ATTACHMENT 1

OUTLINE FOR COMPENSATORY MEASURES HEAVY LOAD DROP PROTECTION

DISCUSSION

Due to physical proximity, several piping segments are in jeopardy of being broken or crimped by a postulated heavy load drop from the outside lift system (OLS). 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 refueling water storage tank (RWST) , and their associated piping. A postulated pipe break of the ERCW system could occur in the yard (i.e., underground) or in a pipe tunnel. A postulated rupture of the RWST and PWST could occur such that the water also goes into the pipe tunnel or onto the ground in the yard. Additionally, a fire protection supply line is present in the tunnel. Of these, the lines that have potential consequences for safe operation of Unit 2 are the ERCW supply and return headers. Since the consequences of ERCW pipe ruptures or crimping are potentially serious for Unit 2, compensatory measures will be written to re-direct required ERCW system flow and ensure safe shutdown of Unit 2. The compensatory measures will be implemented using an abnormal operating procedure (AOP).

The zone where the potential for damage to the ERCW supply and return headers exists is the load path between the reactor building openings and the upending/downending area for the SGs starting at ~7 feet from the inside of the parapet wall until the upending/downending area is reached. For the shield building concrete sections, the steel containment vessel and steam generator compartment roof concrete sections, the zone is between the reactor building openings starting at ~7 feet from the inside of the parapet wall and until the load is 3 ft above grade. Refer to Figure 5-2 in Topical Report 24370-TR-C-002 for details on the load paths. If a load drop occurs in this zone, preparations will immediately commence for an orderly shutdown of Unit 2.

Analysis of this postulated event is documented in TVA calculation MDQ00006720000095. This analysis has shown that all direct Unit 2 safe shutdown systems will receive the design ERCW flow rates and will be able to continue to perform their function indefinitely with no operator action. Those functions that have high impact on core damage frequency such as reactor coolant pump (RCP) seal injection and seal cooling will continue to function as designed.

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Certain components that receive flow from the 1A and 1B ERCW supply headers are indirect Unit 2 safe shutdown components, and may not receive their design flow rates. The following listing is the indirect Unit 2 safe shutdown components that might not receive their design flow rates following a postulated heavy load drop:

- Emergency diesel generators
- 'A' auxiliary air compressor
- Main control room chillers
- Component cooling system pump space coolers
- Electrical board room chillers
- 'A' 6.9-kV shutdown board room chiller
- Cooling to auxiliary building gas treatment system.

The emergency diesel generators will be aligned to their alternate source from the control room immediately (i.e., 10 minutes) upon reported heavy load drop in the critical zone. For the other components above, the limiting response time to restore the design functions is a minimum of 18 hours. The 18 hours is based on the time it takes for the control room to reach its upper temperature limit. The required actions following a load drop to align the alternate water sources to these components can easily be accomplished within the timeframe available.

In order to obtain the 18-hour timeframe, a number of plant alignment changes are being performed as prerequisites to the heavy load lifts. However, caution is being used such that no prerequisite would adversely impact items required to be OPERABLE by the technical specifications. The prerequisite 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.

A temporary wall will be erected at the entrance to the pipe tunnel from auxiliary building to contain the water. The temporary wall will be 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.

In recognition of the plant consequences of a postulated load drop event, Sequoyah plant staff will be examining closely other scheduled maintenance tasks as well as any failures that may arise, per the requirements of 10 CFR 50.65(a)(4).

The actions contained in this compensatory measure will ensure that Unit 2 can be safely brought to cold shutdown and maintained in cold shutdown in the unlikely event of a heavy load drop damaging the ERCW piping as outlined in Topical Report 24370-TR-C-002.

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ACTIONS PRIOR TO COMMENCEMENT OF OLS HEAVY LOAD LIFT

Specific actions required to be completed prior to commencement of an OLS heavy load lift that passes near or over ERCW supply and/or discharge headers.

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. Place temporary instruments at:

   Differential pressure gauge at 1A and 1B component cooling system (CCS) pump space cooler. If the differential pressure gauge reads less than 70 inches, the coolers are inoperable.

   Pressure gauge at A and B main control room chiller drain valve. The chiller operation is in doubt if the A-train gauge reads less than 20 psig or the B-train pressure gauge reads less than 30 psig.

6. Close 1&2-FCV-67-22 & 24, ERCW cross-tie valves.

7. Throttle full open 0-67-552, 0-67-551, 0-FCV-67-152, and 0-FCV-67-151, and place clearance on the power to 0-FCV-67-152.

8. Station Operations personnel to visually monitor the crane activities.

9. Station Operations personnel to be available to go to 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).

10. Station Operations personnel to be available to monitor temporary gauges in the auxiliary building and to observe for leakage through the temporary wall.

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11. Immediately prior to any heavy load lift, mark the ERCW header pressures and flows on the main control room indicators. Also, mark the Unit 1 RWST level and the PWST level on the main control room indicators.

12. Ensure that no air filters, herculite, etc., is covering the grating over the passive sump or the handrails around the opening.

13. Isolate the high pressure fire protection (HPFP)/flood mode pump pipe by closing valves 1-26-575 and 1-26-653.

14. Throttle supply header manual isolation valves 1-67-727A and 1-67-727B to pre-determined position

15. It is preferred that the 'B' shutdown board room (SDBR) chiller be running. This is a preferred action.

16. Connect hose from turbine building space cooler to ERCW at station air compressors (SAC). Use check valve at ERCW pipe, valve line in ahead of time.

17. Get the flood mode spool piece ready to install. Preference is to install on Train A. Set up valve alignment in AOP such that 'A' SDBR and 'A' auxiliary control air (ACA) can be fed. Final determination of which train to connect to depends on damage sustained.

18. Throttle valves 0-67-1500 & 1501 & 546C, and revise throttling of existing throttled valves 0-67-1506 and 0-67-1509. This is a preferred action to reduce the potential for cavitation. There is no actual effect on the compensatory measures if this action is not performed.

19. Have two running ERCW pumps per train.

20. Isolate ERCW, CCS, and spent fuