CR	3.76E-02	1.31E-01	1.03E-01	1.06E-02

SUMMARY OF X/Q CALCULATION

The calculation performed to estimate worst-case atmospheric dispersion factors (X/Qs) at the EAB and at the LPZ for a hypothetical steam generator drop accident occurring at any point along the steam generator haul route [Figure 1] during the SGR is summarized below.

INPUT

1. *The representative meteorological data used were as reported in UFSAR Section 2.3 and Tables 2.3.2-23 through 29, "Joint Percentage Frequencies of Wind Speed by Wind Direction" for the 10-meter level (1/1/72-12/31/75).*

2. *Distance/location information for the EAB and LPZ were as reported in UFSAR Table 2.3.4-1 and Section 2 3.4.2, respectively. Centered on the Unit 1 containment vent, a radius of 4828m was used for the outer boundary of the LPZ.*

3. The containment building cross-sectional area (1800 m^2) used in estimating the atmospheric dispersion factors were as reported in UFSAR Section 2.3.5.2.

4. A haul route drawing [Figure 1] shows the locations of the steam generator haul route and the old steam generator storage facility (OSGSF).

5. The elevation at the top of the containment building is 856 04 ft with a grade elevation of 705 ft. Therefore, the physical height above ground level of the containment building is 151 ft (46 m).

ASSUMPTIONS

1. It was assumed that a postulated steam generator drop could occur at any point along the identified haul route from the Unit 1 containment to the OSGSF.

2. To account for the reduction in vertical cross-sectional area due to the sloping roof of the containment building, the top of the containment was assumed to be about 45 m for modeling purposes.

METHODOLOGY

1. The primary methodology used was based on information contained in the following guidance documents:

   a. NRC Regulatory Guide 1.145 - "Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants", Rev. 1.

   b. NUREG/CR-2858, "PAVAN: An Atmospheric Dispersion Program for Evaluating Design Basis Accidental Releases of Radioactive Materials from Nuclear Power Stations".

2. Since for a non-buoyant ground-level release, ground-level pollutant concentrations decrease with increasing downwind distance, the shortest distance from the haul route to the EAB and LPZ for points corresponding to the 16 wind direction sectors was determined. Based on guidance provided in Section 1.2 of Regulatory Guide 1.145, the assumed release points for each of the 16 directions were determined from Figure 1 as the minimum distance between any point on the haul route and the EAB for each direction. The shortest distances between each direction-specific release point on the haul route and the EAB are presented in Table 1. The LPZ is the area within a 4828 m (15840 ft) radius measured from the Unit 1 shield building vent. The shortest distance from the haul route to the LPZ was found to be 4196 meters measured from the OSGSF (located near the north end of the haul road) in the north direction. Since shorter distances are generally associated with less dispersion, this minimum distance was conservatively used in all directions to calculate the X/Q values at the LPZ (see Table 1).

3. UFSAR Section 2.3.2.4 indicates that terrain variations in the site region are minimal. Therefore, site-specific terrain adjustment factors (TAFs) were not used in the model.

4. The PAVAN model was configured to calculate X/Q values assuming both wake-credit allowed and wake-credit not allowed. The closest EAB is located 666 feet (203 m) from the haul road in the N and NNW directions [Table 1]. The containment buildings are 151

feet (46 m) above grade. The maximum wake-influence distance between a wake-producing structure and a release point was assumed to be 10 "building heights" downwind of the structure. This distance was based on guidance contained in Regulatory Guide 1.23, Proposed Rev. 1, for the siting of meteorological instruments away from wake-producing objects/structures. The shortest distances from the haul road to the EAB are less than 10 building heights away in the NW, NNW, N, NNE, NE, and ENE directions. Receptors at these sectors are therefore located within the building wake influence zone induced by the containment building. Thus, the PAVAN "wake-credit allowed" scenario results were used for the X/Q analysis at these sectors. However, the entire LPZ, which at its shortest distance from the haul route is 13765 feet (4196 m), is located beyond this wake influence zone. Thus, the PAVAN "wake-credit not allowed" scenario results were used for the X/Q analysis at the LPZ.

5. As described in Section 1.4 of Regulatory Guide 1.145, the 0-2 hour and annual average 5% site limit X/Q values were used to determine the X/Q values for the intermediate time periods by the logarithmic interpolation approach described in the PAVAN computer code.

6. Based on Regulatory Guide 1.145, the 0.5% sector X/Q or the 5% overall site X/Q, whichever was higher, was selected. Summarized below (Table 2) are the maximum X/Q values for the EAB and LPZ.

Table 1 - Shortest Distances from the Haul Route to the EAB and LPZ

Sector	Distance from Unit 1 Shield Bldg Vent to EAB		Shortest Distance from Haul Road to EAB		Shortest Distance from Haul Road to LPZ	
	feet	m	feet	m	feet	m
N	3100	945	666	203.0	13765	4195.6
NNE	2402	732	800	243.8	13765	4195 6
NE	2300	701	1200	365.8	13765	4195.6
ENE	1824	556	1450	442.0	13765	4195.6
E	1850	564	1760	536 4	13765	4195.6
ESE	2001	610	1900	579.1	13765	4195.6
SE	2100	640	2065	629.4	13765	4195.6
SSE	2300	701	2167	660.5	13765	4195.6
S	2851	869	2500	762.0	13765	4195.6
SSW	3225	983	2967	904.3	13765	4195.6
SW	4199	1280	3556	1083.9	13765	4195.6
WSW	2999	914	2560	780.3	13765	4195.6
W	2201	671	1940	591.3	13765	4195.6
WNW	2149	655	1690	515.1	13765	4195.6
NW	2175	663	1045	318.5	13765	4195.6
NNW	2402	732	666	203.0	13765	4195.6

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Table 2 - EAB and LPZ X/Q Values

Time Period	Exclusion Area Boundary		Low Population Zone	
	Sector / Distance (m)	Max. X/Q Value (sec/m^3)	Sector / Distance (m)	Max. X/Q Value (sec/m^3)
0 to 2 Hours	N / 203 m	2.71E-03	N/A	N/A
0 to 8 Hours	N / 203 m	1.84E-03	SSW / 4196 m	4.51E-05
8 to 24 Hours	N / 203 m	1.52E-03	SSW / 4196 m	3 39E-05
1 to 4 Days	N / 203 m	1.00E-03	SSW / 4196 m	1.82E-05
4 to 30 Days	N / 203 m	5.50E-04	SSW / 4196 m	7.42E-06

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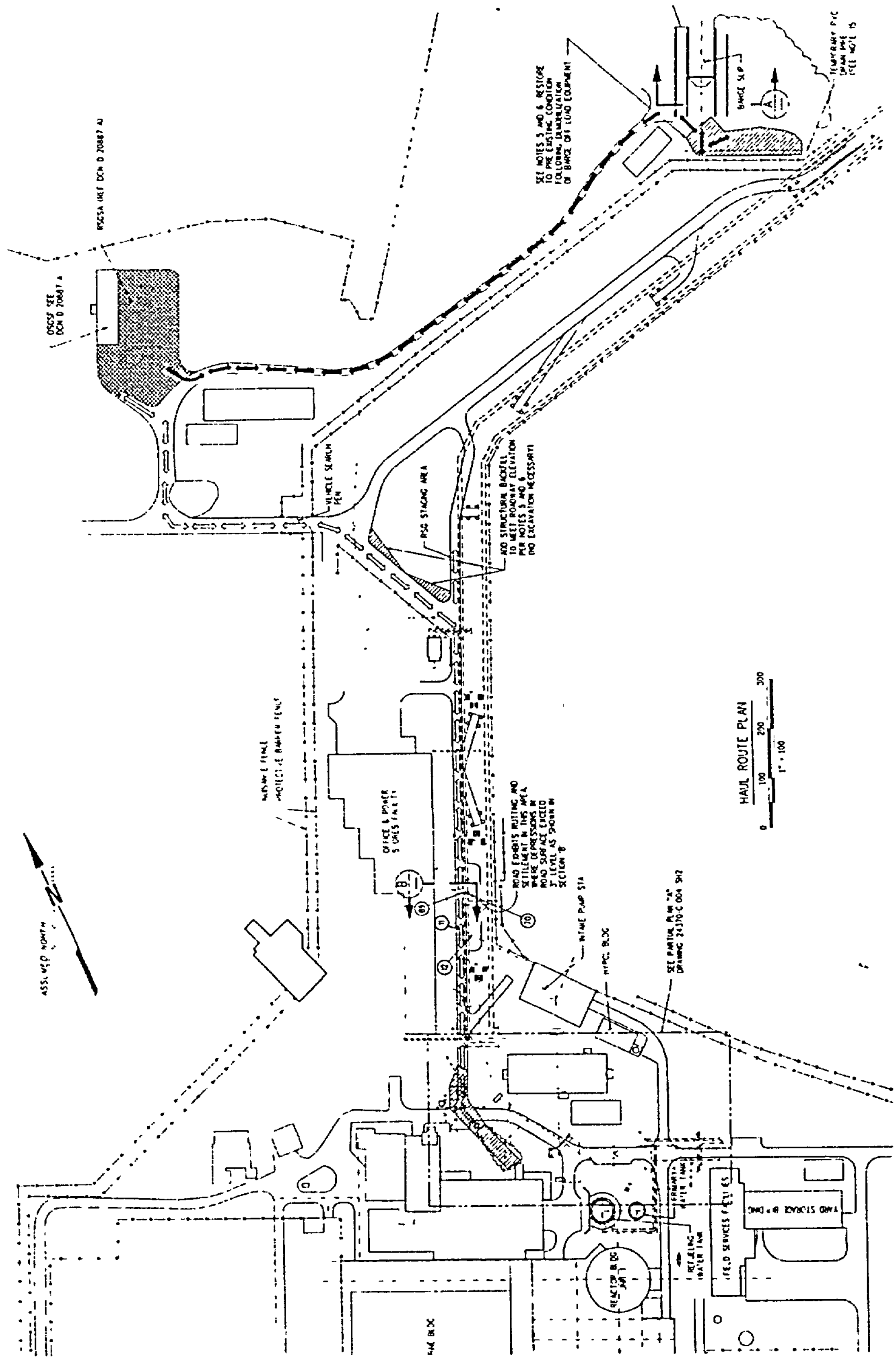
Figure 1 – Steam Generator Haul Route



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9. Describe the attributes of the heavy lift plan for the various loads to be lifted. Specifically identify who is responsible for the development and approval of the lift plan, and are persons responsible for plan development registered professional engineers having specialized knowledge of critical lift operations? Demonstrate that the plan, in part, is based upon the following: (1) the rated capacity and operational limitations specified by the crane's load chart, (2) measured, as opposed to calculated, weights for the materials to be hoisted; (3) thorough studies of wind speed and its effect on crane and hoisted load, and (4) consideration of the effects of ground conditions and all dynamic forces on the crane's stability.

   The heavy lift plan is detailed in engineering packages, which define the requirements for the safe rigging of the heavy loads associated with the steam generator replacement project. These engineering packages were developed by registered professional engineers having special knowledge of critical lift operations with many years of experience performing this type of work. The engineering packages associated with the rigging plan include the following details:

   - *Calculations to determine the critical aspects (e.g. lifted weight, center of gravity, size, etc) of items to be rigged*
   - *Qualification of rigging components*
   - *Qualification of rigging equipment*
   - *Comparison of lifted load with crane capacity*
   - *Allowable load paths and allowable locations of cranes with respect to the load paths*
   - *Crane foundation design and construction details*
   - *Relocation details for underground utilities in the OLS foundation area*
   - *Qualification and design of SG rigging attachment points*
   - *Load test requirements*
   - *Evaluation of safety-related buried commodities in the vicinity of heavy lift load path for a postulated load drop from the OLS*
   - *Evaluation of the OLS for seismic and wind/tornado loads*
   - *Evaluation of the response from a postulated drop onto the shield building*
   - *Load path restrictions (path, height)*
   - *Operating weather restrictions (detailed in the response to Question 15) and associated work instructions*
   - *Protection details for safety-related SSCs if a load drop occurs*
   - *Contingency measures for the realignment of plant systems if a load drop occurs*

   The list of components provided in the response to Question 32 indicates which component weights were measured and which weights were calculated.

   Incorporated into these engineered products are the load limitations derived from the crane manufacturer's load charts. For lifting of the SGs, calculated weights will be confirmed by load cell measurements upon initial lift operations The potential for winds to influence the safety of the lift operations will be controlled administratively using the monitoring described in the response to Question 10. As noted above, cranes performing heavy lift operations will be limited to locations where ground conditions have been examined and evaluated; more specifically, the OLS will be supported on an engineered pile foundation.

   A specific work plan and inspection record (WPIR) is written for each work activity associated with the above rigging plan that invokes the necessary requirements dictated by the *engineering packages. These WPIRs are prepared by construction personnel who plan the overall steam generator replacement construction/maintenance program. Each WPIR is reviewed by design engineering personnel to confirm that the planned activities are within the limits established by the engineering evaluations and analyses.*

10. Will cranes (outside lift system (OLS) and mobile cranes used to erect the OLS) and work areas be equipped with strategically located instruments to monitor wind velocity (speed and direction) at or near the elevation of hoisted loads? If not, provide a justification for not making the necessary provisions to vigorously measure wind velocity. If monitoring will be done, describe how and provide the basis for the monitoring scheme chosen.

    The OLS is provided with sophisticated electronic and computer based controls and instrumentation. It has two anemometers for measuring wind speed, one in the boom tip and a duplicate at the top of the back stay. The anemometers are verified to be operational prior to the boom/back stay being erected.

    The mobile cranes are generally not equipped with wind speed monitoring capabilities To assure that any restrictions on the wind speeds are implemented, the mobile cranes will rely on the site wind speed readings that are recorded at the site meteorological tower. In general, the mobile cranes have restrictions on their operational wind speed, as well as other operational limitations To assure that the crane manufacturer's operational limitations are followed, there are job specific construction procedures in place for the work associated with assembly/disassembly of the OLS. These controls regulate the construction activities. In addition, there is a work and inspection record (WPIR) specifically written for each work activity that invokes the requirements dictated by engineering, including wind speed limitations on crane operation. Meteorological forecasts will also be used to monitor wind speeds.

11. What actions will be taken to ensure the crane is equipped with correctly calibrated instruments to accurately monitor all parameters affecting safe crane operation?

    The instrumentation on the OLS (PTC crane) was last calibrated in September 2002. Instrument calibration is normally performed once a year, or as required by clients. Calibration of the OLS load cell instrumentation will be performed as required in ASME NQA-1 Subpart 2.15. As indicated in the response to Question 12.(1), the OLS will be load tested prior to use. Since this load test will be performed with a test load of known weight, it will confirm the calibration of the OLS load cells. Additionally, the OLS boom radius indication readouts will be verified during the load test, which will also verify the incline meter readings. The safe load indicator, which stops crane operation unless the operation improves the safety margin, will also be tested during the load test.

    Prior to the erection of the boom/jib, the anti-two block switches, minimum and maximum radius switches, airplane warning lights, and boom stops will be checked..

12. Section 5.1 of the topical report states that the rated load for the proposed crane configuration for the Sequoyah steam generator replacement (SGR) ranges from 440.8 tons (400 metric tons) to 517.9 tons (470 metric tons), depending on the lift radius. The OLS does not completely conform to the requirements of ANSI B30.5, "Crawler, Locomotive, and Truck Cranes," and the load test requirements of B30.5 in Section 5-2.2.2 do not subject the OLS to complete functional testing with and without the load following erection. Provide a response to the following:

(1) Will a load test of the OLS at 110-percent of the largest postulated load to be carried by the OLS be performed and what is that load and how is it determined? Will full performance tests with 100-percent of the largest postulated lifted load for all speeds and motions for which the system is designed be performed?

The OLS has been tested by the manufacturer to 125% of rated capacity. After erection on site a functional test of the OLS will be performed over the area of motion required for the lifts to be conducted during the steam generator replacement outage After the functional test is performed a load test will be conducted using a 550 kip test load (Note: this load is less than the largest load lifted by the OLS during the SGR). This load will be lifted and then the boom extended so that the crane is at 110 percent of rated capacity (ASME B30.5 5-2.2.2(a)(1)). This 550 kip test load will be limited to 2 feet above grade so that any underground safety-related SSCs will not be detrimentally affected if a load drop occurs. This 550 kips test load is the minimum load that is required to anchor the OLS during inclement weather.

(2) How will verification be performed during and following erection of the OLS, the proper assembly of electrical and structural components?

The OLS will be assembled by the manufacturer in accordance with the erection manual. The crane will be assembled and configured per instructions and drawings detailed in Section 4 of the OLS Users Manual by operators provided by the owner/designer who are well trained with full knowledge and understanding of the crane/manual and experienced in assembling and operating the PTC Crane. The OLS structure (e.g. bearings, gearboxes, bolts, shafts, wires, structural members, welds, etc.) will be inspected for wear and damage in accordance with the criteria set forth in Section 7 of the user manual. If any wear or damage is found, the appropriate corrective action will be taken. Following the erection of the OLS, functional tests will be performed that will verify proper assembly of the electrical and mechanical components Once functional tests are complete and acceptable, the OLS will undergo load testing as described in the response to Question 12.(1).

(3) How will TVA verify the integrity of all control, operating, and safety systems of the OLS following erection?

OLS monitoring instrumentation has been calibrated by Lloyds Register Rotterdam, The Netherlands, certificate NR 9855917, dated 01-03-1999 when the OLS was manufactured. Functional tests over the intended range of use and a load test will be performed on the OLS after it is erected to assure that the control, operating, and safety systems of the OLS are functioning properly. The load measuring devices on the OLS will be verified during the 110% load test that is performed after the OLS is erected on site.

(4) How will TVA demonstrate the ability of the OLS to protect against an overload situation to include the ability of the OLS to withstand a load hang-up.

The load measuring devices on the OLS will provide load indication to the OLS operator. A redundant Load Moment Safety System progressively warns and then disables crane operations. Once a system is disabled, only OLS operations that will improve safety margins will be allowed by the system.

13. Will lifting devices that are not specially design meet the guidelines of NUREG-0612, Section 5.1.1(5), as set forth in ANSI B30.9,"Safety Standard for Cranes, Derricks, Hoists, Hooks, Jacks, and Slings?" In addition, do the interfacing lift points on the old/new steam generators such as the lifting lugs meet the guidelines of NUREG-0612, Section 5.1.6(3)(a) or (b)?

The lifting devices that are not specially designed (i.e., commercially available rigging components such as wire rope slings) are required by the specification to which the rigging is designed and furnished to comply with ANSI B30.9. The design load by which the sling is selected includes the static load plus all dynamic loads (e.g., impact and wind), and these loads are documented in calculations prepared by the rigging contractor and reviewed by Bechtel. The guidance of Section 5.1.6 of NUREG-0612, which addresses Single-Failure-Proof Handling Systems, has not been applied to the interfacing lift points for loads handled by the OLS. While the OLS incorporates many redundant and safety-enhancement features, it is not considered a single-failure-proof lifting system. Consequently, the other heavy load handling and plant safety provisions, including compensatory measures, have been made a part of the load handling plan

14. Provide a description of how the OLS is anchored to the platform and describe the critical locations in the load carrying parts of the OLS for the various boom configurations. During a design basis earthquake with or without the largest postulated lifted load to include pendulum and swinging loads, demonstrate that the OLS will remain anchored to the platform and that the platform and OLS will be prevented from overturning.

The OLS will be supported on top of an 8 ft wide, 78.5 ft diameter concrete ring foundation that is supported by approximately 71 piles to bedrock and has an integral concrete cap that is a minimum of 4 ft thick. The crane base is supported on 24 independent jack stands, which are resting on top of the pile cap. Each jack stand is approximately 5 ft x 7.5 ft. Lateral loads are resisted by friction between the stands and the concrete. The OLS was evaluated in Reference 21 of Topical Report 24370-TR-C002 for stability and stress under the minimum design basis earthquake event for the proposed SGR lift configurations in both the loaded and not-loaded conditions. Due to the very low natural frequency of the pendulum (~0.1 hz) with a SG as the lifted load, the lateral displacement response of the SG center-of-gravity relative to the boom tip is less than 0.25 ft. The corresponding lateral load applied to the boom tip is approximately 2 kips, which is negligible for crane stability and stress calculations. Therefore, lateral loading of the boom tip due to "swinging" was neglected in the stability and stress calculations. Calculations have determined that the minimum factor-of-safety against overturning during a seismic event is 1.13 The factor-of-safety against sliding during a seismic event is 1.55 The factor-of-safety against torsional sliding is 1.91.

For the stress analysis of critical crane components it was conservatively assumed that all the members and connections have an interaction of 1.0 for combined stresses at their maximum working allowable for dead + lifted loads (D+L). The interaction value of 1.0 under D+L load condition is a baseline number for quick evaluation under D+L+E load condition, where E is the seismic SSE load condition and acceptance was yield stress for seismic II/I qualification.

15. What are the minimum wind conditions for operation of the OLS, how was the minimum wind condition determined, and what is its basis? If these conditions are encountered during heavy load lifts what actions will be taken to secure the load and place it in a safe condition? How long will it take considering side loads effects could cause the OLS to tip over?

The maximum wind speed allowed during operation of the OLS (PTC Crane) when the lifted load is more than 3 ft off the ground is 10 m/s (22 mph) in any direction measured at the boom tip. This operating wind speed is specified in the PTC Crane Manual and the Load Capacity Charts. The wind load due to this maximum wind speed has been accounted for by the manufacturer in the crane structural and stability calculations based on which the safe working load specified in the Load Capacity Table was arrived at with safety margins specified in the lifting codes The maximum wind speed allowed during operation of the OLS when the lifted load is less than or equal to 3 ft off the ground is 15 m/s (33 mph) in any direction measured at the boom tip. The Lifting Capacity of the OLS was determined by the manufacturer in accordance with the following codes (lifting codes): DIN 15018 Parts 1 & 3, DIN 15019 Part 2, DIN 15020 Part 1 and DIN 1055 Part 4, ASME B30.5-1994, SAE J987, SAE J765 and CE. It is noted that the OLS comes instrumented with a wind speed anemometer mounted at the boom tip.

The rigging contractor's calculation provides a comparison between the actual calculated wind force on the steam generator, using a wind speed of 50 mph, and the allowable lateral load on the steam generator, per the OLS manufacturers requirements. This comparison indicates that even with a wind speed of 50 mph the wind force on the steam generator will be approximately 55% of the allowable wind force. Keeping in mind that the allowable wind speed will be limited to 22 mph (10 m/s) in the high lift position and 33 mph (15 m/s) in the lowered position one can see that sufficient margin remains to maintain the OLS in a safe condition.

In case the wind at the tip is expected to exceed the specified 10 (15) m/s (22 (33*) mph), the crane will be secured in the configurations below as specified in the PTC Crane Manual and the rigging contractor's calculation:*

Wind Speed at Tip	Mainmast Angle	Jib Offset Angle	Slew Drive	Load
10 (15*)-22 m/s (22 (33*)-49 mph)	All angles allowed $0^\circ - 85^\circ$	Minimum 10°	Braked	Lower Block (**) suspended
22-30 m/s (49-67 mph)	80°	10°	Braked	Lower Block (**) suspended
30-46 m/s (67-103 mph)	80°	10°	Braked + Park Brake	Lower Block (**) secured with 200 tonne (440 kip) pretension to 250 tonne (550 kip) ballast
>46 m/s (> 103 mph)	Boom lowered	Jib lowered	Free	Not applicable

(*) Only when lifted load is carried not more than 3 ft above grade.
(**) Lower Block is the terminology used by the crane manufacturer for the main hook block or load block.

The above table shows that the load may remain suspended from the lower block (main hook block) for wind speeds up to 67 mph with the slew drive braked and the mainmast and jib offset angles configured as specified in the table. The maximum time required to bring the OLS from the operating configuration to the specified configuration is less than 15 minutes.

For wind speeds anticipated in the 67-103 mph range, the OLS shall be configured with regard to mainmast and jib offset angles as specified in the above table and the following cases apply with regard to the load: (1) If the load on the hook is equal to or greater than 550 kips, the load will be partially lowered to the ground so as to maintain a pretension of 440 kips; (2) If the load on the hook is less than 550 kips or there is no load on the hook, the load will be lowered and removed from the hook and the lower block tied off to a 550 kip ballast with a 440 kip pretension. The maximum time required to bring the crane from the operating configuration to the specified configuration is less than 30 minutes.

When wind speeds could exceed 103 mph (this would be expected to occur only during tornadoes) the boom and the jib will be lowered. The time required to accomplish this is about 2 hours.

The OLS manufacturer has qualified the crane for wind effects, including side load effects, for wind speeds up to 103 mph, with the lower block secured to a 550 kip load and pretensioned to 440 kips. Thus, the OLS will not tip over from side load effects for wind speeds up to 103 mph. The design basis wind speed for Sequoyah Nuclear Plant is 95 mph (UFSAR Section 3.3.1.4). Wind speeds exceeding 103 mph can be expected only during a tornado. However, for the Sequoyah SGR Project, all heavy lift operations using the OLS will commence only after confirming, based on weather forecasts and reports, that no severe weather conditions are expected for the duration of the lift. In the event a tornado watch or warning is announced in accordance with Procedure AOP-N.02, crane operations shall cease and the boom and jib will be lowered and oriented in a N-E direction as indicated on Figure 5-2 of Topical Report 24370-TR-C-002.

16. The submittal indicates that the mobile (lattice boom and/or truck) cranes used in the assembly/disassembly of the OLS will have a current certification and will be load tested during production. However, the licensee did not indicate if the mobile cranes will be "proof tested" to ensure proper operation. Demonstrate the operability of the mobile cranes prior to assembly of the OLS by testing in accordance with B30.5. Will a 110-percent static load test be completed and will full performance tests with 100-percent of the largest postulated lifted load for all speeds and motions for which the system is designed be conducted prior to heavy lift operations?

    The term "proof tested" refers to the performance testing of the crane features. Section 5-2.2 of ASME B30.5 discusses operational tests and rated load tests for mobile cranes. Operational crane tests are performed at the time of production of the crane and the crane manufacturer maintains records of these tests During the assembly/disassembly of the OLS and prior to each shift usage of the mobile cranes, a 20-point checklist of the crane features will be conducted by that shift operating team and signed off.

    If a load sustaining part of a crane (other than the wire rope) is altered, replaced, or repaired, ANSI B30.5 requires that the crane be load tested using a maximum of 110% of the manufacturer's load rating. For wire rope replacement, a functional test is performed using normal operating loads. The mobile cranes used in the assembly/disassembly of the OLS will follow these ANSI requirements

17. The submittal states that restrictions on the use of these cranes (mobile cranes-lattice boom and/or truck) will be imposed to specify the weather conditions under which they may be operated and how and when to secure the mobile cranes in case of inclement weather; and the restrictions are designed to preclude adverse interactions with safety-related SSCs."

With respect to the use of the mobile cranes for assembly and disassembly of the OLS, provide a response to the following:

(1) Describe the restrictions for use of the mobile cranes during assembly/disassembly of the OLS.

Use of the mobile cranes for OLS assembly/disassembly will be governed by the following restrictions:

a Load handling with the mobile cranes is limited to an approved area around the OLS boom location shown on Figure 5-2 of Topical Report 24370-TR-C-002.
b. The load imposed on the ground by the crane is limited to the calculated allowable ground bearing pressure.
c One-foot thick timber mats will be placed as a precautionary measure on grade over safety-related utilities (ERCW pipes) on the load path, if any, and loads traveling over safety-related SSCs shall be carried as low to grade as possible.

Wind related restrictions on mobile crane operation are detailed in the response to 17.(2) below.

(2) What are the minimum wind conditions for operation of the mobile cranes? How was the minimum wind condition for operation determined and what is its basis (e.g., dead weight of the boom with maximum postulated lifted load)?

Load handling operations with the Manitowoc 4100 cranes used for assembly/disassembly of the OLS will cease and the cranes will be put in a safe configuration when winds exceed 35 mph. This wind speed is based on the crane manufacturer's operating manual If other mobile cranes are used during the assembly/disassembly of the OLS, load handling operations will cease when winds exceed the manufacturers maximum recommended wind speed for safe operation

Mobile crane operations will cease and the cranes will be put in a safe configuration if a tornado watch or warning has been announced in accordance with Procedure AOP-N.02, "Tornado Watch/Warning".

(3) Describe the safety-related systems, structures and components (SSCs) that could potentially be affected by a dropped load during assembly/disassembly of the OLS. What effects could a load drop, during assembly/disassembly, have on Unit 1/Unit 2 operations?

The SSCs in the vicinity of where the OLS will be assembled/disassembled are the essential raw cooling water (ERCW) system piping, refueling water storage tank (RWST), and fire protection piping. Refer to the response to 17.(6) below for a discussion of the protection being provided to preclude any adverse effects of a load drop.

(4) Describe how an operator, to include those responsible for operations, will be notified of the minimum wind conditions for operation What actions will be taken if it is determine that winds near or at the limiting conditions for operations have been reached? How long will it take to perform these actions?

The maximum operating wind speed will be relayed to the operator during the pre-job briefing and they are also included on the operating load path drawings. A field engineer that accompanies the person-in-charge (PIC) will be in constant contact with the control room and crane operators not involved with the lift. The control room operators and crane operators will pass on wind information to the field engineer as necessary. The field engineer will keep the PIC notified, as required, as weather conditions change. Typically, lifts will commence only after reasonable assurance is obtained with regard to favorable weather and wind conditions at least for the duration of the lift thereby precluding any limiting conditions. However, in the event that winds increase to near or at the limiting conditions, further actions will proceed to place the crane in a safe and optimum configuration in accordance with the drawings, crane operating manual and site procedures, which will implemented through the Work Plan and Inspection Record (WPIR) for the activity.

(5) Since the mobile cranes have the potential to interact with safety-related SSCs during assembly/disassembly describe the safe load paths for these cranes. What processes or procedures will be used to ensure mobile crane operator remain within the safe load paths?

The approved area around the OLS boom location shown on Figure 5-2 of Topical Report 24370-TR-C-002 is a safe load path as long as the cranes are operated in accordance with the restrictions listed in 17.(1) and 17.(2) above and the protection listed in 17.(6) below is in place.

(6) Describe whether or not the mobile cranes during assembly/disassembly, with its largest postulated load, will fail and potentially impact safety-related SSCs.

Protection (see Sections 7.5 and 8.3 of Topical Report 24370-TR-C-002) for safety-related SSCs has been designed and will be used With this protection in place, safety-related SSCs will not be affected by a load drop from or overturning of a mobile crane.

18. The submittal in Section 4 2(2) states that crane operations will be conducted by highly trained and qualified personnel. Also section 4.2(3) references sections 5.1 and 5.2 as providing the details of operator qualifications that conform to ANSI B30.5. With respect to operator qualifications provide a response to the following·

(1) Describe how qualification program satisfy the requirements in Section 5-3 of ANSIB30.5.

The qualification of the operators will include the requirements specified in ANSI B30.5. The operators will successfully pass a complete physical, which covers all aspects of the standard prior to obtaining approval for crane operation and site access. The testing will include a complete physical, a MMPI psychological test, and training and testing to site procedures.

All OLS operators are being supplied by the manufacturer and have many years experience operating this crane. In addition, the operators will be trained per applicable portions of TVA Procedure MMDP-2, "Safe Practices for Operation of Overhead Handling Equipment".

(2) ANSI B30.5 in Section 5-3 states that only designated operators shall operate the crane. However, designated operators are selected or assigned by the employer or the employer's representative as being qualified tc perform specific duties. If operators are not employed by SQN or are employed by SQN what requirements/criteria are used to designate operators as being qualified (e.g., physical faculties and fitness, deviations from physical qualifications, grounds for disqualifications, required safety instruction, written examination, and performance test, as well as specific crane written examination and experience requirements)?

See response to 18.(1) above.

19. NUREG-0612, Control of Heavy Loads at Nuclear Plants, provides guidelines in Section 5.1.1(7) for crane designs which rely on criteria within ANSI B30.2 and CMAA specification number 70. Section 2-1 of B30.2 provides criteria for construction and installation and CMAA 70 specifies design stresses, service classification, and structural design, mechanical design, electrical and electrical equipment. However, B30.5 provides no criteria for crane design. What are the critical load bearing parts, load controlling parts, and operational safety devices of the OLS and how do the operational safety devices work together to ensure safe load handling (i.e., interlocks, upper hoist limit switch, lower hoist limit switch, rotate limit switch, emergency stop switches, locking devices, overload indicators, radius indicator, and overspeed, pressure, and temperature devices with shutdown capability if any)?

The OLS is designed, built, and tested to criteria based on the following DIN standards:

- *DIN 15018 Part 1, Cranes: Steel structures, verification and analyses*
- *DIN 15018 Part 3, Cranes: Principles relating to steel structures; Design of cranes on vehicles*
- *DIN 15019 Part 2, Cranes: Stability for non-rail mounted mobile cranes; test loading and calculation*
- *DIN 15020 Part 1, Lifting Appliances: Principles relating to rope drives; calculation and construction*
- *DIN 1055 Part 4, Design loads for buildings; Imposed loads – wind loads on structures unsusceptible to vibration*

In addition, the following codes are also listed in the crane user manual. ASME B30.5-1994, SAE J987, SAE J765 and CE

The OLS was tested to 125% of its rated load after it was manufactured. ANSI inspectors, along with the DIN inspectors, witnessed this test and have certified the crane.

The OLS has dual engines, dual hydraulic systems and dual computers. It is capable of performing its intended function with one of each system out of operation. In the event that all power and hydraulic systems fail, the load can be safely lowered using a 12-volt car battery and the manual controls

Beyond the OLS's dual systems, an operational safety device called a redundant Load Moment Safety System is integrated into the computer system, which progressively warns and then disables operations, subsequently allowing only operations, which improve the safety margins.

20. The topical report provides no information on the haul route from the transport location identified on figure 5-2 and whether the potential to interact with safety-related SSCs exists along the haul route, and whether those SSCs could either withstand the impact of a dropped SG or will be protected to preclude them from damage. What is the distance between the lay down area and the old and new SG storage area and what is the method used to load test the haul route (civil/structural)? What are the safety-related components that are located along the haul route that could be impacted by a dropped SG? What safety functions/systems would be impacted? What measures are to be taken to preclude a SG drop along the haul route and preclude the identified components from being damaged if a SG drop occurred?

The steam generator (SG) haul route is shown on Figure 1 in the response to Question 8. The distance from the downending/upending (lay down) area to the old steam generator storage facility (OSGSF)/replacement steam generator storage area (RSGSA) is approximately 2,180 ft. A review of structures, systems, or components (SSCs) in the vicinity of this portion of the haul route determined that the only safety-related SSCs are the ERCW ductbanks, manhole (MH) groups 31 and 32, handhole (HH) group 52, and 36 inch diameter ERCW piping. The manhole and handhole groups are associated with the ERCW ductbanks. There are no safety-related utilities close enough to the portion of the haul route between the replacement steam generator (RSG) barge offload area and the RSGSA to be affected by a load drop.

As noted in Section 6.3 of Topical Report 24370-TR-C-002, Section 9.2.2 of the UFSAR indicates that the ERCW system design function is to supply cooling water to various heat loads in both the primary and secondary portions of each unit. The ERCW ductbanks contain cables associated with ERCW trains A and B for both units The manholes/handholes were used for pulling the ERCW cables.

The potential for a load drop in the vicinity of the safety-related SSCs will be minimized by operating the transporter at less than 5 mph, provision of a stable road surface with limited grades, and use of a stable single-wide transporter. Additionally, the height of the transporter will be restricted in the vicinity of safety-related SSCs.

Although the SG transporter is considered rugged equipment, it is not specifically designed to withstand external events addressed by 10CFR50, Appendix A, GDC 2, which are part of the Sequoyah design basis. The probability of an external event occurring when the transporter is near a safety-related SSC, and which causes a heavy load drop that results in loss of the adjacent SSC is extremely low. However, to conservatively address the worse case consequences, a test weight or steam generator drop off the transporter was postulated to occur anywhere along the haul route in conjunction with a plant external event.

An evaluation of the impact of a load drop on the nearby safety-related SSCs determined that the ERCW ductbanks are adequate to withstand the impact without any protection. The 36 inch diameter ERCW piping is adequate provided that 2.5 ft of sand fill (or equivalent) is provided along the ERCW pumping station access road. The manhole and handhole groups are adequate provided that 2.5 ft of wood cribbing is placed along the perimeter on three sides of MH 31A1, MH 32A1, and HH 52A1. With this protection in place, there will be no impact on safety-related SSCs as a result of a load drop from a transporter.

In lieu of performing the haul route load test with a fully loaded transporter, the test will be performed by loading the test vehicle with enough test weights to produce a subgrade *bearing pressure equivalent to or greater than that caused by a loaded transporter. The purpose of the load test is to develop a test pressure that will identify any soft spots in the surface course/subgrade requiring repairs The entire haul route will be load tested prior to the SG transport. The entire haul route need not be tested all at one time; individual segments may be tested at different times. Load drop protection need only be present immediately prior to and during passage of the load (test load or SG).*

21 In accordance with recommendations provided in NUREG-0612, Section 5.1, discuss the potential for accidental dropping of the steam generator inside the reactor containment building. Discuss the potential consequences that could result from dropping the steam generator, any compensatory measures that could be implemented to minimize and mange the damage from the drop. Provide rationale for choosing a clearance of 20 feet (ft) above the dome for lifting the steam generators when its been analytically determined that at 12.75 ft or greater a dropped SG would perforate the dome and steel containment vessel.

Accidental dropping of a SG inside containment has been evaluated as part of the rigging engineering package and associated 10CFR50.59 evaluation. Lifting of heavy loads inside or above the Unit 1 containment with the OLS (PTC Crane) will not commence prior to completion of defueling. Since all fuel will be removed from the containment and the Spent Fuel Pit (SFP) will be isolated from containment, a load drop from the OLS inside or above the containment will not result in 1) releases of radioactive material due to damage to spent fuel, 2) damage to fuel or fuel storage racks, or 3) damage to the reactor vessel or spent fuel pool that causes a loss of water and the fuel to be uncovered.

Equipment required for safe shutdown may be affected by a load drop from the OLS inside or above the containment. Since Unit 1 is already shutdown and defueled, loss of this equipment would not affect the ability to shutdown Unit 1. However, some of the equipment that may be impacted is common with Unit 2. Common systems are the Essential Raw Cooling Water (ERCW) system, Component Cooling System (CCS), and Control Air System. To assure that a load drop from the OLS inside or above the containment will not affect the ability to shutdown Unit 2, the isolation valves outside containment for the ERCW system and CCS will be closed prior to lifting heavy loads with the OLS. The isolation valves for the Control Air System inside containment are located well away from any potential load drops and would not be affected. Since the isolation valves will not be affected, any break in control air lines due to a load drop results in loss of air and failure of the isolation valves in the safe (closed) configuration. Therefore, a load drop from the OLS inside or above the containment will not affect the ability to shutdown either unit.

If a SG drop is postulated to occur while the SG is above the containment shield building dome, it is assumed to fall vertically onto the dome directly below where it is suspended at the time. If the SG were to roll off of the dome it could potentially hit the Auxiliary or Control Buildings and impact the spent fuel pool and/or equipment required to safely shutdown Unit 2. As noted above, with the reactor defueled, outside containment isolation valves for ERCW and CCS closed, and the SFP isolated from the containment, a load drop inside containment will not impact fuel or prevent the safe shutdown of Unit 2. Given the consequences of a SG impacting the Auxiliary or Control Buildings, handling of the SGs must be done in a manner that assures that if a SG drop occurs above the containment dome, it penetrates the dome rather than rolling off of it.

Since it is difficult to predict where the SG will go following an impact from an arbitrary height onto the dome, an analysis was performed to determine the minimum height above the dome *a SG would need to be suspended to guarantee that it would penetrate the Shield Building. This minimum distance between the SG and the Shield Building dome will be maintained by lifting the SGs vertically through the Containment openings until the defined minimum clearance is attained. The SGs will then be translated horizontally to the outer edge of the Containment as shown on Figure 5-2 of Topical Report 24370-TR-C-002.*

As detailed in Section 7.1 of Topical Report 24370-TR-C-002, a SG drop from a height of 12.75 ft or greater will perforate the concrete Shield Building dome and Steel Containment Vessel (SCV). A drop from this height ensures complete penetration of the SG through the dome and into the Containment Building, as opposed to a response characterized by impact with and deflection off the Containment dome. To be conservative, a minimum clearance from the Shield Building dome of 20 ft will be used when lifting the SGs. This 20 ft clearance is within the lifting limit of the OLS.

22. Explain what is meant by discharge piping (e.g., is it the discharge to the ultimate heat sink or is it the flow of cooling water to safety and non-safety related loads)? If discharge is to the safety and non-safety related loads describe the effects of an ERCW Train A discharge piping failure for both units on plant operations from a heavy load drop from the maximum postulated lifted load. What safety related SSCs will be affected and what compensatory measures will be implemented to minimize and manage the damage from the drop?

Refer to the response to Question 28 for the definition of "discharge piping." The compensatory measure will require that the spent fuel pool cooling be aligned to the Unit 1 Train A Component Cooling system as a prerequisite Therefore, there are no actions to be taken after a postulated load drop to protect the Spent Fuel Pool Cooling function. The ERCW supply to the 1A Component Cooling Heat Exchangers is from the Unit 2 A-train supply header, and the return is to the B discharge header. Therefore, the postulated flow blockage of the A discharge header will have no effect on the SFP cooling. In the event of a pipe rupture on the 1A ERCW supply header, the compensatory measure will isolate the break and full ERCW flow will be returned to the 2A ERCW supply header. The short interruption in flow will result in a negligible increase in spent fuel pool temperatures.

23 The topical report in Section 8.3 for the Unit 2 ERCW supply piping determined the peak particle velocity from a drop load using reference 14. However, reference 14 indicated that criteria for underground utilities are not available, which includes pipelines. Moreover, reference 14 indicated that criteria should be based on available controlled tests and not on evaluations. The load used in reference 14 was a two-ton ball dropped from 40 feet which is a few orders of magnitude lower than the largest postulated load that can be potentially dropped at SQN (400-500 tons). What assumptions were made, such as soil type, soil compaction, depth of piping, vulnerability of supply piping during the lift (length of time during lift that makes this situation plausible), difference in loads evaluated in reference 14, and height of lifted load above surface? How were uncertainties accounted for in the calculation considering that the reference provides no criteria to evaluate peak particle velocities in soil for underground utilities? What was the calculated peak particle velocity and pressure 63 feet away from the drop and what design pressure is the piping designed to withstand? Does the compacted soil around the piping act as a missile shield to protect the ERCW supply line piping and if so how was this factored into the evaluation?

The references pointed to in this response are listed at the end of the response.

The pipes being evaluated are the Unit 2 30 inch diameter ERCW pipes (0.375 inches wall thickness, material carbon steel conforming to ASTM A155, KC 60, Class 2 with Fy = 32 ksi) running in the N-S direction on the east side of the Unit 1 Shield Building. The subject pipe is a flexible pipe. The postulated drop is the drop of the SG while traversing the load path segment above the dome at or near the parapet along the peripheral circumference resulting in a first impact near the dome periphery or parapet and then falling over to the ground. The nearest Unit 2 ERCW pipe is located at a distance of ~131 ft in plan from the load path at the point it crosses the Shield Building Dome parapet wall The impact location of the SG after a postulated drop was conservatively estimated as 60 ft from the subject pipes The soil cover above the pipes is ~2.5 ft. From the subsurface investigation report for the SGR project (Reference (c)), the soil layer near the postulated impact location and at the location of the pipes is stiff Clay Fill The soil properties used in this evaluation were primarily based on Reference (c).

The impact from a dropped SG causes waves (body and surface) to propagate in the soil media. These waves are transmitted outward from the impact location (energy source) and are attenuated with distance. Displacement waves move away from the source of a vibration at a constant velocity, called the propagation velocity, that depends in magnitude on the properties of the media and upon the type of wave that is produced. The parameter that is commonly used to describe ground motion is particle velocity. Particle velocity is the velocity of displacement of an individual particle as a vibration wave passes through the particle location. Propagation velocity is simply the rate at which the vibrational disturbance or wavefront moves from the source. Propagation velocity depends on the characteristics of the transmitting medium (soil, rock, etc.), while particle velocity is a function of the amount of energy imparted to the soil at the source, of the distance between the particle and the source, and of any energy losses during transit.

The methodology used for evaluating the buried pipe was as follows:

A. <u>Determine the peak particle velocity of the soil at the location of the pipe</u>

The magnitude of the vibration at a distance due to propagation of shock waves from a source is a function of the energy at the source (effect of source energy) and the distance from the energy source (effect of transmitting media). It has been found by investigators (see References 9, 22, 31 in Reference (a)) that the peak particle velocity is the most useful measure of the vibration magnitude. Combining the effects of distance and energy, the attenuated peak particle velocity (ppv or V_s) of the shock waves at the location of the buried pipe that is quite a distance away from the impact location is determined using the general scaled-distance wave propagation equation of the following form, proposed by Wiss in Reference (a).

$$V_s = K\left(\frac{D}{\sqrt{E}}\right)^{-n}$$

where,

V_s= peak particle velocity, in inches per second;
D = distance, in feet, from the point of impact or energy source
E = impact energy in foot-pounds of the energy source
K = intercept, in inches per second, (value of vibration amplitude at $D/\sqrt{E} = 1$ (ft/lb)$^{1/2}$

n = slope or attenuation rate

The above equation can also be expressed in the following convenient form:

$$V_s = K\left(\frac{\sqrt{E}}{D}\right)^n$$

The values of the parameters K and n vary and are essentially dependent on the soil type through which the shock waves propagate. The value of n generally lies between 1.0 and 2.0 with a relatively common value of 1.5 (Reference (a)) Figure 5 of Reference (b) reports test data from field measurements of particle velocity versus √E/D for different soil types (clay, wet sand, dry sand and rubble). This chart was prepared by Wiss (Reference 2 in Reference (b)). From this chart it is noted that the lines for each soil type are linear when plotted to a log-log scale. The values for K and n are determined by fitting the data for clay soil in Figure 5 of Reference (b) into the scaled-distance wave propagation equation. The parameters K and n in the general scaled-distance wave propagation equation, which can be determined from the test data in Figure 5 of Reference (b) for different soil types, are basically a function of the soil type and therefore the data from this chart are applicable and conservative regardless of the magnitude of the impact energy. The values of K and n were determined to be 0.112 and 1.5, respectively. Using values of W = 733 kips (maximum lift weight of a replacement steam generator, including attached weights of insulation, trunnions, upper lateral supports and bumpers), H = 135 ft (drop height), and D = 60 ft (distance of impact location from the pipes), the computed value of V_s at the location of the pipes for the subject drop parameters was 19.92 ft/sec.

Further, although the empirical parameters K and n in the scaled-distance wave propagation equation of the form presented by Wiss were determined based on available test data of low intensity wave propagation of relatively minor tremors, its use for a relatively high energy impact of a nuclear steam generator drop is conservative due to higher damping as explained below The impact energy from a drop, E = (W x H), is conserved after impact as:

$$E = E_{propagated} + E_{dissipated}$$
$$= \tfrac{1}{2} m_s V_s^2 + E_{dissipated}$$

where,

m_s = the soil mass effective in ground motion (increases with distance from impact location)
V_s = the soil particle velocity (decreases with distance).

The propagated energy initiates soil motion In a low intensity impact, only a very small portion of the energy is dissipated and most of the energy is propagated. However, high intensity impacts, in comparison, have much higher damping since a more significant portion of the impact energy will be dissipated at the source point itself by physically displacing the soil in the near vicinity of the impact location where the missile penetrates into the soil and a relatively smaller portion gets propagated as stress waves. Alternatively, the dissipated energy can be regarded as irrecoverable energy due to the extensive localized plastic deformation in the vicinity of the impact. Therefore, the peak particle velocity predicted using the empirical scaled-distance wave propagation will be overestimated since it will correspond to a higher sqrt(E)/D than actually is, and hence the conservatism in the prediction.

B. <u>Determine the free field soil pressure at the pipe-soil interface</u>

Based on one dimensional wave propagation considerations, the ppv (V_s) computed above is then used to estimate the free field soil pressure (σ_F) on the buried pipe using the relationship between dynamic stress and particle velocity given by the equation (References (d and j):

$$\sigma_F = \rho_s C_{ps} V_s$$

where,

ρ_s = *the density of the soil in which the wave travels*
C_{ps} = *the propagation velocity of the shock waves (Rayleigh waves in this case) through the media soil; Note that the product $\rho_s C_{ps}$ is referred to as soil impedance.*
V_s = *the peak particle velocity at the point of interest in the media (e.g. location of the buried pipe)*

The above evaluation assumes the energy is transmitted in a homogeneous, isotropic, elastic half-space. Although soils are not ideally elastic, they behave in a reasonably elastic manner especially at distances away from the impact location. The impact from a dropped SG causes waves to propagate in the soil media. These waves are distributed as body waves and surface waves. The greatest portion (~67%) of the energy imparted to the soil is transmitted as Rayleigh or surface waves (R-waves) followed by shear waves (~26%) (References (g), (h) and (j)). The ERCW pipes are located relatively near the surface. The shock waves that could load the pipe will be the waves propagating horizontally along the surface soil layer. For these reasons surface (Rayleigh) wave velocity will be used for C_{ps} in the above equation. The shear wave propagates at a velocity, C_s, given by (Reference (i), Chapter 3) as:

$$C_S = \sqrt{\frac{G}{\rho_S}}$$

where,

G = shear modulus
ρ_S = *mass density of the soil material*

The velocity of propagation is slightly slower than the shear wave velocity and is taken as $0.95C_s$ (Reference (i), Chapter 3). The estimated free field soil pressure associated with the traveling shock wave at the location of the pipe was 395 psi

C. <u>Evaluate the buried pipe based on the free field soil pressure on the pipe</u>

For flexible buried pipes, due to their flexibility, the primary performance limit or failure mode under shock wave loading is excessive diametric deformation (deflection) that could result in reversal of curvature of the wall from (Chapter 4 of Reference (e) and Reference (f)). Reversal of curvature is a deflection phenomenon and will not occur if deflection is controlled. Reference (e) provides guidance regarding the dynamic load factor (DLF) and deflection evaluation for shock wave loading of buried flexible piping. Reference (e) recommends use of a DLF value of 1.20 (on the free-field pressure magnitude) in conjunction with the free-field static pressure loading for the determination of maximum dynamic soil pressure (on the free-field pressure magnitude). The resulting pipe radial displacement in the direction of wave loading is determined using the modified Spangler equation for flexible pipes (Chapter 4 of Reference (e))

$$\Delta X = \frac{\gamma_B W_c r^3}{EI + 0.061 E_{sr} r^3} \frac{1}{0.913}$$

where,

E = modulus of elasticity of pipe wall
I = moment of inertia of pipe wall
r = radius of pipe
Esr = pipe-soil interaction modulus
$W_c = DLF \times \sigma_F \times (2r)$
γ_B = bedding factor

The internal pressure (design pressure of ERCW pipe is 160 psig) in the pipe was conservatively neglected. Per Chapter 4 of Reference (e) and Chapter 3 of Reference (f), a steel pipe will be in a state of impending failure by reversal of curvature at a deflection of about 20 percent of the pipe diameter. Since the postulated drop of the SG is an extreme event whose occurrence is highly improbable, a deflection of up to 10 percent will be considered acceptable under the resulting shock wave loading and allows a reasonable margin of safety against failure/collapse of the pipe. Thus, adequacy of the pipe against collapse is judged on the basis of a 10% maximum radial deflection criterion. This criterion is thought to be fairly conservative, especially considering that the internal pressure of the pipe (which will counteract the wave loading) is neglected. The deflection in the subject evaluation was determined to be 9%. The apparent circumferential stress in the pipe wall can be estimated as pd/(2t) = (1.2 x 395 psi) x 30 inches / (2 x 0 3125 inches) = 22.75 ksi against a minimum yield strength of 32 ksi.

<u>The evaluation includes the following conservatisms:</u>

(i) The internal pressure of water in the ERCW pipe (design pressure is 160 psig), which counteracts the effects of shock wave loading, was neglected.
(ii) Although the design thickness of the pipe wall is 0.375 inches, the pipe wall thickness was taken as 0 3125 inches for calculations.
(iii) The load path for the SGs when they are near the periphery of the dome is in a northerly direction. Due to the slope of the dome, the direction of the swing, and configuration of the channel end nozzles, the direction of fall is likely to be in a northerly direction in which case the distance of the impact location from the ERCW pipes will be well over 100 ft. However, it was postulated that the fall takes place in an easterly direction, thereby reducing the distance of the impact location to the Unit 2 ERCW pipes to 63 ft. Further, due to the weight and configuration of the SG (cg), a drop near the edge of the dome is likely for the SG to quickly find a path vertically downward in the immediate vicinity of the Shield Building wall in which case also the impact location will be well over a 100 ft from the pipes. The distance used in the computations was 60 ft. Hence, the distance of the impact location from the pipe used in the computations is quite conservative.

(iv) *The scaled energy equation as applied to the response of high energy impact is conservative (see discussion in Section A above)*

(v) *The time taken for the SG to traverse the load path above the dome near its periphery will be small (~ 5 minutes for each generator lift). Therefore, the time duration during which the postulated drop is plausible is very small.*

The following references were used in developing the above evaluation for the buried pipe:

(a) *Wiss, J.F., Construction Vibrations: State-of-the-Art, Journal of the Geotechnical Engineering Division, ASCE, Volume 107, No GT2, February 1981, pp 167-181. (same as Reference 14 in Topical Report 24370-TR-C-002)*

(b) *Lukas, Robert G , Densification of Loose Deposits by Pounding, Journal of the Geotechnical Engineering Division, ASCE, Volume 106, No. GT4, April 1980, pp 435-446 (same as Reference 15 in Topical Report 24370-TR-C-002)*

(c) *TVA Document SQ-RPT25.92, Revision 00, Replacement Steam Generator Project Foundation Soil Sample Analysis Report, Unit 1.*

(d) *Wong, F.S., and Weidlinger, P., Damping of Shallow –Buried Structures due to Soil-Structure Interaction, The Shock and Vibration Bulletin 52, Part 5 of 5, May 1982, pp149-154, Naval Research Laboratory, Washington D.C.*

(e) *Bulson, P.S., Buried Structures, Static and Dynamic Strength, Chapman and Hall, London, 1985*

(f) *Moser, A.P., Buried Pipe Design, McGraw Hill Inc., 1990.*

(g) *Heckman, W.S., and Haggerty, J.D., Vibrations Associated with Pile Driving, Journal of the Construction Division, ASCE, Volume 104, No. 004, December 1978*

(h) *Winterkorn, H.F., and Fang, H.Y. (Editors), Foundation Engineering Handbook, Van Nostrand Reinhold Company, 1975, Chapter 23.*

(i) *Wu, T.H., Soil Dynamics, Allyn and Bacon, Inc., Boston, 1971.*

(j) *Dowding, Charles H , Construction Vibrations, Prentice Hall, 1996.*

24. Section 8.3 of the topical indicated that the ERCW duct banks would be negatively impacted from an old steam generator/ replacement steam generator (OSG/RSG) drop. What safety-related equipment/functions would be impacted from a dropped OSG/RSG? What is the depth of the duct banks below the surface and what is the maximum pressure the duct banks can withstand without risk of failure? What were the assumptions in the analysis and what were the soil pressures 1 foot above, at the duct bank surface, and 1 to 3 feet below the duct banks as a result of dropping an OSG/RSG? What is the depth of soil to be added to account for a potential load drop? Specify what soil type, total area to be covered, and compaction requirements for the additional fill, and provide a drawing indicating the locations where fill will be added.

The ERCW ductbanks that are the subject of this question are: (1) Ductbank between manhole MH12 and handhole HH3 (called ductbank DB1); and (2) Ductbank between manhole MH12 and handhole HH29 (called ductbank DB2). See Figure 5-2 of Topical Report 24370-TR-C-002 for location of the ductbanks. They are both well over 200 ft in length. The critical postulated impact was from a flop-over fall after the SG has been dropped from the OLS, the load being carried at a height of 3 ft above grade along the load path at or near the ductbank locations. From TVA drawing 10N251, the grade above the ductbanks in the fall zone of a dropped generator varies from 704 ft to 707 ft. Based on the subsurface investigation performed for the steam generator replacement, the soil around the ductbanks *is clay fill. The soil properties used in the evaluation were based on the above subsurface investigation and from the UFSAR.*

The highest elevation of top of ductbank DB1 is 698.42 ft. The minimum grade elevation above this ductbank is 704.5 ft. The highest elevation of top of ductbank DB2 is 695.5 ft. The minimum grade elevation above this ductbank is 704.5 ft.

The methodology used in evaluating the ductbanks under the dynamic impact loading is as described below:

The impact energy from a flop-over fall of the SG about its base is estimated using principles of dynamics. The depth of penetration of the dropped steam generator (steam dome portion) into the soil and the resulting contact-pressure (pulse) time history were estimated considering the bearing resistance of the soil stratum overlaying the duct bank using Meyerhoff's bearing capacity equations. Suitable attenuation of the surface pressures were considered based on Boussinesq's equation thereby obtaining the spatial distribution of the impact loading on top of the ductbank. The depth of penetration into soil was estimated as 1.63 ft. The maximum attenuated pressure on top of the ductbanks from the impact loading due to a flop-over fall of the SG were estimated to be 107psi and 81psi for ductbanks DB1 and DB2, respectively.

The duct banks were then analyzed dynamically as beams on an elastic foundation subjected to the attenuated pressure time-history loading. A free-free boundary condition is considered at both ends of these ductbanks because the presence of 1/2 inch compressible expansion joint material at the ductbanks end connections with the attached pull-boxes/manholes.

The total response of the ductbank was calculated by performing modal superposition of the response of the first 25 modes of vibration. Response parameters such as deflection, shear and bending moment were thus obtained and the acceptability of the response was then assessed.

ERCW ductbanks DB1 and DB2 were shown to remain adequate to withstand impact loading due to flop-over effect after a postulated SG drop from the OLS provided the grade elevation above these ductbanks in the fall zone were at least 707 ft. The critical response parameter was the bending moment. The maximum bending moment under impact loading in ductbank DB1 was determined to be 535 k-ft against its ultimate capacity of 608 k-ft. The maximum bending moment under impact loading in ductbank DB2 was determined to be 368 k-ft against its ultimate capacity of 597 k-ft. It is noted that an evaluation of ductbank DB1 using the soil depth above it as the minimum existing grade elevation (~EL 704.5 ft) showed that the bending moment exceeded the capacity slightly. Therefore, it was decided to conservatively protect the ductbanks by raising the grade level above both ductbanks to EL 707 ft in the fall zone of the SG.

This will require areas above the ductbank, within the fall zone, with grade elevation below 707 ft to be raised to 707 ft using any earthfill placed in a standard way. The areas where fill may be required are shown as the cross-hatched area on Figure 5-2 of Topical Report 24370-TR-C-002. The depth of fill required will vary from 0 ft to 2.5 ft. Since this is a protection for impact loading and a reasonable moment margin being available, there are no specific compaction requirements, because the energy will be dissipated by displacing the soil even if it is not well compacted. If well compacted, the energy will be further attenuated *through the additional soil depth. Timber mats, having good energy absorbing properties, may also be used in lieu of earthfill.*

25 What impact will the closing of valves 1-26-575 and 1-26-653 have on the operability of the HPFP? What compensatory measures are going to be implemented during the periods of valve closure? For mobile cranes operating during assembly/disassembly of the OLS is there adequate depth of cover for fire protection piping to prevent mechanical injury?

The piping from valve 1-26-575 to valve 1-26-653 comprises one of the 4 feeders to the Auxiliary Building Fire Protection ring header. The Auxiliary Building ring header design requirements are that no more than one of the feeders be out of service. Normal plant processes will be used to document the isolation of the feeder. Therefore, there is no impact on operability of the HPFP from the isolation of this piping segment.

The only action required from the isolation of this piping segment to prevent the isolation of a second feeder to the Auxiliary Building ring header. Current plant processes will be used for these administrative controls.

As indicated in Appendix A to Topical Report 24370-TR-C-002, the fire protection piping inside the ERCW pipe tunnel will be isolated prior to commencement of load movements with the OLS. The purpose of this action is to minimize the potential contribution of water from fire protection piping on flooding of the ERCW tunnel due to failure of the fire protection piping inside the ERCW tunnel as a result of a load drop from the OLS. Isolation of this piping segment will also eliminate any possibility of depressurizing the HPFP system due to the postulated load drop, thus reducing the actions that must be performed following a load drop. As indicated above, isolation of this portion of the fire protection piping will not affect the operability of the fire protection system.

Underground fire protection piping in the yard areas where the mobile cranes are operating has been evaluated for the surcharge loads created by the mobile cranes. This piping is not adversely affected by these surcharge loads As indicated in the fire protection system design criteria document, sectionalizing valves are provided to isolate potential faults. A fault in the fire protection piping due to a mobile crane load drop is no different in its consequences than a fault created by other means Therefore, the consequences of a load drop from a mobile crane would be mitigated by closure of the appropriate valve(s).

26. Although safe load paths have been identified on figure 5-2 of the rigging and heavy load handling topical report the staff believes that it will be difficult for the operator to stay within the safe load path during the various lifts. Describe the communications plan, administrative controls, crane operator actions, and crane automatic actions used to control the lift within the safe load path identified in figure 5-2 of the topical report

The safe load paths can be followed by the OLS instrumentation. The instrumentation accurately indicates the radius of the load. The slewing of the OLS will be directed by the Mammoet/RI superintendents under the direction of the PIC (person-in-charge). For the initial pick, the boom will be located over the load using a Total Station surveying system. The load path is designated as being at a certain radius for which there is instrumentation in the cab that accurately locates the load. In addition to the instrumentation the load path will be marked on the ground. The rigging operation will be directed by the PIC who will be in constant radio communication with the crane operators and load tenders inside containment. A field engineer will be with the PIC who will be in constant communication with the operator *in the control room Figure 5-2 presented in Topical Report 24370-TR-C-002 is only a schematic designed to illustrate the load path in a general way. The actual implementation drawing precisely defines each segment of the load path.*

27 How much time will expire during the movement of an old steam generator along the load path (from the containment to the transporter) where interaction with safety-related SSCs could occur? How much time will expire during the movement of the replacement steam generators along the load path (from the transporter to containment) where interaction with safety-related SSCs could occur? What is the total time to move the OSGs and RSGs between the transporter and inside containment? What is the total time the SGs will be in a position to drop and cause damage to the safety-related SSCs (consider SSCs that may be impacted along the haul route from the transporter location to the storage facility)?

We anticipate that from the time an OSG starts exiting the containment dome until it is positioned to start downending on to the transporter will be approximately 1-1/2 hours. This time is also true for a RSG once it is upended and ready to start towards containment until it is inside the containment dome Safety-related SSCs (e.g., ERCW piping and ductbanks) could potentially be impacted by a SG drop for a time duration of approximately 1 hour during this portion of the lift. As described in Topical Report 24370-TR-C-002, protection and compensatory measures will be in place during this portion of the lift to prevent damage to and/or mitigate the consequences of damage to these SSCs.

The time to downend or upend a SG is anticipated to be approximately 2 hours. No safety-related SSCs could be impacted during this portion of the lift by a SG drop.

The anticipated time to haul an OSG once it is ready for transport until it is set in the OSGSF, including removal of the tiedowns, is approximately 3-1/2 hours. Although there are safety-related SSCs (ERCW piping and ERCW ductbanks and associated manholes and handholes) buried adjacent to the haul route, as detailed in the response to Question 20, protection will be provided prior to movement of the SGs along the haul route such that these SSCs will not be damaged as a result of a load drop from the transporter.

28. An OSG/RSG drop over Unit 1 ERCW would require realignment of the component cooling water system from Unit 2 to provide spent fuel pool cooling With Unit 1 defueled (full core off load to the spent fuel pool) how long will it take to reach the limiting temperature for the spent fuel pool? The licensee has committed to realigned the component cooling water system from Unit 1 to Unit 2 to provide spent fuel pool cooling in the event of a load drop What actions are necessary (automatic and manual) and how long will it take to complete the realignment?

In the Topical Report 24370-TR-C-002 submittal, the term "discharge piping" refers to the return to the ultimate heat sink However, note that the piping susceptible to damage from the heavy load drop includes supply piping from the pumps to the various heat loads, as well as the return to the ultimate heat sink. In TVA terminology, "supply header" refers to piping between the pumps and the various loads. "Return header" refers to piping that returns water from the various loads to the Ultimate Heat Sink. A load drop could either rupture the piping or crush the piping resulting in loss of flow. The specific piping affected is the 1A Supply Header, 1B Supply Header, and A Discharge Header. Any or all of these pipes may be affected simultaneously. The effects of the load drop on the system will vary depending on the specific set of problems.

The ERCW system is not strictly unitized. The Unit 1 A-Train (1A) and Unit 1 B-Train (1B) supply headers mostly supply Unit 1 loads. However, some loads are common to Unit 2. Among these loads are the Diesel Generators, the space coolers for the Component Cooling System pumps, the A-train Auxiliary Air Compressor, the Control Air Compressors, the Main Control Room chillers, and the Electrical Board Room chillers. Please refer to the 47W845 series TVA flow diagrams. For ease of understanding, a simplified flow drawing has been prepared indicating the locations of the potential damage. This drawing has been included as Figure 6-1 in Topical Report 24370-TR-C-002, and an electronic file is available upon request

TVA is making the assumption that the rupture of a supply header will result in complete de-pressurization of the affected train, and also that flow blockage from a crushed pipe is total. Therefore, if the 1A supply header is ruptured, then all of the Train A ERCW flow is lost, including the Unit 2 supply header. Likewise, if the A discharge header is crushed, then all Train A loads feeding into that header will have their flow stopped, including Unit 2 equipment.

Due to the rapid flooding that could occur in the worse case events, a wall is being erected at the entrance to the tunnel containing the potentially affected piping. This wall will eliminate flooding of the auxiliary building from concern.

The potentially affected SSCs from the worse case scenarios include all components that receive ERCW flow, Unit 1 and Unit 2. However, compensatory measures will be put into place that will ensure that safe shutdown capability exists on Unit 2. The compensatory measures will isolate a ruptured supply header to restore flow to the otherwise unaffected Unit 2 piping. Isolated components may selectively have alternate supplies placed in service. In the event of a crushed discharge header, the compensatory measure will crosstie the discharge headers such that the A-train discharge flow is routed through the B Discharge Header. A ruptured discharge header requires no action to isolate the leak. Crushed supply headers may require that alternate supplies be opened.

For additional details on the compensatory measures, refer to Attachment 3, "Outline for Compensatory Measures for Load Drop from OLS". Also, excerpts from UFSAR Chapter 9.2.2 are included for the system description of ERCW.

29. The licensee has committed to develop and issue plant procedures to delineate specific actions required in case of a heavy load drop. What will be the principle attributes of the plant procedures? When will the procedures be completed, who will require training on these procedures, and how far in advance will training be completed relative to heavy lift operations?

The actions for a heavy load drop will be contained within an Abnormal Operating Procedure (AOP). The major concern with a heavy load drop is the potential effect on Essential Raw Cooling Water (ERCW) to the operating Unit 2 and the potential for Auxiliary Building flooding The AOP will contain specific guidance to address a total ERCW flow blockage due to ERCW pipe crimping as well as a complete pipe rupture. The guidance will include proper parameters to monitor for evaluation of ERCW flow to Unit 2 with applicable shutdown criteria and guidance to maintain safe plant conditions. The AOP will also contain specific guidance for monitoring and controlling Auxiliary Building flooding that may occur from a pipe rupture. The AOP will be entered and implemented just prior to a heavy load lift occurring with the operating crew remaining in the AOP during the duration of the heavy lift. All of the operating *crews will receive training on this procedure during a cycle of operator requalification training that will be conducted in early 2003 prior to the Unit 1 Steam Generator Replacement Outage. In addition, 'just-in-time' refresher training will be conducted to specific applicable crew(s) prior to each heavy lift. The AOP will be completed in time to support the training that will occur during requalification training.*

30. The licensee has committed to isolate shared systems with Unit 2 or verify that they are capable of being isolated following a load drop, prior to handling a load over the containment with the outside lift system What systems are shared between Unit 1 and Unit 2 that could be impacted from a load drop over/in the vicinity of the containment? What Unit 2 safety-related functions could be impacted from such a load drop? How much time do the plant operators have to isolate these systems and how long will it take to perform the isolation functions?

    A review has been performed to identify any SSCs necessary to maintain safe shutdown that are shared with Unit 2 and located inside of the Unit 1 Containment that could potentially be impacted by the drop of a heavy load inside of the Unit 1 Containment during the defueled condition. Prior to the use of the OLS for handling of heavy loads inside and above the Unit 1 Containment during the defueled condition, the Essential Raw Cooling Water (ERCW) system and Component Cooling System (CCS) will be isolated with valves located outside of Containment. In addition, the Spent Fuel Pit (SFP) shall be isolated from the Unit 1 Containment. The isolation valves for the Control Air System inside containment are located well away from any potential load drops and would not be affected. Since the isolation valves will not be affected, any break in control air lines due to a load drop results in loss of air and failure of the isolation valves in the safe (closed) configuration.

31. What compensatory measures will be taken to minimize leakage through the temporary Unit 1 pipe tunnel wall from affecting safety-related equipment in the auxiliary building?

    As indicated in Section 8.2 of Topical Report 24370-TR-C-002, a wall will be installed in the ERCW tunnel near the Auxiliary Building interface. Since the wall has been designed for the hydrostatic head generated if the tunnel was completely filled with water and an impact load associated with the rushing water just after a pipe break, leakage through the wall is not expected following a load drop that results in the failure of piping inside the tunnel. Installation of this wall will be completed prior to movement of heavy loads that could cause a failure of the piping and tanks that penetrate the ERCW pipe tunnel.

    UFSAR Section 9.3.3.7 states that the Auxiliary Building has a passive sump that collects water from annulus drain sumps, and blowout panels located in the floors of the pipe chases and the Containment Spray and RHR pump rooms. Any leakage through the temporary pipe tunnel wall will eventually drain to the passive sump. Per UFSAR Section 6.3.2.11, the passive sump has a capacity of 209,000 gallons and a water level sensor in the passive sump alarms in the Main Control Room. Prior to the commencement of heavy load lifts with the OLS, the passive sump will be emptied.

32. What components are included in the weight of the lifted loads? List the loads to be lifted and whether the lifted loads are calculated or estimated. What means will be used to verify the weight of the lifted loads in the field?

    The generators will be lifted with rigging devices attached as well as any equipment (nozzle closure plates, insulation, upper lateral support, etc.) that will be attached during movement.

DRAFT

The lifted weight for the old steam generators (OSGs) includes the following components:

- *Generator (calculated)*
- *Lifting device (calculated)*
- *Internal water and sludge (conservatively estimated based on past projects)*
- *Nozzle cover plates (calculated)*
- *Lower lateral bumper blocks (calculated)*
- *Upper lateral support (calculated)*
- *Insulation (calculated)*
- *Rigging (calculated)*

The lifted weight of the new steam generators (RSGs) includes the following components:

- *Generator (calculated – will be confirmed during offloading upon arrival at site)*
- *Lifting trunnions (calculated – will be confirmed upon delivery to site)*
- *Lower lateral bumper blocks (calculated – will be confirmed after removal)*
- *Upper lateral support (calculated – will be confirmed upon delivery to site)*
- *Insulation support rings (calculated – will be confirmed upon delivery to site)*
- *Rigging (calculated)*

The OLS has a load cell incorporated into the crane that will be able to confirm the weight as each lift is performed. The OLS will lift the generators a few inches off their support and then hold. At this point the weight of the load will be confirmed and a systems check will be performed on the OLS prior to movement.

33. In Appendix A there is an item to "...develop and issue plant procedure(s) to delineate specific actions required in case of a heavy load drop." How will this condition, drop of the load, be communicated to the nuclear plant operators or site personnel?

    During the heavy load lifts, personnel observing the load lift will be in direct communication with the Main Control Room to relay status information of the lift to the Operating Crew. In addition, personnel will be in direct communication with the Main Control Room to monitor for Auxiliary Building flooding should a heavy load drop occur.

34. On page 12 it is stated that "[t]he input spectrum used for the horizontal direction is an amplified response spectrum at ground surface for an average soil depth to bedrock of 30 ft soil deposit and reduced to correspond to the minimum design basis from reference 27 which provides 5% damped free field top of soil response spectra curves for the Sequoyah Nuclear Plant for soil depths of 40 ft and 20 ft." Define or explain the meaning of "the minimum design basis." Was the amplified response spectrum input at the ground surface or 30 ft below it? How was the amplified response spectrum "reduced to"...as you stated? Explain in detail the way that you convolved the rock motion up through soil layers to obtain the amplified ground motion. Include details on the soil properties (seismic velocities, densities, soil modules and damping values, etc.) and soil layer thicknesses.

    Section 2.5.2.4 of the UFSAR provides discussion on the chronological sequence of development of the Sequoyah seismic design basis spectra at top of bedrock. The seismic safe shutdown earthquake (SSE) "minimum design basis spectrum" at top of bedrock for Sequoyah Nuclear Plant, as stated in Section 2.5 2.4 (p 2.5-21) of the UFSAR, is the modified Housner spectrum based on a peak acceleration (ZPA) of 0.18g and are indicated *as "Minimum Design Spectra" in Figures 2.5 2-11 through 2.5 2-14 of the UFSAR. Further from Section 2.5.2 4 of the UFSAR, it is noted that TVA used a more conservative arithmetically averaged response spectra generated by four artificial records as the SSE design response spectrum. These spectra are indicated as "Actual Design Spectra" in UFSAR Figures 2.5 2-11 through 2.5.2-14. Figures 2.5 2-11 through 2.5.2-14 of the UFSAR thus illustrate the relationship between the minimum design response spectra and the actual design spectra for different damping ratios.*

The seismic evaluation of the OLS (PTC Crane) is based on an appropriate ground spectrum corresponding to the minimum SSE design basis spectra. It is also noted that since the OLS is a temporary system that will be in service for approximately 3 months, the probability that it will experience an earthquake of the magnitude of the plant seismic design basis during its period of service is much smaller than that for a permanent plant structure, and hence use of a spectrum based on the minimum seismic design basis of the plant is very conservative. Table 3.7.1-1 of the Sequoyah UFSAR specifies a maximum damping of 5% for Category I bolted steel structures for SSE. Regulatory Guide 1.61 allows 7% for bolted steel structures for SSE. In order to keep the analysis conservative, the OLS seismic analysis is based on 5% damping response spectra.

The OLS is supported on a concrete ring foundation seated on a large number of piles anchored to bedrock. Based on borehole data taken during soil investigation for the Steam Generator Replacement Project (SGRP), the average depth of soil deposit above bedrock at the location of the OLS is approximately 30 ft. Since the OLS will be supported on top of a ~ 30 ft thick soil deposit above bedrock, the response spectra used in the analysis is an amplified spectrum at ground surface corresponding to the "minimum design basis" spectrum (see Section 2.5.2.4 and Figure 2 5.2-14 of UFSAR) for SSE at top of bed rock, as explained below.

The input spectrum used for the horizontal direction is an amplified response spectrum at ground surface. For a given soil deposit, the amplified ground spectrum is essentially a function of the depth of soil deposit. Reference 27 in Topical Report 24370-TR-C-002 provides 5% damped free field top of soil OBE response spectra curves for Sequoyah Nuclear Plant for soil depths of 40 ft and 20 ft. It is noted that the ground spectra developed in Reference 27 in Topical Report 24370-TR-C-002 are an average based on the four artificially generated time histories used to develop the more conservative "actual design spectra" (see Section 2.5.2.4 and Figure 2.5.2-14 of UFSAR). Reference 27 in Topical Report 24370-TR-C-002 further makes reference to TVA Report CEB-80-15, Rev. R0, "Preliminary Response Spectra for Ground Motion in Area of Diesel Generator Building and Cooling Towers". A 10% broadened SSE ground response spectrum for 5% damping for 30 ft depth of soil corresponding to the "minimum design basis spectra" in Figure 2.5.2-14 of the UFSAR was developed from the 20 ft and 40 ft curves in Reference 27 in Topical Report 24370-TR-C-002 as follows:

(i) The OBE ground spectra for 30 ft depth of soil was approximated by averaging the 20 ft and 40 ft response spectra curves on sheet 2 of Reference 27 in Topical Report 24370-TR-C-002. It is noted that this is conservative.

(ii) The SSE ground spectra for 30 ft depth of soil was obtained by multiplying the OBE curve obtained in step (i) above by 2 (see Section 2.5.2.4 of UFSAR).

(iii) The SSE ground spectrum obtained in step (ii) above is further reduced to correspond to a time history corresponding to the "minimum design basis" spectrum for a given frequency by multiplying by a factor given by the ratio of acceleration value

from the minimum design spectra to the corresponding acceleration value from the actual response spectra for that frequency.

(iv) The frequency axis is broadened by 10% (+ or -) to obtain a 10% broadened SSE ground horizontal response spectra for use in the seismic evaluation of the OLS.

The amplified input horizontal spectra for the OLS analysis, developed as explained above, were input at ground surface. Since the OLS is supported on a concrete ring foundation seated on a large number of piles that are supported well into bedrock, the vertical response spectrum used for the crane seismic analysis was the minimum design basis vertical spectrum for 5% damping from Figure 2.5.2-14 of the UFSAR. The vertical response spectrum used is 2/3rd (per Section 2.5.2.4 of UFSAR) the horizontal minimum design spectrum

The bedrock motion was amplified upward through the soil in Reference 27 in Topical Report 24370-TR-C-002 and TVA Report CEB-80-15. The amplified spectra from these references, used for developing the response spectrum for the OLS analysis, were developed based on a soil structure interaction evaluation methodology described in Section 3.7.1.6 of the UFSAR, extracts of which are reproduced below:

> *"For Category I structures (see Table 3.7.1-1) founded upon soils the rock motion was amplified to obtain the ground surface motion by considering the soil deposit as an elastic medium and making a dynamic analysis of a slice of unit thickness using only the horizontal shearing resistance of the soil A damping ratio of 10 percent is used for the soil. The four artificial earthquakes mentioned in Section 2.5.2.4 were considered as the input motion at the top of rock. Once the time history of surface accelerations was known, a response spectrum was produced for the analysis of the soil-supported structure. The ground surface response spectrum determined by a linear amplification of the bedrock motion was broadened by ±10 percent in order to obtain a design response spectra. The broadened curve was used as input to the dynamic seismic analysis."*

35. On page 13 it is stated that "[r]igging operations will not be performed when wind speeds exceed the maximum operating wind speed for the OLS." What is the wind speed measured in miles per hour considered to be the maximum operating wind speed? How was the maximum operating wind speed derived? Was there a stability analysis for the crane performed by considering the effects of the maximum operating wind speed on the crane and SG? If yes, provide the analysis results. If not, provide your justifications for the choice of the maximum operating wind speed

The maximum wind speed allowed during operation of the OLS (PTC Crane) when the lifted load is more than 3 ft off the ground is 10 m/s (22 mph) in any direction measured at the boom tip. The maximum wind speed allowed during operation of the OLS when the lifted load is at 3 ft or less off the ground is 15 m/s (33 mph) in any direction measured at the boom tip. See the response to Question 15 for a discussion of how this wind speed was derived. The stability analysis performed indicates that the OLS will be maintained at worse case 80% of the tipping load.

ATTACHMENT 3

OUTLINE FOR COMP MEASURE FOR RSG DROP

DISCUSSION

Due to physical proximity, several piping segments are in jeopardy of being broken or crimped by a heavy load drop. These lines include the Unit 1 B-Train (1B) Essential Raw Cooling Water (ERCW) supply header, the Unit 1 A-Train (1A) ERCW supply header, the A ERCW discharge header, and the Unit 1 Primary Water Storage Tank (PWST) and Refuel Water Storage Tank (RWST) along with the associated piping. The ERCW piping could rupture in the yard, i.e., underground, or in the Pipe Tunnel The RWST and PWST can rupture such that the water goes into the tunnels or onto the ground in the yard The RWST and PWST piping can rupture such that the water from the tanks will go into the tunnels or flow up around the tank onto the ground Additionally, a fire protection supply line is present in the tunnel. Of these, the only lines that have consequences for the operating Unit 2 are the ERCW supply and return headers Since the consequences for Unit 2 of pipe ruptures or crimping are potentially serious, a compensatory measure will be written The compensatory measure will be implemented by an Abnormal Operating Procedure (AOP)

In development of the AOP, a balance is being struck between several issues. Since time response in the various scenarios is important, as many actions as practical are being performed as prerequisites to the heavy load lifts However, caution is being used that no prerequisite would adversely impact items required to be OPERABLE by the Technical Specifications. Actions in the AOP will be written to address total flow blockage and total pipe rupture. However, the most likely affects to be seen include relatively small amounts of leakage or a pipe partially crimped but with no detectable flow loss. Since isolation of both the 1A and 1B supply headers removes from service both trains of equipment important to Unit 2 safety, isolation will not be performed unless the leak rate requires it. Analysis has shown that substantial leak rates can exist without adversely impacting equipment important to Unit 2 safe shut down Therefore, the AOP will contain isolation criteria based on leak rates for which isolation is to occur. The isolation criteria will consider the following items:

- Whether one or both trains are affected.
- Specific equipment important to Unit 2 safe shutdown that would be lost by supply header isolation. The equipment potentially affected includes the Diesel Generators, the Main Control Room Chillers, the 'A' Auxiliary Air Compressor, and the space coolers for the Component Cooling pumps
- The leak rate at which the safety related equipment on the Unit 2 headers will have flow degraded to the point of inoperability.

If a load drop occurs in the zone where the potential for piping damage exists, a rapid assessment will be performed of control room indication using pre-determined criteria placed in the AOP The assessment will first decide if the drop occurred in the zone where the potential for damage to the ERCW exists. If the drop occurred in this zone, preparations will immediately commence for an orderly shutdown of Unit 2. Any necessary field actions would then be evaluated for performance. Preliminary analysis has indicated that, even with total pipe severance of the 1A and/or 1B supply headers that the equipment most time-sensitive to damage (ie, centrifugal charging pumps) will be able to operate for extended periods without the planned operator actions occurring.

Due to the sizing of the piping and the ERCW system loads that would be present during the heavy lift timeframe, the supply headers would have to be almost completely crushed in order for the damage to even be detectable on instrumentation. Specifically, the flow that will be present during the critical outage timeframe in the 30" supply headers is expected to 300-500 gpm.

Certain actions in the compensatory measure will be taken prior to any heavy lift These actions have the purpose of simplifying and reducing the actions needed after a load drop. In all cases, the operator actions needed in the event of a heavy load drop are simple and few. The worst case scenario that would require operator action to be performed quickly is the clean guillotine type break in the supply headers. One of the prerequisite actions is to throttle valves upstream of the potential drop zone such that leakage from a break in this area will be minimized, thus extending the time available to Operations personnel for isolating a break.

If the criteria in the AOP requires the isolation of both the 1A and 1B ERCW supply headers, Tech Spec LCO 3.0 3 would be applicable for Unit 2 due to loss of cooling water to the common Emergency Diesel Generators, the common Main Control Room Chillers, and to the common space coolers for the Component Cooling pumps In order to restore this equipment to service, several actions are available. The Diesel Generators have alternate supply valves that would be opened from the Control Room. These valves would feed ERCW to the Diesel Generators from the opposite train, i.e., the A-train D/G would receive ERCW from the B-train. The Control Room chillers and Component Cooling pump space coolers will be restored either by opening the valves that connect the A-train to the B-train in the Auxiliary Building, or by installation of temporary spool pieces that will enable the ERCW to be fed from either the non-safety related Raw Cooling Water system or the High Pressure Fire Protection system

If a heavy load drop ruptured any of the various pipes in the ERCW pipe tunnel, the tunnel would become flooded. The temporary wall being erected at the entrance to the Pipe Tunnel from Auxiliary Building will contain the water. The temporary wall is being fitted with a sight glass and instrument connection which will allow detection and quantification of the water accumulating behind this wall. Means will be provided to quantify any leakage through the wall.

Specific actions to be accomplished after a heavy load drop are as follows:

- Rupture of the 1A ERCW supply header - Leakage more than the predetermined amount would be isolated by closing a single motor operated valve. If header isolation has occurred, open the alternate feed to the Diesel Generators. Monitor conditions at the temporary wall in the pipe tunnel.

- Rupture of the 1B ERCW supply header - Leakage more than the predetermined amount would be isolated by closing a single motor operated valve. If header isolation has occurred, open the alternate feed to the Diesel Generators. Monitor conditions at the temporary wall in the pipe tunnel.

- Rupture of the A ERCW discharge header - No immediate actions will be taken. The 2A Motor Driven Auxiliary Feedwater pump will be evaluated for adequate suction head. Monitor conditions at the temporary wall in the pipe tunnel

- Crimping of the 1A ERCW supply header - No actions will be taken unless other failures occur.

- Crimping of the 1B ERCW supply header - No actions will be taken unless other failures occur.

- A combination of crimping or isolation of the supply headers such that the 1A and 1B supply headers both have zero flow - Either open the train cross tie valves in the Auxiliary building or place in service the tie-in to supply components fed from the 1A and 1B ERCW supply headers in the Auxiliary Building from the Raw Cooling Water system or from the High Pressure Fire Protection system. If header isolation has occurred, open the alternate feed to the Diesel Generators

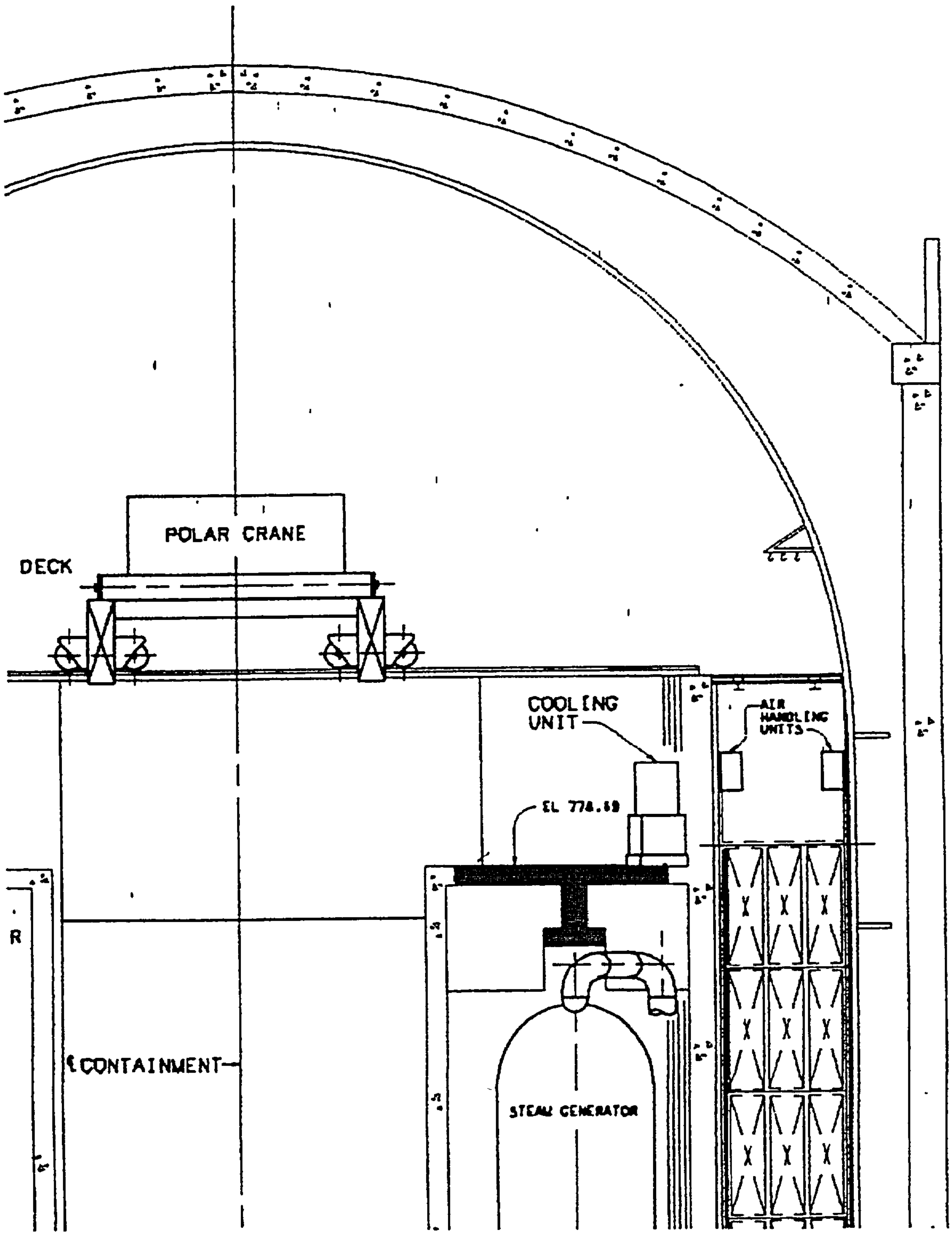
- Crimping of the A ERCW discharge header - No specific actions are required

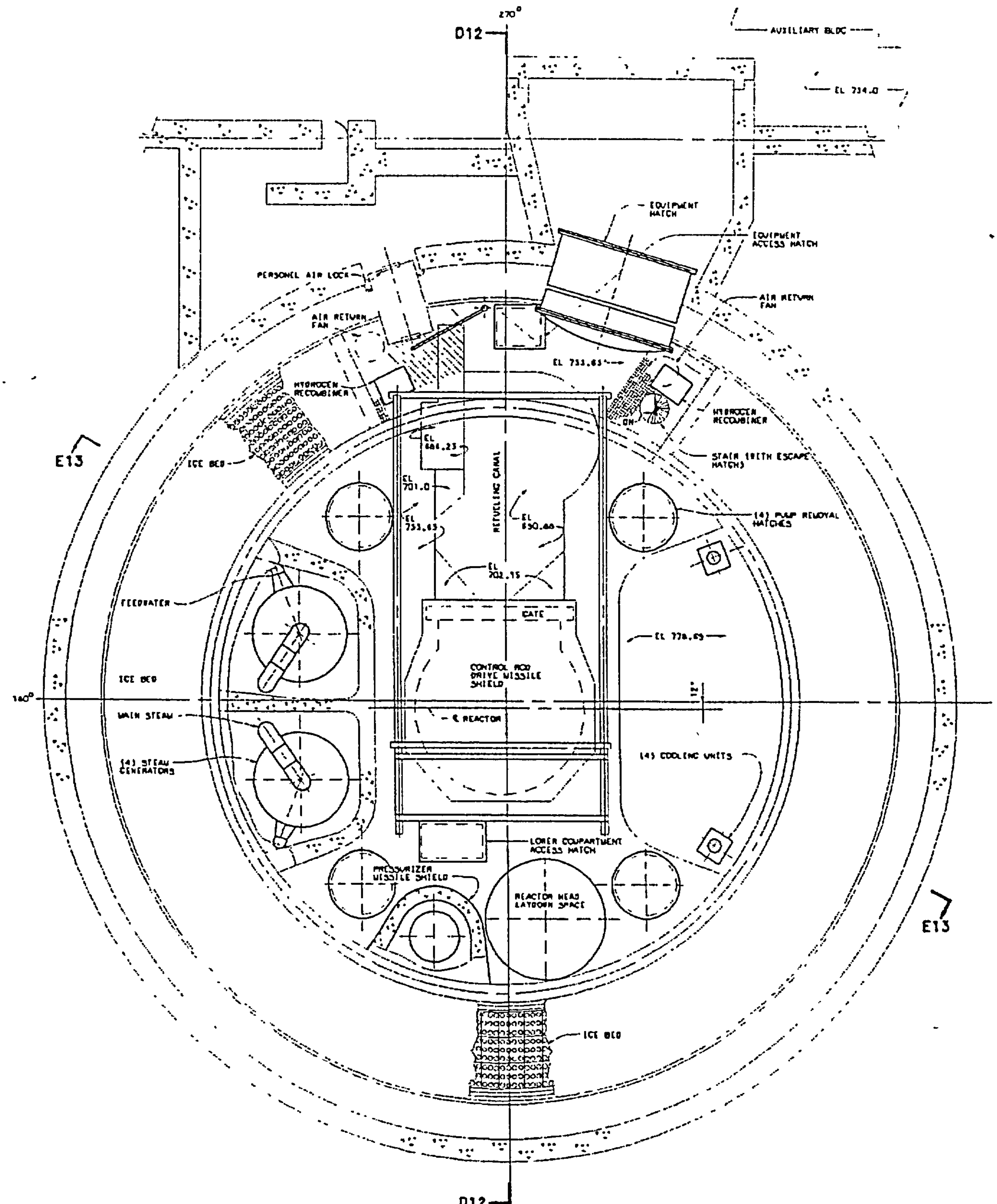
PREPARATORY ACTIONS:

These actions are in process of being finalized to ensure the greatest possible simplification of actions needed after a heavy load drop

1. Ensure that the temporary wall in the Unit 1 ERCW pipe tunnel is intact, all openings are sealed, and that a sight glass and pressure gauge are installed in the wall.
2. Develop criteria to quantify water accumulation behind the wall.
3. Ensure the Auxiliary Building passive sump level is less than 12".
4. Install a temporary weir or develop other criteria to enable quantification of the leakage entering the Auxiliary Building from the pipe tunnel.
5. Install temporary pressure and flow gauges at appropriate locations in the Auxiliary Building to monitor Unit 1 ERCW supply header pressures.
6. Ensure that the Spent Fuel Pool (SFP) is aligned to the Unit 1 CCS.
7. Close 1&2-FCV-67-22 & 24, ERCW cross-tie valves
8. Throttle open 0-67-552, 0-67-551, 0-FCV-67-152, and 0-FCV-67-151.
9. Station Operations personnel to visually monitor the crane activities.
10. Station Operations personnel at the ERCW pump station in order to isolate 1-FCV-67-489 (B1B-B ERCW strainer isolation) and 1-FCV-67-492 (A1A-A ERCW strainer isolation).
11. Station AUO on El. 669 to observe for leakage through the temporary wall.
12. Immediately prior to any load lift, mark the ERCW header pressures and flows on the MCR indicators. Also, mark the Unit 1 RWST level and the PWST level on the MCR indicators.
13. Ensure the 2B CCP is running.
14. Ensure that no air filters, herculite, etc. is covering the grating over the passive sump or the handrails around the opening
15. Isolate the High Pressure Fire Protection (HPFP)/Flood Mode Pump pipe by closing 1-26-575 and 1-26-653
16. Throttle supply header manual isolation valves 1-67-727A and 1-67-727B to pre-determined position

STEAM GENERATOR COMPARTMENT ROOF SPLICE PLATE DESIGN

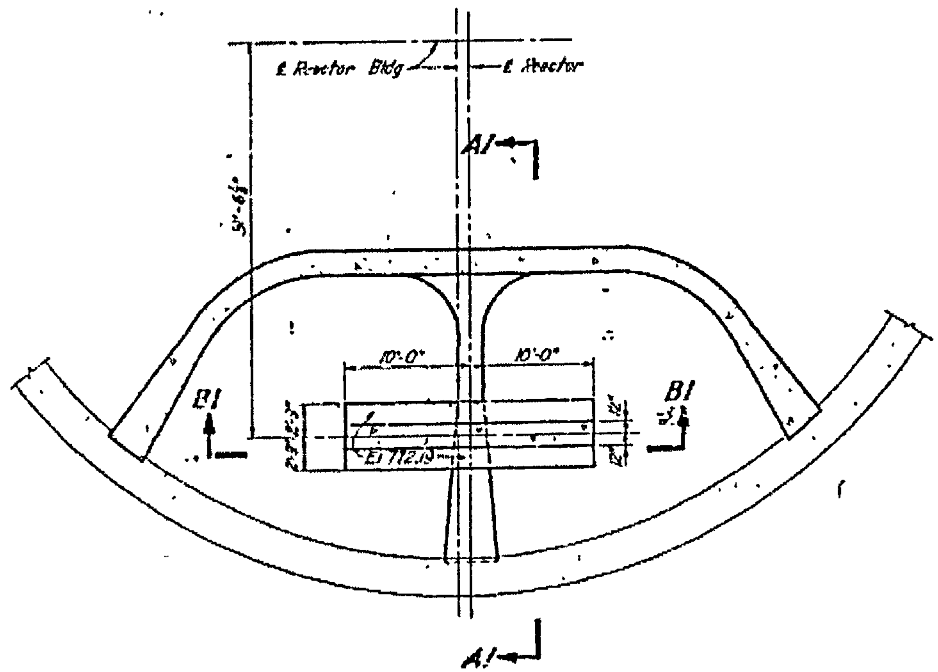

POLAR CRANE
DECK
COOLING UNIT
AIR HANDLING UNITS
R
℄ CONTAINMENT
STEAM GENERATOR



PLAN-UPPER COMPARTMENT

DESIGN OPTIONS

- Roof plug / splice plate
- Pourback
- Safety considerations
- Cleanliness inside containment
- ALARA



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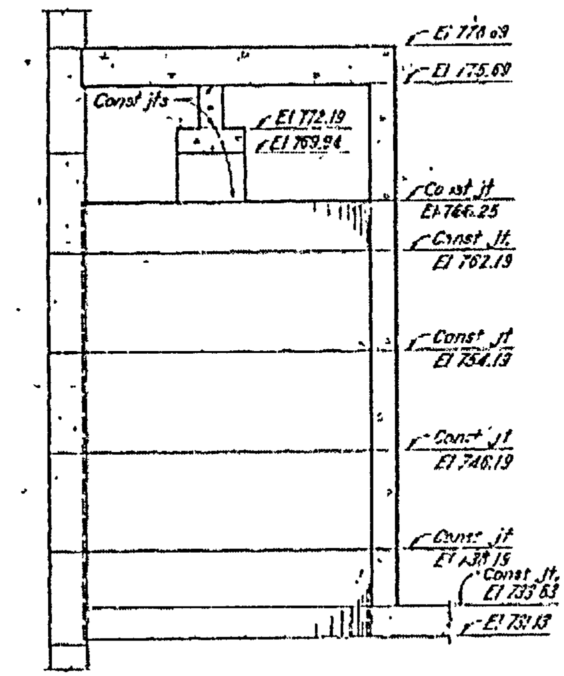
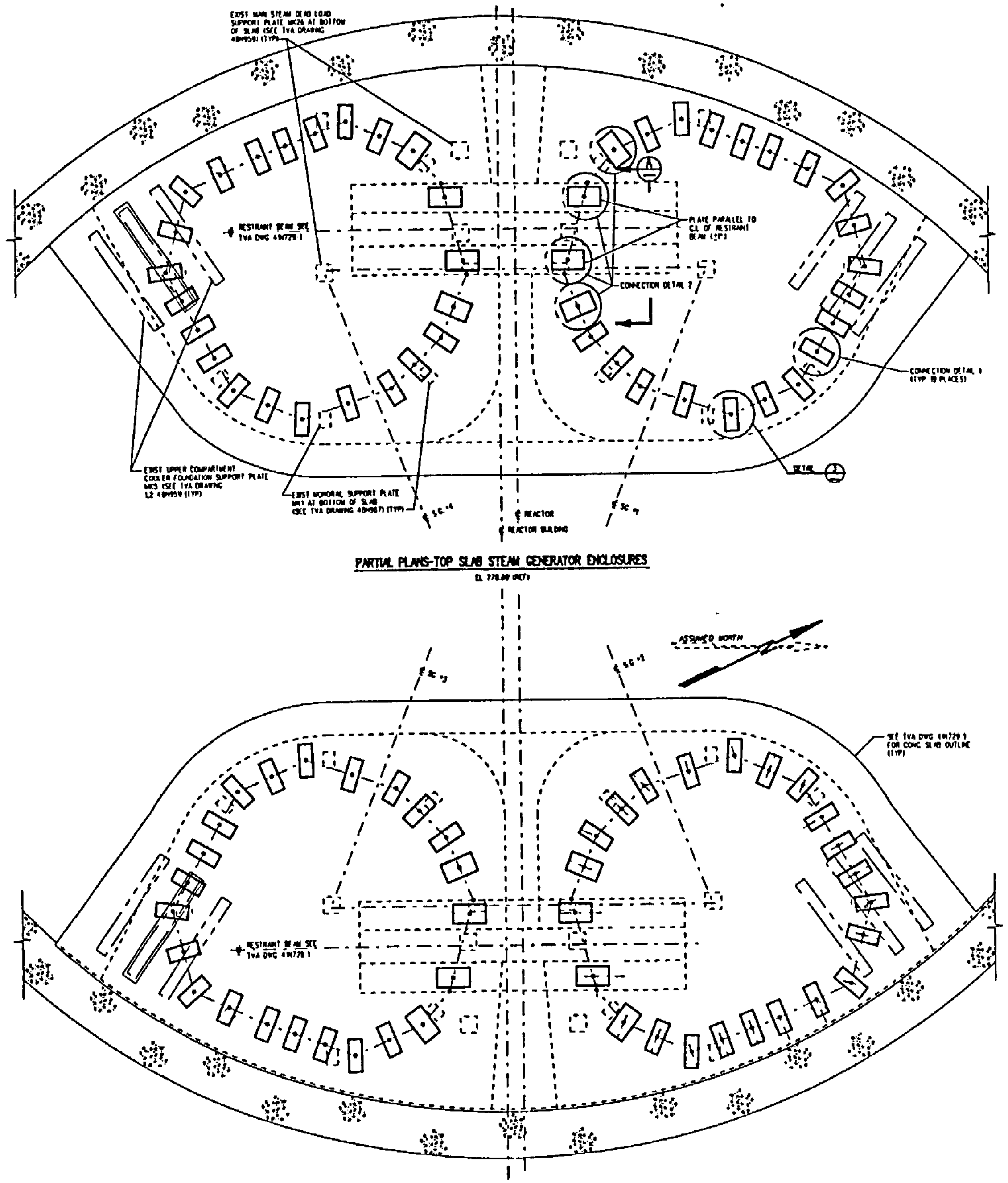
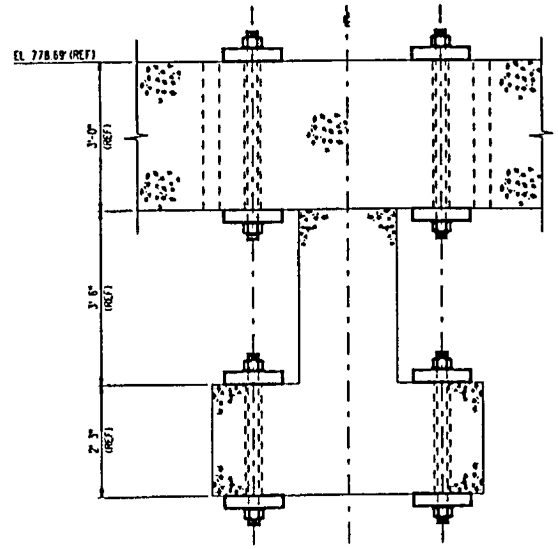


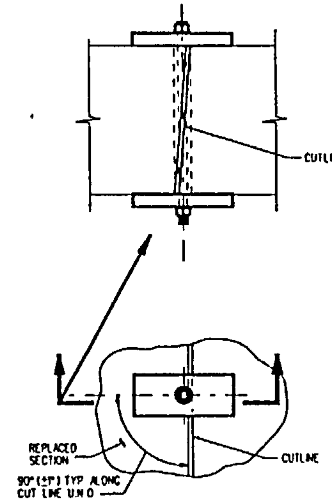
A1-A1



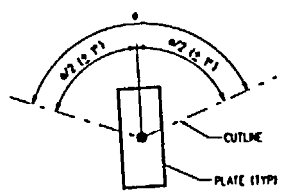
PARTIAL PLANS-TOP SLAB STEAM GENERATOR ENCLOSURES



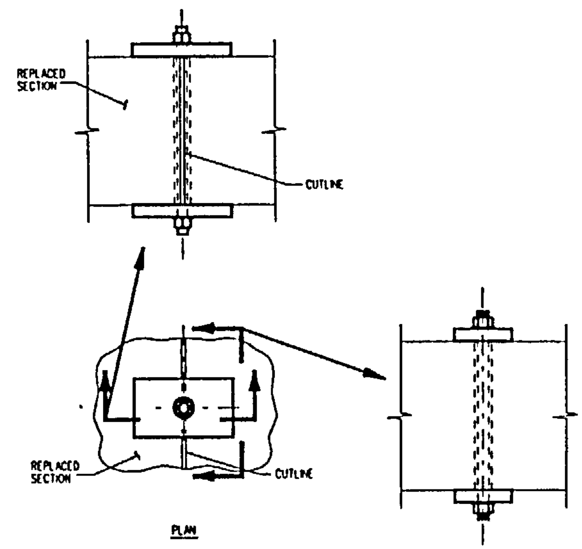
SECTION A



PLAN
CONNECTION DETAIL 1



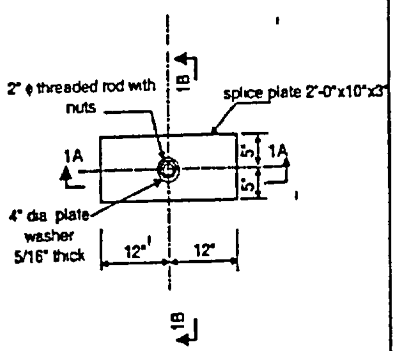
DETAIL 3
TYP AT CUTLINE INTERSECTIONS



PLAN
CONNECTION DETAIL 2

CONNECTION TYPES

CONNECTION TYPE 1
19 places



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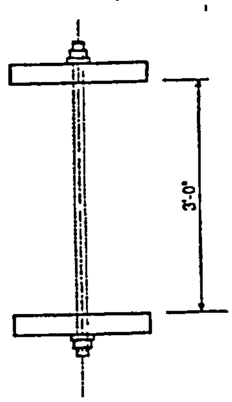
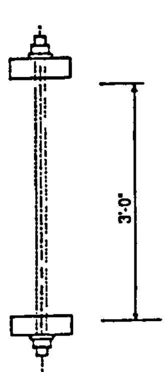
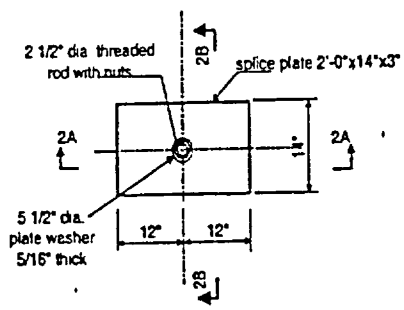


Section 1A-1A

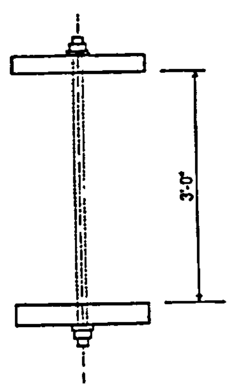


Section 1B-1B

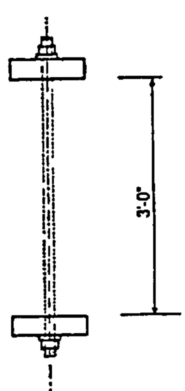
CONNECTION TYPE 2
4 places



Plan

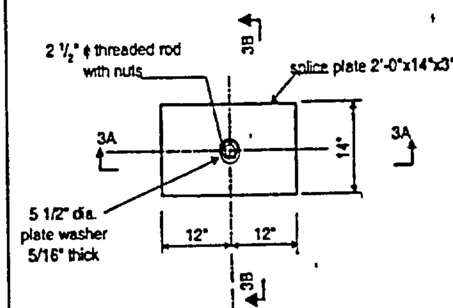


Section 2A-2A

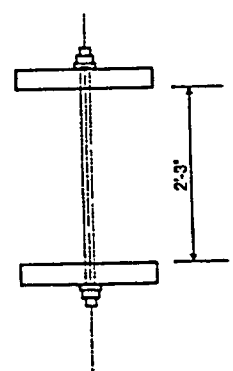


Section 2B-2B

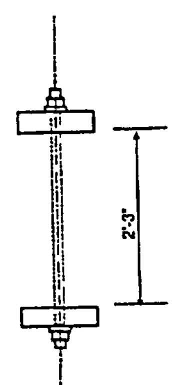
CONNECTION TYPE 3
2 places



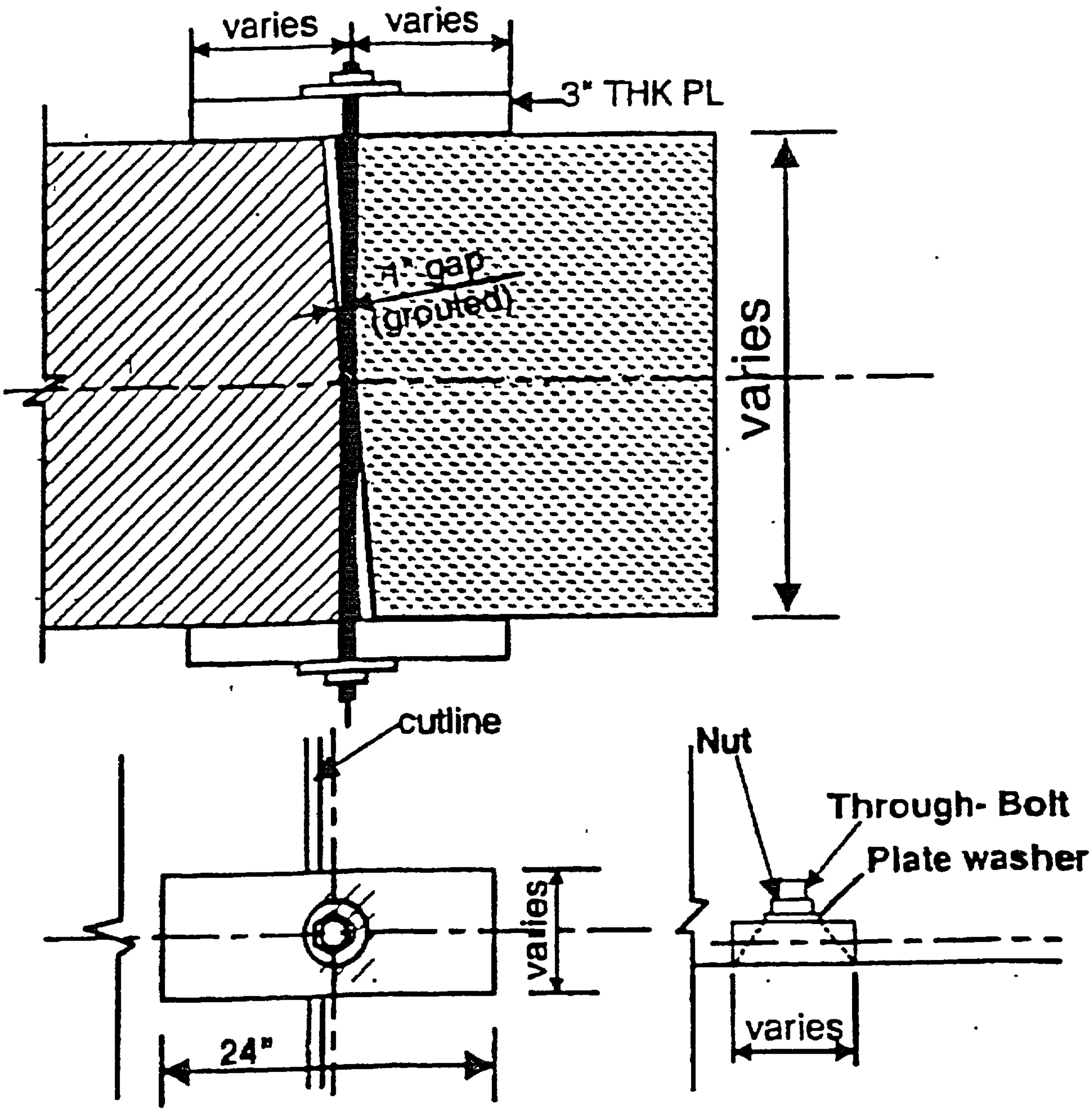
Plan



Section 3A-3A



Section 3B-3B


varies
varies
3" THK PL
1" gap
(grouted)
varies
cutline
varies
24"
Nut
Through- Bolt
Plate washer
varies

REMOVAL & INSTALLATION

- Bore holes at appropriate locations on the cut line
- Saw cut straight lines between bore holes to create access opening
- Remove plug
- Extract generator
- Bush-hammer concrete plug to provide annular gap for grout

REMOVAL & INSTALLATION

- Replace generator
- Position cut plug
- Install splice plates and thru bolts – snub tight
- Grout annular gap and cure
- Torque bolts to appropriate pre-load

Note: Mock-up will be used to simulate cutting and lifting

STRUCTURAL BEHAVIOR CONSIDERATIONS

Vertical Load

- Bolt pre-tension
- Shear transfer through splice plates
- Beam-on-elastic foundation action

Lateral Load

- Load transfer through grout
- No lateral load transfer by friction at plate/concrete interface

Bearing

- Conical dispersion through plate
- Beam-on-elastic foundation pressure distribution

DESIGN APPROACH

A number of configurations were investigated, with and without pre-load

- Conservative design approach
- Designéd for shear transfer through splice plate
- Shear transfer through grout neglected
- Concrete bearing based on:
  - Nut bearing area
  - Pressure distribution from beam-on-elastic foundation response

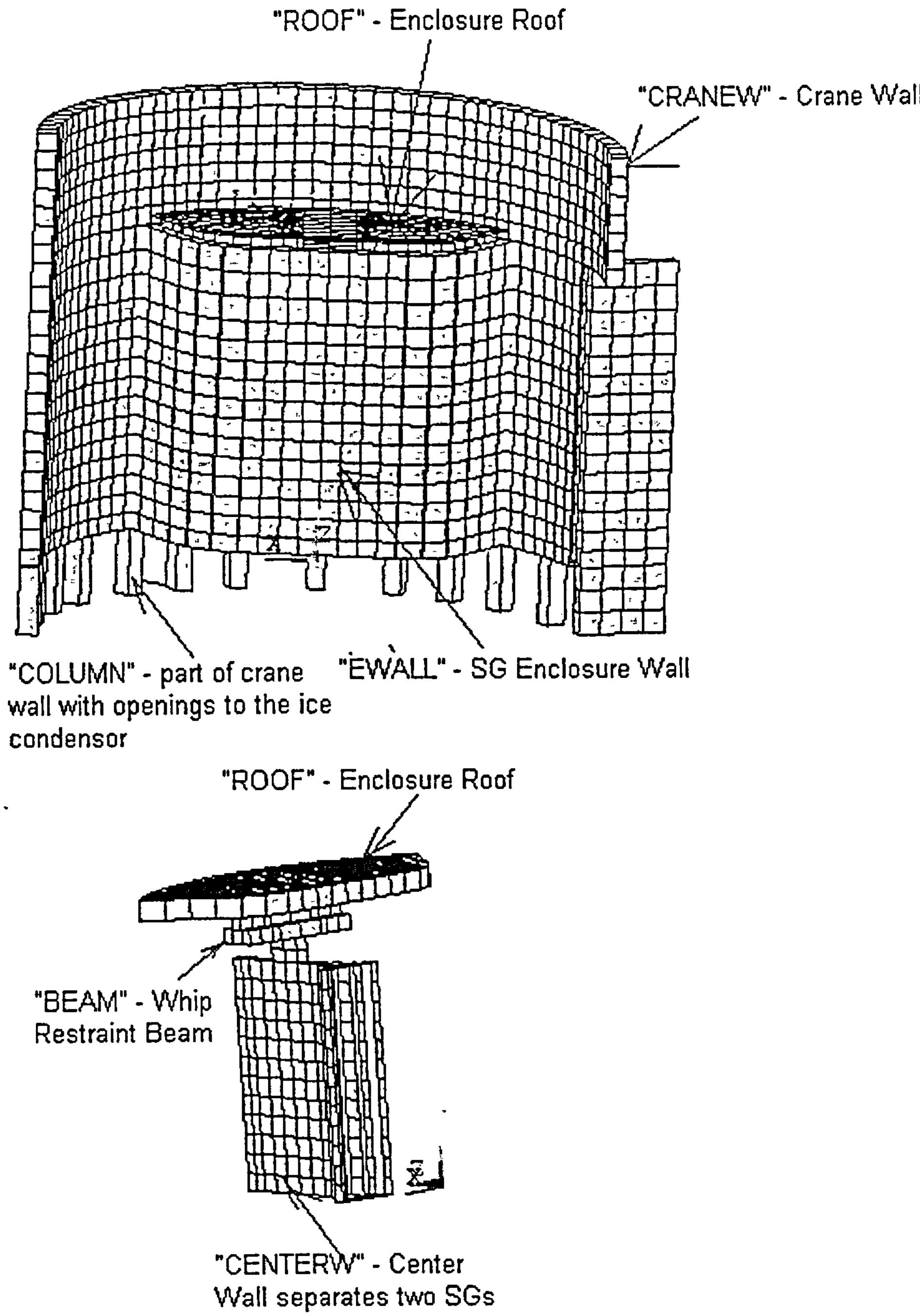


Figure 7-3 – Finite Element Model "SGE1" and "SGE2" and Element Groups and Global Coordinate Systems (Reference 6)

SPECIAL CONSIDERATIONS

- T-beams
- Entire SG Compartment has been re-evaluated for this modification – Loads from that 3D F.E. analysis were used to design the splices
- In F.E. model, nodes along cut line transmit vertical forces and compression only
- Micro-cracks

Table 7-2
Loading Combinations, Load Factors and Allowable Stresses for SG Compartment Roof Modification (5)(6)

Category	T_a	D	$L_{(1)}$	P_a	T_o	F_{eqo}	F_{eqs}	R_o	R_a	Y_r	Allowable Stresses
Service:											(Flexure)
											$f_c = 0.45 f'c$
Const	---	1.0	1.0	---	1.0	---	---	---	---	---	$f_s = 0.50 f_y$ (3)
Normal	---	1.0	1.0	---	1.0	1.0	---	1.0	---	---	(Shear) 50% of Factored (3)
Factored: Extreme Environmental	---	1.0	1.0	---	1.0	---	1.0	1.0	---	---	(Flexure) $f_c = 0.75 f'_c$ $f_s = 0.90 f_y$ (4)
Abnormal	1.0	1.0	1.0	1.5	---	---	---	---	1.0	---	(Shear) (2) $v_c = 2\sqrt{f_c'}$
Abnormal/ Severe Environmental	1.0	1.0	1.0	1.25	---	1.25	---	---	1.0	---	$\phi = 0.85$
Abnormal/ Extreme Environmental	1.0	1.0	1.0	1.0	---	---	1.0	---	1.0	1.0	

NOTES:

1. Includes all temporary construction loading during and after construction of containment.
2. v_c is lower for tension members and is given by $v_c = 2\sqrt{f_c'}\,(1 + 0.002N_u/A_g)$, with Nu negative for tension.
3. The allowable stress is increased by 33-1/3% when temperature effects are combined with other loads.
4. The tensile strain may exceed yield when the effects of thermal gradients are included in the load combination, i e., f_s can be <= f_y, and ε_s can be > ε_y when thermal effects are included.
5. The load combinations, load factors and allowable stresses in this table are based on the ASME Section III Division 2, 1975, which are, in general, consistent with the proposed ACI 359 - ASME Section III Division 2, 1973 with the exception of load factors associated with the Y_r load.
6. Structural steel components of the splice-plate connections were designed in accordance with TVA Design Criteria SQN-DC-V-1.3 2, Miscellaneous Steel Components for Class I Structures.

LOADS NOMENCLATURE:

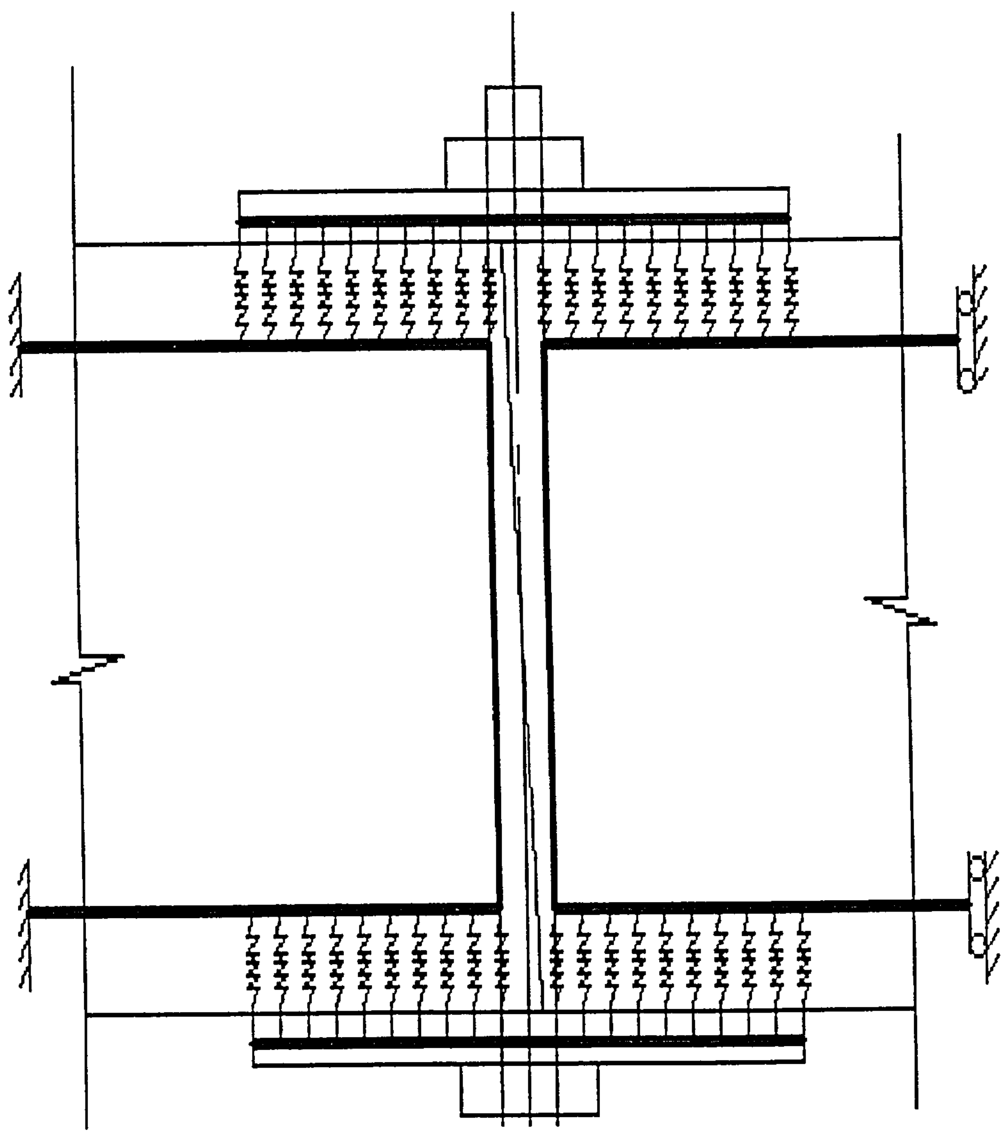
- D — Dead loads, or their related internal moments and forces
- F_{eqo} — Operating basis earthquake
- F_{eqs} — Design basis earthquake
- L — Live load, or their related internal moments and forces
- P_a — Accident/incident maximum pressure
- R_o — Piping loads during operating conditions
- R_a — Piping loads due to increased temperature resulting from the design accident
- T_a — Thermal loads under the thermal conditions generated by the postulated break and including T_o
- T_o — Operational temperature
- Y_r — Reaction load on broken pipe due to fluid discharge (corresponds to R_r in ASME Section III, Division 2, 1975)

* The term "design basis earthquake" has the same meaning as the term "safe shutdown earthquake."

MATERIAL PROPERTIES

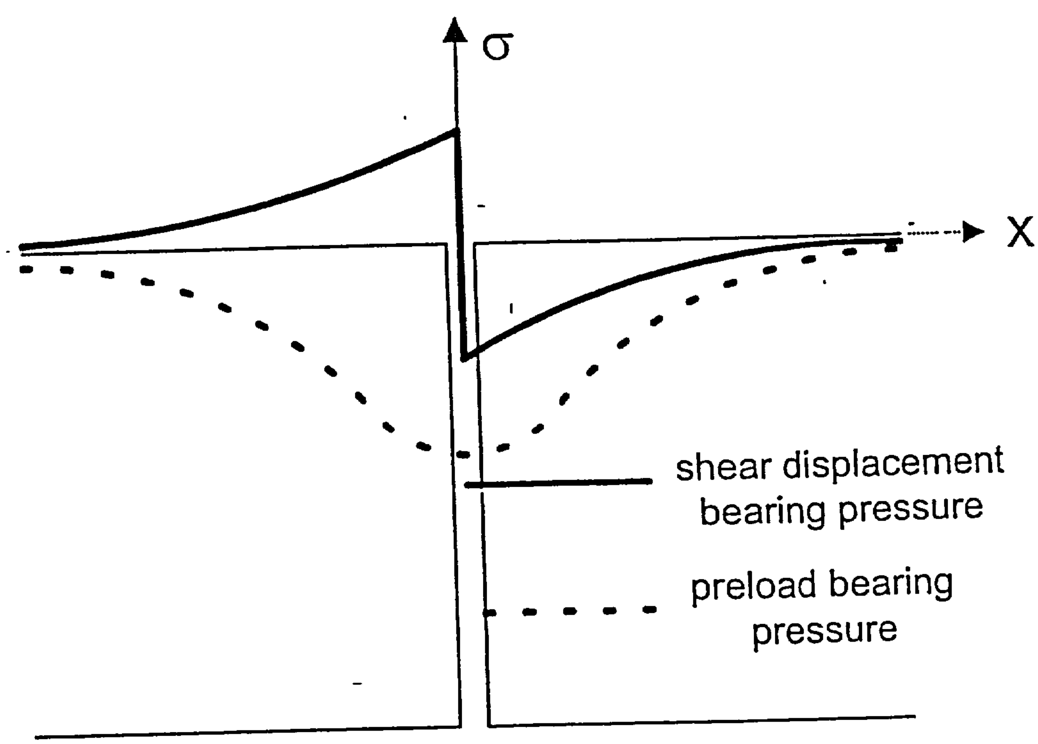
- 5700 psi concrete
- 5000 psi non-shrink grout
- ASTM A572 Gr 50 splice plates
- ASTM A193 Grade B7 threaded rods

STRUCTURAL MODEL



BEARING PRESSURE

DISTRIBUTIONS



CONCLUSIONS

- Splice plate connections satisfy design allowables
- Splice plate connection performs its safety function

RAI Questions 36 thru 39

- Topical Report No. 24370-TR-C-003, “Steam Generator Compartment Roof Modification”

(See Attached Questions and Responses)

36. On page 25 the report states, that "[m]ost of the connections consist of two splice plates, one at the top side and the other at the bottom side of the roof slab. The splice plates clamp the two roof slab sections together by means of a single threaded rod (with a nut and washer at the two ends running vertically through the plates and slab thickness in the core-bore holes." Since the words "most of the connections" were used instead of "all of the connections," describe other types of connections that are used for connecting the cut out portion of concrete compartment roof to the remaining portion of the concrete compartment roof. Also, since only one single threaded rod is used to connect the two roof slab sections, there is a reliance on a friction force between the steel plates and concrete, generated by the clamping force as a result of the post-tensioned threaded rod, to tie the two sections together. The staff finds that the friction force should not be relied upon for a positive connection. Discuss the rationale to address the staff's concern in this regard.

The other connections consist of two pairs of splice plates, one pair at the roof slab and the other pair at the reinforced concrete t-beam on the underside of the roof slab. Figure 7-2 in Topical Report 24370-TR-C-003 shows the different connection details. The design for both the single pair and two pair connections is based on the same engineering principles.

The splice plate connection design does not rely on friction. The joint between the steam generator compartment existing structure and the "plug" is modeled as a pinned connection that is only capable of transmitting vertical shear and in-plane compressive forces. The splice plate connections were evaluated for the effects from three possible worse case conditions. The conditions for each case are discussed below. None of the evaluations rely on friction between the plates and concrete.

Case 1 - The splice plates transfer the loads from the plug to the existing structure through flexure in the plate. This approach requires that sufficient bearing area under the plate be developed to avoid overstressing the concrete in bearing. This approach also conservatively assumes no grout in the annular space such that the concrete under the bearing area of the plate is not confined, resulting in lower allowable bearing stress in the concrete. This approach assumes that the bolt is not pre-loaded but only snug tight.

Case 2 - The bolts in the splice plate are pre-tensioned such that the splice plates act as clamps subject only to shear. This approach requires that the concrete under the splice plate be capable of resisting the bearing stress due to the pre-tension load in the bolt, which is higher than the anticipated design load. Again, this approach assumes no grout similar to Case 1. The use of 70% Fy for the pre-tension load in the bolt is consistent with AISC for high strength bolts.

Case 3 - The bolts in the splice plate are pre-tensioned as in Case 2, after the grout has reached its design strength. This Case reflects the actual installation sequence. This approach requires that the grout and in-situ concrete be capable of resisting the bearing stress due to the pre-tension load in the bolt. This approach takes advantage of the increased bearing stress allowable since the concrete and grout is confined. The design compressive strength of the grout is less than that of the in-situ concrete, however, and will govern the design.

The structural responses were within allowables for each of these cases.

37. On page 25 the report states that the bolt holes and annular space between the cut out portion of the concrete compartment roof to the remaining portion of the concrete compartment roof would be sealed by non-shrink grout, and that "conservative estimates (Reference 8) of the flow path through these micro-cracks yield values that are 1.6 percent of the total design bypass leakage flow area of five square feet discussed in Updated Final Safety Analysis Report Section 6.2.1.3.5." Reference 8 is TVA Calculation SCG-1S-609, Evaluation of Steam Generator Compartment Modification - Finite Element Analysis Results, Revision 0. Clarify how the calculated leakage was obtained from the finite element analysis results. Describe how the micro-cracks of the grout were mathematically modeled and the amount of leakage was calculated.

The grout is considered to be capable of transmitting in-plane compression only. In the finite element model, the interface nodes at the cutline between the cut-out "plug" and the remaining section of the enclosure roof were connected by elements capable of transmitting in-plane compression (provided by grout) and out-of-plane (vertical) shear (by splice plate connection) only. For the interface connection elements that are in in-plane tension, the analysis yields relative in-plane displacements between the two connected roof sections, which is considered to be the "micro-crack" width. Based on the analysis, the maximum relative displacement or "micro-crack" width under various loading combinations was found to be 0.015 inches, which is due to displacement of the plug under various loading combinations. By multiplying the maximum width of the crack times the perimeter of the plug, the maximum flow path was conservatively determined to be 0.084 ft^2 and it was this flow path area that was compared with the total design bypass leakage flow.

38. After reattaching the cut out portion of the concrete compartment roof to the remaining portion of the concrete compartment roof, what type of tests will be performed to verify that the leakage is within the allowable limit?

As described in UFSAR Section 6.2.1.3.5, a design basis bypass leakage flow area of 5 ft^2 has been established as the basis for analyses to assess the effects bypass leakage on containment pressure following postulated pipe breaks in the lower compartments. Following the replacement of the cut SG compartment concrete sections, the remaining annular space will be filled with a non-shrink grout that has been proven effective in sealing concrete barriers. The sealing of the annular space and the structural capabilities of the splice plates in restoring the compartment roof define the leak resistance of the restored structure. Additional contributions to the existing leak paths, other than the potential for micro-cracks discussed in Section 7.0 of Topical Report 24370-TR-C-003, are not envisioned from the repair.

The design of the lower compartment of the ice condenser containment is not conducive to a separate pressure or leakage test due to designed-in vents and passages. This limitation appears to be acknowledged by Standard Review Plan 3.8.3, which defines no required pressure proof-testing for the ice condenser compartments.

39. On page 27 the report states that "[t]he nodes at the cut-line along which the splice-plate connections are located were realistically modeled to transmit vertical forces and in-plane compression only." Was a zero force assumed in the vertical direction to be taken by the grout? If not, provide the justification on the amount of force in the vertical direction to be attributed to the grout.

As stated in the report and discussed in response to Question 34 above, the nodes along the cut-line are modeled to transmit vertical forces and compression only. The grout is not considered to resist any forces other than compression so that there is zero shear force resisted by the grout.

RAI Questions 1 thru 7

- Topical Report No. 24370-TR-C-001, “Alternate Rebar Splice - Bar Lock Mechanical Splices”

(See Attached Questions and Responses)

DRAFT

Draft Responses to NRC Request for Additional information on Topical Reports

Topical Report No. 24370-TR-C-001, "Alternate Rebar Splice - Bar-Lock Mechanical Splices"

1. Provide a copy of the Bechtel/INEEL test report for the Bar-Lock Mechanical Splices. The report should include information on who performed the splice tests, their qualifications, and how the tests were performed.

   A copy of the Bar-Lock test report prepared by INEEL is provided as Attachment 1. This report summarizes the test plan, results of rebar material testing, couplers tested, and results of the tensile and cyclic testing of the couplers.

   Based on the INEEL test plan, Bechtel developed a specification that defined the testing requirements. These test requirements were incorporated into the work plan and inspection record (WPIR) for controlling the Satec test machine setup, preparation of the Bar-Lock test specimens, and performance of the testing.

   Bechtel personnel performed testing of the Bar-Lock couplers at the Sequoyah site using a Satec 600VTL test machine. These personnel were trained by Instron/Satec in the use of the test machine.

   Calibration of the test machine was performed prior to its use and after completion of the Bar-Lock testing. Bechtel QC personnel reviewed the calibration documentation for acceptability.

   Rebar and coupler test specimens were prepared in accordance with Bar-Lock guidelines and the requirements of the Bechtel specification by personnel trained either by a Bar-Lock representative or by Bechtel personnel certified by Bar-Lock. TVA and Bechtel QA/QC personnel periodically monitored the preparation and testing of the test specimens.

   An INEEL representative was present during the initial setup of the Satec machine, programming of the test software, and witnessed the coupler testing.

2. Describe TVA's involvement, if any, in the Bechtel/INEEL test program.

   TVA was heavily involved in the Bechtel/INEEL test program

   - *TVA reviewed and approved the following specifications, procedures and test plans associated with the procurement, testing and installation of the Bar Lock couplers.*
     - *24370-C-311, "Technical Specification for Purchase of Bar-Lock Couplers"*
     - *24370-C-312, "Technical Specification for Installation of Bar-Lock Rebar Splices"*
     - *24370-C-602, "Technical Specification for Qualification Testing of Bar-Lock Mechanical Rebar Splices"*
     - *Construction Procedure CP-C-13, "Bar-Lock Rebar Splices"*
     - *"Test Program Plan for Qualification of Bar-Lock Coupler System for Use in Nuclear Safety-Related Applications", prepared by Idaho National Engineering and Environmental Laboratory*
   - *TVA Civil Engineers attended the vendor training session conducted at SQN on August 21, 2001.*

- *TVA Engineering and QA personnel witnessed the preparation of several test assemblies on August 21-22, 2001.*
- *TVA Engineering and QA personnel also witnessed testing of several specimens throughout the duration of the test program from October 11, 2001 to October 19, 2001.*
- *TVA reviewed and approved the Mechanical Testing Program and Performance Analysis, prepared by Idaho National Engineering and Environmental Laboratory.*

3. Clarify whether TVA has evaluated and determined that the QA programs of the reinforcing bar supplier (Consolidated Power Supply), the reinforcing bar fabricator (Birmingham Steel Corporation), the manufacturer of the Bar-Lock coupler (including lockshear bolt, and serrated rail), and the contractors who performed the tests (Bechtel/INEEL), meet the 10 CFR 50, Appendix B requirements? Provide the results of TVA's evaluations of theses QA programs.

   TVA has reviewed and approved Bechtel's Sequoyah Steam Generator Replacement (SGR) Project Nuclear Quality Assurance Manual. The policies in this manual correspond to each of the 18 criteria of 10CFR50, Appendix B and meet the requirements of ANSI N45.2 and N45.2 series standards and QA related NRC regulatory guides.

   Bechtel, in its role as a contractor to TVA, imposed the applicable 10CFR50 Appendix B requirements along with the technical and document submittal requirements on the subcontractors involved in the material supply, fabrication, and testing of the rebar and Bar-Lock couplers. Bechtel reviewed the quality programs for the rebar supplier (Consolidated Power Supply), the manufacturer of the Bar-Lock coupler (Valley Machining), and INEEL, and where appropriate, required changes to these programs to bring them into compliance with the requirements of 10CFR50, Appendix B. Bechtel specifications required their subcontractors to extend the specification requirements to their contractors.

4. On page 10 the report states that Bechtel has witnessed and verified implementation of Bar-Lock's manufacturing quality control processes and procedures for compliance with the applicable provisions of American National Standards Institute/ American Society of Mechanical Engineers (ANSI/ASME) N45.2. Identify and submit for staff's review the applicable provisions of ANSI/ASME N45.2 that were considered. Discuss how the Bar-Lock's manufacturing quality control processes and procedures comply with the 10 CFR 50, Appendix B requirements?

   The provisions/requirements of ANSI/ASME N45.2-77 that were considered applicable to the manufacturer of the Bar-Lock couplers (Valley Machining) are:

   5. *Quality Assurance Program*
   6. *Organization*
   7. *Procurement Document Control*
   8. *Instructions, Procedures, and Drawings*
   9. *Document Control*
   10. *Control of Purchased Material, Equipment, and Services*
   11. *Identification and Control of Materials, Parts, and Components*
   12. *Control of Special Processes*
   13. *Inspection*
   14. *Test Control*
   15. *Control of Measuring and Test Equipment*
   16. *Handling, Storage, and Shipping*

17. *Inspection, Test, and Operating Status*
18. *Nonconforming Items*
19. *Corrective Action*
20. *Quality Assurance Records*
21. *Audits*

Review of the Bar-Lock manufacturing processes along with the provisions of the specification for the purchase of the Bar-Lock couplers as described below assures that the corresponding requirements of 10CFR50, Appendix B are also met.

A specification, written for the purchase of the Bar-Lock couplers, identified the technical requirements the Bar-Lock manufacturer was required to meet. These requirements covered applicable codes and standards, quality, shipping, handling, storage, critical processes and parameters, and documentation. Bechtel QA personnel performed surveillances during the manufacturing of the Bar-Lock couplers to verify that the manufacturing process was performed in a manner that was consistent with the specification. The critical processes identified in the specification and the results of the Bechtel QA surveillances are summarized below:

a. *Application of material traceability identification on bolt, tube, and saddle material*

The material traceability of each heat lot of material for the tubing, hex stock for bolting, and square stock for the saddles was verified by review of the mill tag affixed to each bundle of material and visual verification of the physical markings on the stock. The material test reports were reviewed to verify material composition and strength were as required by the specification.

b. *Tapping of bolt hole*

The drilling and tapping of bolt holes was performed in one machine operation. The hole locations were checked initially by the machinist and by the inspector when the machine was set up. Set up pieces were identified as such and were not included as part of the production run. When the production run began, the finished holes were checked on a random basis by the machinist and by the roving inspector using a calibrated go/no go plug gauge. In addition, 100% of the threaded holes were verified as completely drilled and tapped since each coupler is fully assembled with the bolts installed at final assembly and inspection. This process was monitored by Bechtel QA and Bar-Lock personnel throughout the drilling and tapping process. No deviations from the design drawing were noted.

c. *Induction heating of bolt tip*

The induction heating process was monitored on a periodic basis by Bechtel QA personnel and by the operator and QC inspector. Six samples were taken by the operator and verified by the QC inspector at approximately four hour intervals during the induction hardening process. The tested bolts all fell within the specified hardness range.

d. *Fusion of saddles to tube*

The weld of the saddle to the tube is critical only to the extent that it needs to hold the saddles in position until the bar is inserted and the bolts set. There is no credit taken for *the weld in the ability of the coupler to withstand the required tensile and cyclic performance criteria. The weld is tested on a random basis by the QC inspector by dropping the coupler from a height of 5 ft onto concrete. If there is no weld failure, the weld is considered acceptable. There were no failures noted during these tests.*

e. Bolt shear testing

Each shear value bolt test was witnessed by Bechtel QA personnel Unique heat lot numbers were assigned to each batch of bolts sent to the heat treatment facility. After heat treating and quench, the bolts were tested at the heat treatment facility for hardness to determine the amount of time and temperature required in the draw furnace. After final treatment the bolts were again checked for hardness to verify conformance with the required hardness. The shear testing for each lot resulted in satisfactory results. Each bolt was stamped during the machining operation with the letters VMC to help assure that no other bolts would be co-mingled with the produced for Sequoyah.

f. Heat treatment condition of saddles

After machining, the saddles were heat treated and case hardened. Bechtel QA personnel witnessed the furnace load time and verified the furnace temperature. Fifty-three saddles of each size were tested to verify that the required minimum case hardening depth and hardness were achieved. The results were satisfactory.

The critical parameters identified in the specification were:

g. Length of tube
h. Inside diameter of tube
i. Outside diameter of tube
j. Number of bolts
k. Saddle location
l. Bolt spacing
m. Bolt edge distance
n. Bolt threads
o. Bolt tip hardness
p. Diameter of bolt shear plane
q. Actual bolt break-point torque values

The critical parameters listed above were verified by Valley Machining machine operators and QC personnel. Bechtel QA personnel verified each of these parameters during regular monitoring throughout the manufacturing process.

All measurements were made using equipment calibrated under a controlled calibration program with standards of calibration being traceable to NIST or another nationally recognized standards. Calibration records were reviewed by Bechtel QA personnel.

The supplier procurement documents from Bar-Lock to Valley Machining were reviewed by Bechtel QA personnel for the coupler design for nuclear safety-related applications. In addition, the procurement documents for the tube material, hex stock for bolts, and square stock for the saddles were reviewed.

Bechtel QA personnel examined a completed container of couplers for shipping preparation and container identification. The preparation was found to comply with the requirements of ANSI N45.2.2, Level C, as required by the specification.

5. On page 11 of the report it states that "[s]ince the Bar-Lock couplers will be used in a nuclear safety-related application, they are subject to a commercial grade dedication program." Describe and submit the commercial grade dedication program for staff's review.

The TVA dedication program for procurement and use of commercial grade items in safety-related applications is based on guidelines contained in Electric Power Research Institute (EPRI) Report No. NP-5652, "Guideline for the Utilization of Commercial Grade Items in Nuclear Safety Related Applications". TVA procedures require the use of one (or any combination of) the methods described in the report for dedication of commercial grade items Based on the nature of the Bar-Lock coupler procurement (i.e., an infrequent procurement of a specialized component), the "source verification" method described in Section 3.3 of the EPRI report was used. Under this dedication process, a component-specific specification was developed (as discussed in the response to Question 4) which established the Codes, Standards and quality assurance requirements for fabrication of the couplers The specification established minimum material and tensile strength requirements based upon the safety function performed by the coupler and identified the critical processes and parameters requiring verification to ensure compliance with the established functional requirements.

To verify conformance with the requirements of the specification, source surveillance of the manufacturer's facility and fabrication activities was performed prior to and during component manufacture. The scope of the surveillance activities verified compliance with the quality assurance and critical parameter requirements of the specification. The results of the inspections, tests and certifications performed during source surveillance activities were documented in a material fabrication report compiled by the manufacturer. This documentation was reviewed by TVA as part of the component receipt inspection and was confirmed to be adequate to establish the component critical characteristics under the "source verification" dedication method outlined in EPRI Report No. NP-5652.

6. On page 12 of the report it states that the records of bolt shear test results were examined Describe how the bolt shear test was conducted and submit a typical bolt shear test result, including the relationship between applied shear force and recorded shear deformation of a test bolt.

The bolt shear-torque test was conducted. The shear-torque was tested by gripping the end of the bolt to secure it, and then torquing the bolt until the head sheared off. The torque wrench used for the test had a memory device capable of recording shear-torque of the bolt head. The bolts were inspected and tested to meet the Bar Lock Bolt Specifications. The major diameter, pitch, fit and length were inspected and recorded. The shear-torque (ft-lbs) value at bolt head break was also recorded. These values were recorded for each sample set on Valley Machining Form POP-05 #3. Typical inspection and testing record sheets are provided as Attachment 2.

The shear deformation at the bolt head was not specifically tested. Any deformation that occurs due to the shear-torque test will be localized, occurring in the shear plane of the bolt head break. The bolt head break is located outside the active area of the coupler and would therefore have no impact on the strength, reliability and function of the coupler.

7. The Bar-Lock coupler system relies on the clamping force generated on the rebars between the lockshear bolts and serrated rails Provide the magnitude of the compressive stress and force on the tip of a lockshear bolt and the strain in the bolt after the bolt installation. Provide the stress relaxation characteristic of the lockshear bolt (relaxation is defined as the loss of its compressive stress under strain for a period of time). Provide evidence that the clamping force generated by the lockshear bolt would not be reduced, as a result of the relaxation phenomenon, to a point that would degrade the proper function of the Bar-Lock coupler system during the life of the plant.

   The Bar-Lock bolt tips are hardened to a level that exceed the hardness of the rebar, ensuring no plastic deformation of the bolt tips. The results of the testing performed at Sequoyah confirmed this design, in that where the splice failure mode was rebar pull-out, the rebar had been damaged by the bolt tips, while no bolt tip failures were experienced. Note that the splice failure occurred well after the design load was reached. To show that the design properly accounts for the stress and strain is evidenced in the reliability of the couplers tested in this qualification process

   Stress relaxation is associated with materials within or very near their creep temperature ranges. For carbon and low alloy steel bolting, stress relaxation is not considered a concern at ambient temperatures. Under these conditions, the stress in the Bar-Lock coupler is not time dependent

INEEL/EXT-02-01387

Dec 2001

Qualification of the Bar-Lock Rebar Coupler For Use in Nuclear Safety-Related Applications Mechanical Testing Program and Performance Analysis

W. R. Lloyd

INEEL/EXT-02-01387

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Published Dec 2001

Idaho National Engineering and Environmental Laboratory
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Idaho Falls, Idaho 83415-2218

Summary

Bechtel Corporation and INEEL developed and performed an independent mechanical testing and analysis program to assess the mechanical performance characteristics of the Bar-Lock L-Series rebar coupler system. A test plan that exceeded the assessment requirements given in ASME Section CC-4333 was developed. To achieve high statistical confidence in measured sample parameters, e.g. ultimate strength, the number of specimens tested was increased to forty (40) from the ASME Code-required quantity of six (6). Bechtel QA/QC personnel monitored the testing program to ensure that it was performed in accordance with the requirements in Specification 24370-C-602.

Static strength tests of two sizes, #6 and #8, of Bar-Lock coupler assemblies showed that they exceeded the ASME-specified minimum strength levels by large margins. Statistical analysis of the results showed a 99.998% probability that the average strength of a group of coupler assemblies would exceed the ASME static strength requirement of 90% of the joined rebar tensile strength. Assessing the performance of individual coupler assemblies against the ASME-specified minimum strength (75 ksi for the Grade 60 rebar used in the tests) for individual assemblies showed that the average strength of an individual assembly was more than 8 standard deviations above the specified minimum. This corresponds to the probability that essentially 100% of all coupler assemblies would exceed the specified minimum strength.

Forty specimens of each of the two sizes (6L and 8L) of coupler/rebar assembly were tested to determine their cyclic loading durability. The test procedure cycled each assembly between 5 and 90% of specified minimum bar yield strength (60 ksi) 100 times. None of the specimens failed in any manner, e.g. bar break, or bar slip within the coupler.

In an effort to improve the cyclic durability performance assessment, several randomly selected specimens received additional cyclic loading. Each selected specimen had an additional 1000 loading cycles imposed. None of the specimens failed, and none of them showed signs of deterioration through excessive strain accumulation or physical deformation. This provides an empirical indication that the cyclic durability of the couplers will far exceed 100 cycles.

Further, some coupler assemblies randomly selected from those already receiving 100 loading cycles were subsequently loaded to failure monotonically (static strength test). This test determined if the prescribed cyclic loading substantially damages the integrity or strength of the coupler splice assembly. The eight specimens tested all achieved the same nominal strength as like specimens receiving no cyclic loading.

The Bechtel/INEEL test program tested and demonstrated that the mechanical properties of the L-Series Bar-Lock mechanical splices meet the existing Codes and NRC requirements and are an acceptable method of connecting reinforcing bar in nuclear power plant safety-related applications. The large quantity of couplers tested provides a higher confidence that the couplers do meet, and indeed far exceed, those ASME-specified requirements.

CONTENTS

FIGURES

TABLES

Qualification of the Bar-Lock Rebar Coupler for Use in Nuclear Safety-Related Applications: Mechanical Testing Program and Performance Analysis

1. OVERVIEW

Bechtel Corporation and INEEL developed and performed an independent mechanical testing and analysis program to assess the mechanical performance characteristics of the Bar-Lock L-Series rebar coupler system. By design, this program provided a very rigorous test of coupler design mechanical performance, using the qualification criteria of **ASME Section III, Division 2, CC-4333** as a standard of reference.

The Bechtel/INEEL test program tested and demonstrated that the mechanical properties of the L-Series Bar-Lock mechanical splices meet the existing Codes and NRC requirements and are an acceptable method of connecting reinforcing bar in nuclear power plant safety-related applications.

2. TEST PLAN

ASME Section CC-4333 specifies performance criteria to qualify rebar splicing devices for use in nuclear safety-related applications. While the strength specifications are moderately high, the quantity of test specimens required is relatively low. To achieve high statistical confidence in measured sample parameters, e.g. ultimate strength, a larger sample size (n) is required. To achieve the desired level of confidence that any installation of these couplers will have the requisite performance characteristics, the quantity of verification test specimens (the sample set) was increased. For the static strength assessment, the ASME Code requires six specimens be tested, and all six must pass. In this test plan, the quantity was increased to n = 40 for each size tested. For the cyclic durability test, the ASME Code requires three specimens to survive the 100-cycle test. This was increased to n = 40 for each size. Increasing the statistical sample size from six or three to 40 allows a great improvement in the confidence levels (especially for the binomial distribution of the cyclic test) associated with lower bound strength and cyclic durability requirements specified in the Code.

The Bar-Lock testing was monitored by Bechtel QA/QC personnel to ensure that it was performed in accordance with the requirements in **Specification 24370-C-602**.

3. REINFORCING BAR MECHANICAL PROPERTIES TESTS

Mechanical properties for the rebar material used in these tests were determined in accordance with project test procedures, incorporating relevant American Society for Testing and Materials (ASTM) test standards and procedures (**ASTM Designation A 370-96, Standard Test Methods and Definitions for Mechanical Testing of Steel Products**; and **ASTM Designation E 8-99, Standard Test Methods for Tension Testing of Metallic Materials**). All mechanical properties tests were performed on the same universal test machine, using the same measurement transducers. The same test machine, load cell, and extensometer were used in all of the coupler assembly tests as well. Bechtel Quality Assurance Department retains all calibration certification and records for this equipment and these devices.

The reinforcing bar used in the Bar-Lock coupler testing program was ASTM A615 Grade 60 material in #6 (¾ in. nominal diameter) and #8 (1 in. nominal diameter) sizes. Consolidated Power Supply, the vendor of the rebar, provided certified material test reports (CMTRs). The values reported in the CMTRs are based on the results of a single tensile test. The CMTR value, while confirming the nominal material performance, is inadequate to determine "actual" material properties. The ASTM test standard recommends a minimum of three specimens be tested and the results averaged. Additional verification testing was performed as part of this test program to determine the "actual" or measured mechanical properties of the different heats of rebar employed in specimen assembly. Figures 1 and 2 show representative stress-strain curves for both heats of re-bar used in this test program.

3.1 #6 Re-Bar Material

A common heat of rebar (CPS #5898I2899) was used in making up all #6-size coupler test assemblies. Per ASME Section II, Division 2 requirements, the same 10 inch extensometer gage length, as would be used in the #6 coupler assembly tests, was used to measure strain in the tensile properties tests. Seven #6-size plain bar sections from this heat were tested to determine actual tensile properties of this lot of material. Table 1 summarizes the test results. Material properties obtained from Consolidated Power Supply CMTR are provided for comparison.

It is apparent that the differences in yield strength as determined by three different definitions are minimal. For this type of steel, the yield point is the appropriate measurement and provides the most consistent value (smallest standard deviation). Where "measured" or "actual" yield strength is required in the analyses, 67.7 ksi is used for the #6L coupler tests. Where "measured" or "actual" ultimate tensile strength (UTS, or F_u) is required in the analyses, 107.5 ksi is used for the #6 tests.

Table 1. Mechanical Properties of Rebar Used in Test Specimens

	Yield Point (ksi)	0.2%OS Yield (ksi)	0.5% EUL Yield (ksi)	UTS (ksi)	Elongation (%)	E (Msi)
#6 Average	67.7	67.9	68.2	107.5	13.2	27.8
#6 Std Dev	1.03	1.19	1.14	1.12	1.26	0.89
#6 CMTR	--	--	67.6	107.4	15	--
#8 Average	72.6	72.4	72.5	110.1	11.5	29.2
#8 Std Dev	0.45	0.57	0.47	0.74	0.98	0.46
#8 CMTR	--	--	73.1	112.0	14	--
#8 CMTR (C-series only)	--	69.0	--	112.8	16	--

3.2 #8 Re-Bar Material

A common heat of rebar (CPS #5898I3260) was used in making up all of the #8-size coupler test assemblies used in the tensile strength tests. Per ASME requirements, the same 14.5 inch extensometer gage length was used in the tensile properties test as would be used in the #8 coupler assembly tests. Seven #8-size plain bar sections from this heat were tested to determine actual tensile properties of this lot of material. Table 1 summarizes the results of those tests. Material properties obtained from Consolidated Power Supply CMTR are also provided for comparison. Again, the yield point strength is selected for the material yield strength value. Where "measured" or "actual" yield strength is required in the analyses, 72.6 ksi is used for the #8 tests. Where "measured" or "actual" ultimate strength (UTS) is required in the analyses, 110.1 ksi is used for the #8 tests.

3.3 Material for #8 Coupler Size Cyclic Durability Tests

A separate heat of rebar material (CPS #123741) was used to fabricate the size #8 cyclic test coupler assemblies. There are no measured strength parameters (only specified minimums) associated with the cyclic test procedures, so no verification testing of this material was performed. The CMTR-reported values for this heat are provided at the bottom of Table 1 for reference.

4. DESCRIPTION OF COUPLER TEST SPECIMENS

The Bar-Lock couplers used are Bar-Lock's "L-Series" (coupler designations 6L and 8L), which are higher strength rebar couplers for use in tension/compression, seismic and other cyclic load conditions. The specifications for these couplers are provided in Table 2.

Table 2. Bar-Lock L-Series Coupler Specifications (Sizes #6 and #8)

Coupler Designation	For Use on Rebar Size	Coupler Specifications			Bolt Specifications		
		Outside Diameter (inch)	Length (inch)	Nominal Weight (lbs.)	Quantity per Bar	Size (inch)	Nominal Shear Torque (ft.-lb.)
6L	#6	1.9	8.0	4.5	4	1/2	80
8L	#8	2.2	12.3	9.5	5	5/8	180

The component parts of each Bar-Lock coupler consist of a steel tube, "lock-shear" bolts, and serrated rails. Figure 3 (4-1) shows a schematic diagram of the coupler design. The seamless, hot-rolled steel tube conforms to ASTM A-519, with a minimum tensile strength in excess of 100 ksi. The lockshear bolt material is AISI 41L40. The bolts are through-hardened over the entire bolt length and further induction-hardened at the conical bolt tip. The serrated rails are made of ASTM CD1018. They are machined and then carburized to a depth of 0.032 in.

An equivalent testing program was performed for each of the two coupler/rebar sizes tested. For each size, forty test specimen assemblies were made up for tensile strength tests, and forty assemblies were made up for the cyclic durability tests. The test specimen assemblies were made up by steel construction workers using Bar-Lock's assembly instructions in a normal field environment. Assembly of the test specimens was monitored by Bechtel QC personnel.

5. TEST RESULTS

All of the 160 individual coupler specimens tested in this program, and all relevant specimen sample set averages and individual coupler strengths, exceeded the requirements set forth in the ASME Code, Section CC-4333.2.3(a).

Eighty tensile strength tests (forty of each size) were performed on coupler assembly specimens according to relevant sections of ASTM A 370 and E 8, and ASME CC-4333.2.3(a). A representative stress-strain curve for a coupler strength test is provided in Figure 4. No practical differences were observed in the general character of the stress-strain curve of any of the 80 specimens tested. All test data collected included stress, strain, crosshead displacement, applied force, and elapsed time. The actual individual test specimen results obtained through standard analysis methods provided in ASTM E 8 are tabulated in Tables 3 and 4. A representative stress-strain plot for a cyclic test is provided in Figure 5.

Table 3. Tensile Properties for #6 Rebar (Heat ID: 5898I2899)

Specimen ID	HOF Yield (ksi)	UTS (ksi)	ε_f (%)	E (Msi)
U6-2	67.7	106.9	14.0	28.7
U6-5	66.8	106.6	13.5	27.4
U6-9	67.0	107.0	12.9	28.1
U6-11	67.6	107.8	14.2	28.6
U6-12	69.9	109.7	10.6	27.3
U6-14	67.9	107.9	12.9	28.3
U6-18	67.3	106.5	14.1	26.2
Averages	67.7	107.5	13.2	27.8

Table 4. Tensile Properties for #6 Rebar Heat ID: 5898I2899

Specimen ID	HOF Yield (ksi)	UTS (ksi)	ε_f (%)	E (Msi)
U8-11	72.5	110.3	12.9	30.1
U8-12	72.4	108.8	11.2	28.7
U8-13	71.7	109.5	12.2	29.3
U8-14	73.0	111.0	9.8	28.8
U8-16	72.8	110.2	11.0	29.1
U8-18	72.5	110.4	11.7	29.2
U8-20	73.0	110.6	11.5	29.1
Averages	72.6	110.1	11.5	29.2

In addition, several specimens of each size were randomly selected to receive an initial slip test prior to the normal strength test. A statistically-legitimate random selection process, using a random number generation algorithm on a computer, was applied to make the selections. Virgin test specimens were installed in the test machine, and instrumented as for a normal strength test. The applied stress was increased from 0, through 3 ksi, up to 30 ksi, and then reduced to 3 ksi. The change in displacement across the coupler between the two 3 ksi stress levels was measured with an extensometer. Figure 5 shows the traces of applied stress and resultant displacement for the six specimens. In all cases, no measurable slip was detected.[1] The observation of no bar slip within the coupler on initial loading means the coupler will develop full strength without excessive deformation upon initial loading.

5.1 Tensile Test Results

The ASME Code, Section CC-4333.2.3, has several criteria with which coupler performance is compared. The two pertinent criteria for the tensile strength test results are:

1. "... The *average tensile strength*[2] of the splices shall not be less than 90% of the actual tensile strength of the reinforcing bar being tested, nor less than 100% of the specified minimum tensile strength."

2. "... The *tensile strength of an individual splice system* (test specimen)[3] shall not be less than 125% of the specified minimum yield strength of the spliced bar."

The coupler assembly performance for both sizes evaluated exceeded both of these criteria. Table 5 tabulates the results of the individual strength tests. Discussion of the comparisons of test results to ASME specified minimum values follow:

5.1.1 Minimum Average Tensile Strength Comparison

For the lots of rebar tested, the "90% of the actual tensile strength" is the governing criteria. For the size #6 group, the specified minimum average strength value is 96.8 ksi. For the size #8 group, the specified minimum average strength value is 99.1 ksi.

5.1.1.1 Coupler/bar size #6

The sample set of strength data from the coupler/bar size #6 was evaluated for normal (Gaussian) probability distribution using the Wilk-Shapiro W-test and graphical analysis methods. The results show a near normal distribution, i.e. only slight departure from normality. Where necessary in the assignment of confidence limits, the assumption of normality is justified.

The size #6 group (sample set, n = 40) average tensile strength is 106.2 ksi (98.8% of the average #6 bar actual tensile strength), with a standard deviation of only 1.87 ksi. The Code-

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[1] the measured slip displacements, equivalent to less than 0.001 in. over the length of the coupler, were much less than observed hysteresis error in the extensometer.

[2] This is a single average value, calculated from the entire group (sample set) of replicate test specimens, i.e. from one heat of material, in one size.

[3] This is the strength value of each individual test specimen (coupler assembly) consisting of one coupler unit and two attached sections of rebar.

required average strength value of 96.8 ksi (90% of actual tensile strength) is 5.0 standard deviations below the sample average. This corresponds to a probability of less than 3 in 10 million couplers would have strength less than the required 96.8 ksi minimum value. Further, a one-sided test for lower bound was also performed. This test provides a practical lower limit strength value for any #6L coupler assembly. Based upon this data set 99% of all couplers of this type will have a tensile strength greater than 100.13 ksi (with a 99% confidence level). This is a very strong indication that the size #6 coupler design will achieve the required minimum strength. These results are confirmed in a letter report (see Appendix F) from INEEL statistician J.J. Einerson. Mr. Einerson reviewed the statistical analyses of the mechanical test data.

5.1.1.2 Coupler/bar size #8

The sample set of strength data from the coupler/bar size #8 was also evaluated for normal (Gaussian) probability distribution using the W-test and graphical analysis methods. Again, results show only slight departure from normality.

The size #8 group (sample set, n = 40) average tensile strength is 109.0 ksi (99.0% of the average #8 bar actual tensile strength), with a standard deviation of only 2.78 ksi. The required average strength value of 99.1 ksi is 3.6 standard deviations below the sample average. This corresponds to a probability of less than 2 in 10,000 couplers would have a strength less than the required 99.1 ksi minimum value. Further, the one-sided test for lower bound (described above) based upon this data set indicates that, with 99% confidence, 99% of all couplers of this type will have a tensile strength greater than 99.94 ksi (see letter report included in the Appendix). This is a very strong indication that the size #8 coupler design will achieve the required minimum strength.

To assess the general capabilities of the overall coupler design, the results from both sizes tested can be normalized by their respective bar lot (mill heat) tensile strengths and combined into one sample set. In so doing, the conclusion is that the Bar-Lock coupler design produces a splice that will achieve an average strength that is 98.9% as strong as the rebar itself. It is obvious that this greatly exceeds the ASME Code-required 90% value. The cumulative standard deviation is 2.2% of the bar strength, making the required minimum strength 4.0 standard deviations below the sample average. The equivalent likelihood is that only 3 in 100,000 would fail to achieve a strength level equivalent to 90% of the bar ultimate strength.

5.1.2 Minimum Tensile Strength of Individual Specimens

This requirement for each individual coupler tested provides additional assurance that the occasional sample tested that may have a relatively low strength value, as compared to the sample set average, at least has an absolute minimum necessary strength for structural considerations. For the Grade 60 rebar used in this study, this required value is 75.0 ksi, and is the same for all specimens tested. All specimens tested in this test program passed this test, and by a very large margin.

5.1.2.1 Binomial (Pass/Fail) Assessment

In the simplest case, the pass/fail criteria can be applied directly. For the combined sample size of 80, with no observed failures (strength below 75.0 ksi), the statement can be made that with 90% confidence, no more than 2.8% of couplers would fail this test. By the nature of this type of binomial probability distribution (pass/fail), it is difficult to state reliabilities with a higher level of confidence without assessing many hundreds of samples. However, by normalizing the measured individual coupler strengths by the required value, an analysis of the amount of deviation on those values can provide a yet stronger comparison and corresponding statement of reliability.

5.1.2.2 Assessment Using Normalized Coupler Strength Distribution

This distribution of normalized strengths shows that the average coupler strength is 144% of the minimum required level for individual couplers, with a standard deviation of less than 4%. So the required strength value is 11 standard deviations below the sample average. The probability tables do not show probabilities below 8 standard deviations from the mean, but at that value, the probability is less than 2x10-15 that the strength of an individual assembly would be lower than the requirement, i.e. practically impossible.

5.1.2.3 Assessment Using Alternative Strength Criterion

A comment by the US Nuclear Regulatory Commission (USNRC), during a presentation on the Bar-Lock couplers on August 9, 2001, was that the minimum strength criterion for individual test specimens should be based upon the actual, measured yield strength of the bar material, rather than the specified minimum value (as done above, per the ASME qualification specification). This makes more sense from a practical view, and it removes one variable (the specified material yield strength) from the comparison. However, this approach does apply a more stringent test of the coupler capability, since the actual yield strength will always be higher than the minimum allowable. To apply this criterion, the size #6 and size #8 specimens must be treated separately since the measured yield strengths of the two bar sizes are significantly different.

Size #6 Couplers

Using the appropriately normalized test results from the #6 test specimens, the same analysis described above was carried out. The size #6 coupler specimen tensile strengths averaged 106.2 ksi, 25.4% above the USNRC-proposed strength level of 84.6 ksi (125% * 67.7 ksi) with a standard deviation of 1.86 ksi. The proposed minimum strength here is still more than 11 standard deviations above the proposed minimum level, with the probability being essentially zero that any coupler would fail to achieve this strength level.

Size #8 Couplers

Analyzing the normalized test results from the #8 test specimens show their tensile strengths averaged 109.0, 20.1% above the USNRC-proposed strength level of 90.8 ksi (125% * 72.6 ksi) with a standard deviation of 2.81 ksi. The proposed minimum strength here is still 6.5 standard deviations above the proposed minimum level. The resultant failure probability is still less than 1x10-10.

5.1.3 Tensile Strength Performance Exceeds Requirements

The overall strength performance of the Bar-Lock coupler design can be summarized as excellent, based on this comprehensive test program of different size couplers. There were no failures to meet any of the specified or proposed strength criteria in any case. As the various failure probability values indicate, the likelihood of any individual Type 6L or 8L coupler assembly failing to achieve the ASME required strength levels is very low.

Table 5. Re-Bar Splice Assemblies Strength Test Results

Specimen ID (#6)	Failure Type[4]	Final Strain (%)	UTS (ksi)	Specimen ID (#8)	Failure Type	Final Strain (%)	UTS (ksi)
Average	--	NA[5]	106.2	Average	--	NA[b]	109.0
S6-01	O	3.8	107.9	S8-01	O	3.7	109.6
S6-02	P	15.2	108.0	S8-02	T	1.4	96.8
S6-03	P	14.4	98.9	S8-03	O	4.9	109.8
S6-04	P	15.2	106.4	S8-04	O	3.7	110.1
S6-05	O	4.9	107.3	S8-05	P	10.4	108.4
S6-06	O	4.1	107.8	S8-06	T	4.9	109.7
S6-07	O	4.2	107.6	S8-07	T	4.4	110.4
S6-08	P	13.1	106.9	S8-08	T	3.6	109.4
S6-09	T	2.7	103.2	S8-09	O	3.6	110.5
S6-10	O	4.6	107.6	S8-10	T	1.8	102.1
S6-11	P	13.0	107.3	S8-11	T	2.1	106.0
S6-12	O	4.4	105.6	S8-12	*	3.8	108.0
S6-13	T	2.7	103.4	S8-13	O	3.4	110.5
S6-14	P	10.8	105.8	S8-14	T	3.2	110.1
S6-15	P	12.3	104.0	S8-15	*	3.7	106.7
S6-16	O	3.8	108.0	S8-16	T	4.0	111.0
S6-17	P	9.8	103.7	S8-17	T	2.1	104.5
S6-18	P	11.5	106.3	S8-18	T	4.5	109.3
S6-19	P	19.1	106.1	S8-19	T	4.0	109.4
S6-20	P	15.4	107.6	S8-20	O	4.6	110.1
S6-21	P	11.0	106.0	S8-21	T	3.5	109.7
S6-22	P	11.6	105.0	S8-22	T	4.3	109.4
S6-23	T	2.7	103.1	S8-23	T	3.8	109.8
S6-24	O	4.1	107.8	S8-24	T	3.3	108.5

[4] B = bar break outside coupler but within extensometer gage length, O = bar break outside coupler and outside extensometer gage length, T = bar break at tip of first lock bolt, P = bar pulled out of coupler without breaking, * = bar break in interior of coupler

[5] The final strain is dependent on several factors, including mode of failure An average value for all tests has no significance. For example, in a pull-out failure the final strain is determined by the length of time the operator chooses to continue the test once pull-out is observed.

Specimen ID (#6)	Failure Type[4]	Final Strain (%)	UTS (ksi)	Specimen ID (#8)	Failure Type	Final Strain (%)	UTS (ksi)
Average	--	NA[5]	106.2	Average	--	NA[b]	109.0
S6-25	P	11.5	105.1	S8-25	P	10.4	110.0
S6-26	P	11.3	107.9	S8-26	T	4.2	109.9
S6-27	P	12.2	106.4	S8-27	*P	7.0	109.7
S6-28	O	3.9	107.8	S8-28	T	4.1	109.0
S6-29	B	4.8	107.0	S8-29	O	3.8	109.7
S6-30	O	4.3	107.6	S8-30	O	3.5	110.3
S6-31	O	4.4	107.4	S8-31	T	3.9	110.5
S6-32	T	3.8	107.2	S8-32	T	2.5	109.0
S6-33	T	2.9	105.7	S8-33	O	4.4	110.3
S6-34	P	12.6	105.7	S8-34	T	3.5	109.7
S6-35	T	4.4	107.2	S8-35	T	2.5	105.4
S6-36	T	2.8	104.2	S8-36	T	4.1	110.5
S6-37	O	3.8	107.2	S8-37	*	5.0	110.2
S6-38	P	11.5	107.4	S8-38	P	10.3	109.9
S6-39	P	12.9	107.0	S8-39	T	3.9	111.2
S6-40	P	11.3	106.3	S8-40	P	10.2	113.6

5.2 Cyclic Test Results

Coupler assemblies were cyclically tested according to the requirements of ASME CC-4333.2.3(b). Forty specimens of each of the two types (6L and 8L) received 100 load cycles between 5 and 90% of specified minimum bar yield strength (60 ksi). None of the specimens failed in any manner, e.g. bar break, or bar slip within the coupler.

Applied stress and specimen extension data were digitized during the cyclic tests to provided additional insight into the coupler performance under cyclic load conditions. Figure 6 shows a representative plot of stress versus displacement. For clarity, only every tenth cycle is presented. It shows the accumulated slip over 100 cycles to be less than 0.0015 in. This is less than 10% of the elastic deformation that occurs during a single load cycle. The same behavior was observed in all of the tests of both coupler sizes. The couplers showed no significant deterioration (visible, or evidenced by deviation is test data) during the tests.

Based on the binomial probability function (pass/fail testing), and no observed failures in 80 tests, it can be stated with 90% confidence that less than 2.8% of all couplers would fail prior to the completion of 100 loading cycles.

5.2.1 Higher Count Cyclic Tests

In an effort to improve the cyclic durability performance assessment, several of the specimens in each size were selected at random to receive additional cyclic loading. Each selected specimen was subjected to an additional 1000 cycles. None of the specimens failed, and none of them showed signs of deterioration through excessive strain accumulation or physical deformation. While this does not provide a verifiable improvement in the statistical probability of failure (the confidence level is too low to be useful), it does provide an engineering indication that the cyclic durability of the couplers will far exceed 100 cycles.

5.2.2 Residual Strength Tests

Another test was also performed on randomly selected couplers to provide additional information regarding cyclic durability and residual strength. The selected couplers, all having been subjected to 100 loading cycles, were subsequently loaded to failure monotonically. This is the standard "tensile strength test" described in the previous section. The concept here is to determine if the prescribed cyclic loading substantially damages the integrity of the splice assembly. The eight specimens tested all achieved the same nominal strength as the corresponding specimens receiving no cyclic loading. Table 6 summarizes these test results. These observations suggest that cyclic loading in the stress range from 3 to 54 ksi does very little, if anything, to reduce the strength capacity of a spliced joint made using the Bar-Lock L-series coupler.

Table 6. Results of Residual Strength Tests on Load-Cycled Specimen Assemblies

Specimen ID (#6)	Failure Type	Final Strain (%)	UTS (ksi)	Specimen ID (#8)	Failure Type	Final Strain (%)	UTS (ksi)
Average	--	NA	104.9	Average	--	NA	106.7
C6-2	P	3.8	104.3	C8-15			106.6
C6-3	P	3.7	106.3	C8-21			106.0
C6-7	P	5.0	106.2	C8-27			107.6
C6-14	P	7.0	103.3				
C6-15	P	3.7	104.5				

6. COUPLER TEST PROGRAM CONCLUSIONS

The Bar-Lock coupler qualification testing program was carried out on two representative sizes – #6 and #8 – of their L-Series couplers. One hundred-sixty (160) coupler assemblies were tested. Fourteen (14) pieces of plain rebar were tested to determine the actual, or measured, mechanical properties of the two heats of bar material used in the test specimens.

6.1 Tensile Strength

The tensile strength tests on 80 samples from each of the two sizes all exceeded the two ASME requirements by a large margin. Statistical analyses of the test results determined several important performance indicators, all of which suggested that any given coupler assembly would far exceed the

ASME-specified strength requirements. *The overall probability of any coupler assembly (in size #6 or #8) failing to meet the minimum qualification strength criterion is less than* **3 in 100,000.**

There was some variation in strength between the two heats of rebar used in the strength tests. Comparing and correlating these results show that Bar -Lock L-Series coupler splices can be expected to achieve a tensile strength greater than 96% of the actua l strength of the bar material that is connected using the coupler device. While there are not enough different combinations of bar material and coupler size data to make this statement with high probabilistic certainty, the combined test results from thi s program appear similar when normalized by the actual bar strength. Therefore, it is likely these test results are representative of the performance of other sizes of Bar-Lock L-Series couplers. In other words, the mechanical design of the Bar-Lock L-Series coupler is such that spliced joints can be expected to develop over 96% of the actual bar strength.

6.2 Mechanical Slippage in the Couplers

Slip tests performed on selected specimens of both sizes showed a solid mechanical connection between the coupler and the rebar. There was no tendency for the rebar to move within the coupler prior to developing full splice strength. This was expected since the conical-tipped lock bolts physically embed into the bar material providing a physical shear force transfer from bar to coupler.

6.3 Cyclic Loading Durability

All 80 splice specimens that underwent the cyclic loading durability test passed the 100-cycle test, with no obvious physical degradation of the spliced joint. To provide an additional degree of assurance of adequate cyclic durability, selected specimens received 1000 cycles of loading, again with no noticeable physical degradation. Some of the specimens that passed the 100 cycle test were subsequently tested by monotonic loading to failure. The resultant me asured strengths were essentially the same as the virgin strength test specimens (no cyclic loading applied). These results suggest that the design of the Bar-Lock coupler is essentially insensitive to cyclic loading to levels below 90% of the minimum bar yield strength.

6.4 Overall Coupler Performance

All of these test results, compared to the ASME splice system qualification requirements, indicate that the Bar-Lock coupler design for rebar splicing is entirely adequate from a strength point of view for use in nuclear safety-related construction. The large quantity of couplers tested provides higher confidence that the couplers do meet, and indeed far exceed, those ASME-specified requirement.

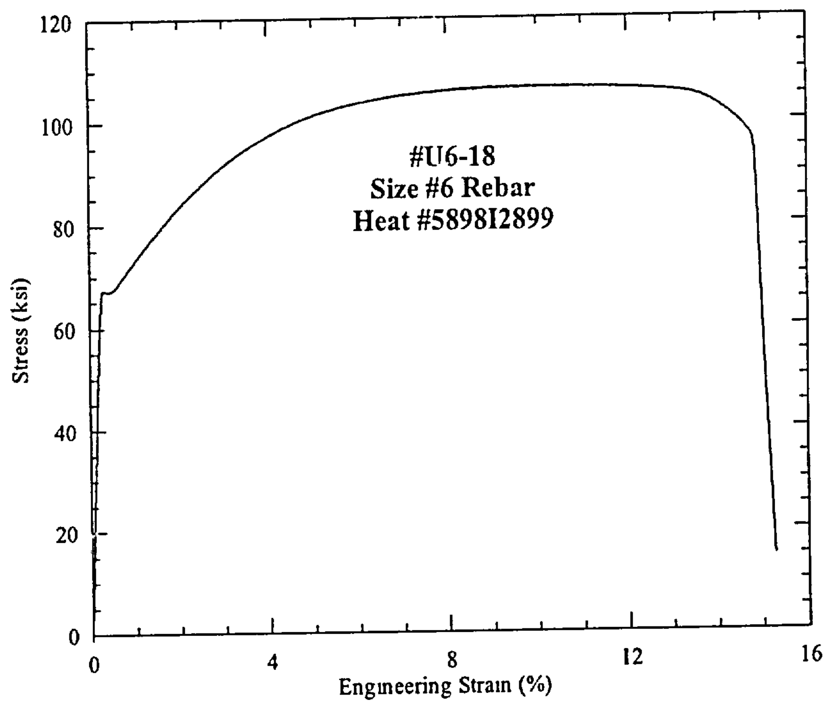

Figure 1. Representative Stress-Strain Curve from #6 Rebar Material

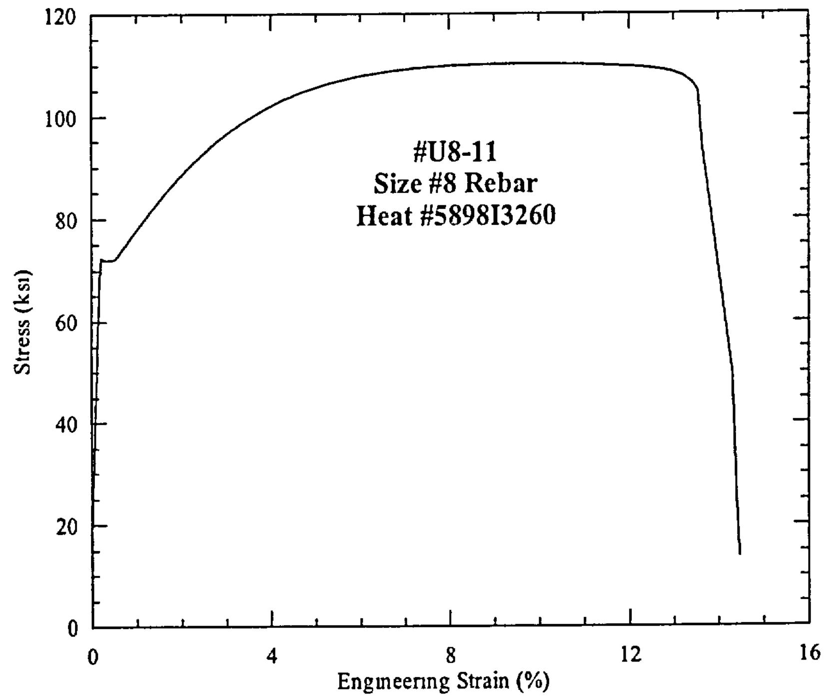

Figure 2. Representative Stress-Strain Curve from #8 Rebar Material

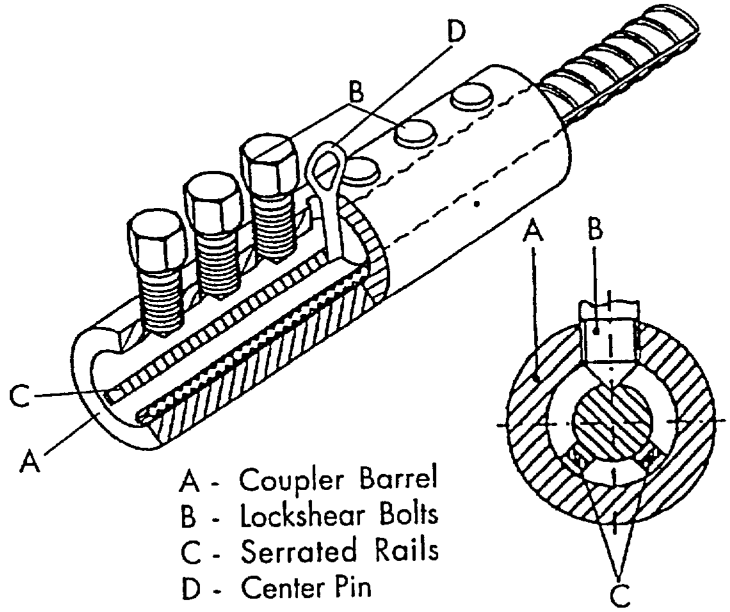

Figure 3. Bar-Lock Coupler Cutaway View Showing Internal Details

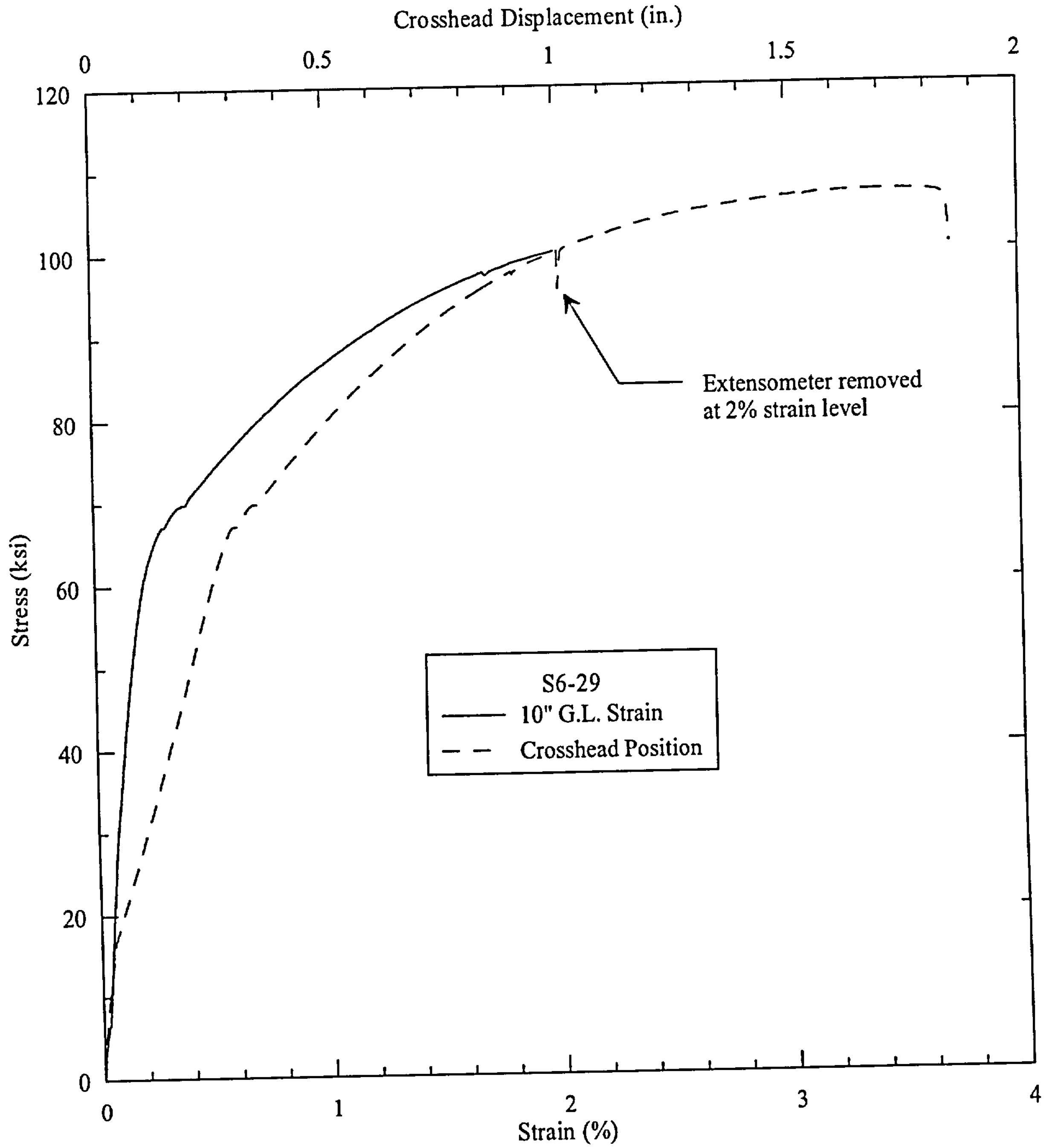

Figure 4. Representative Test Data from a Coupler Assembly Strength Test

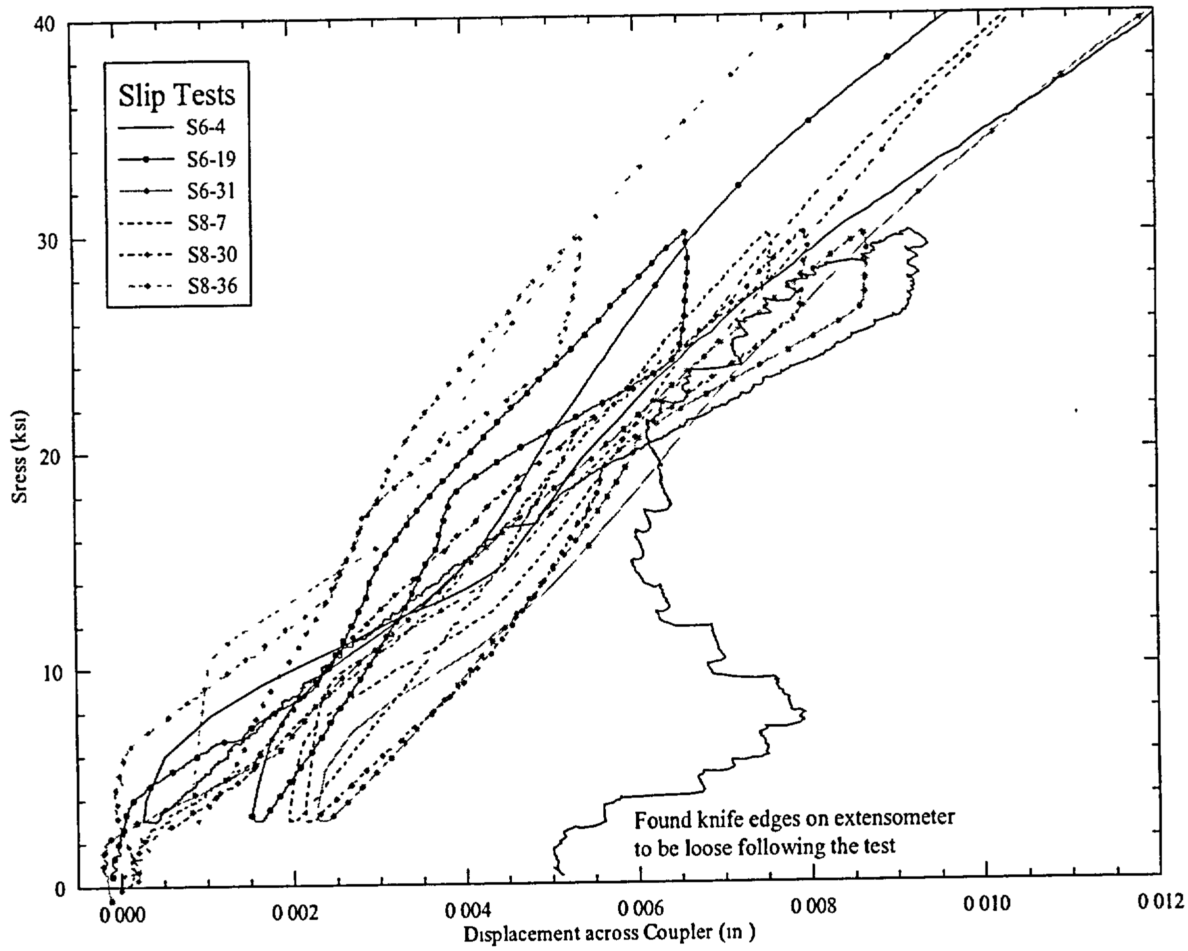

Figure 5. Data Curves Showing Load-Unload Cycle to Assess Bar Slip in Couplers

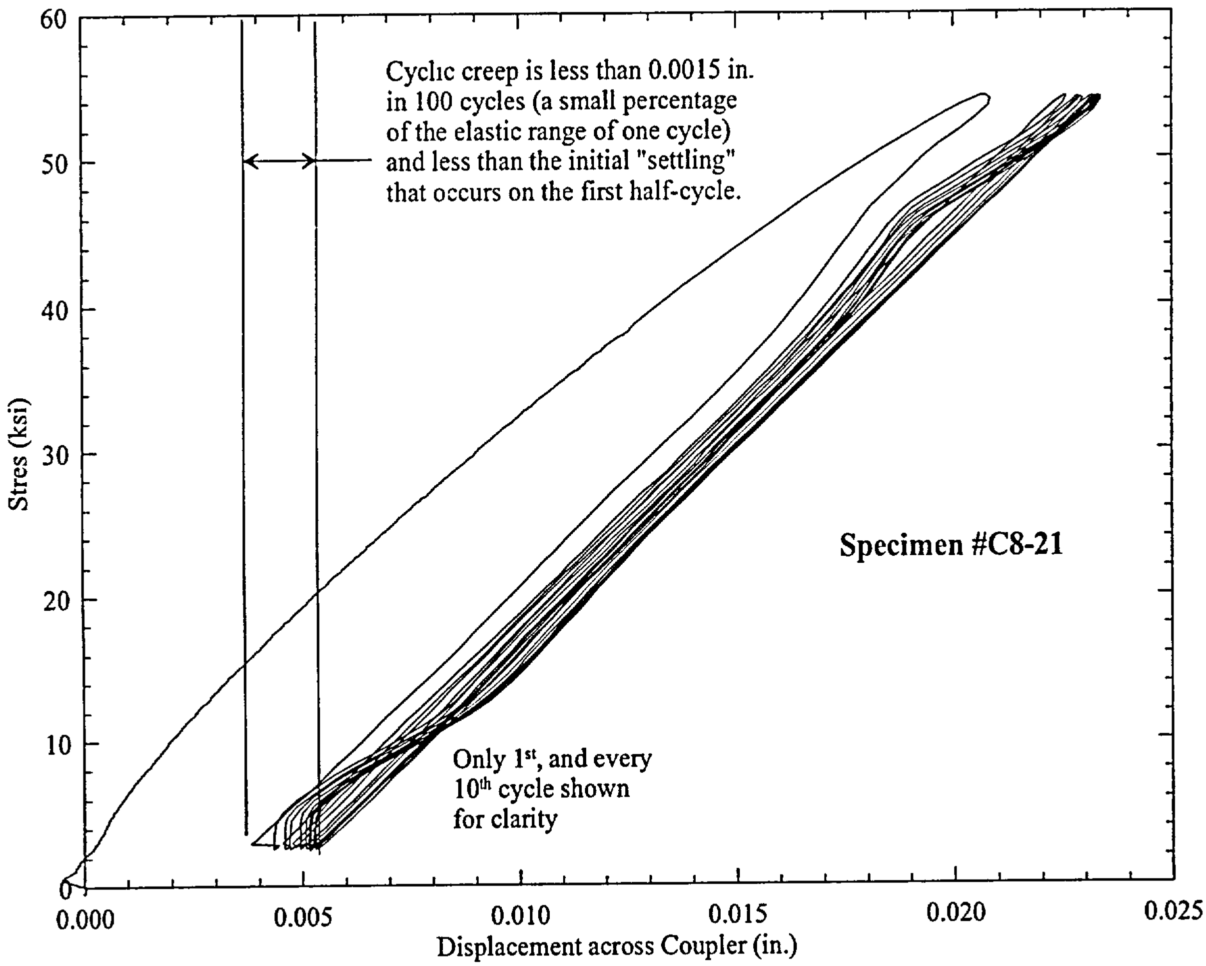

Figure 6. Cyclic Stress-Displacement History for a Typical Test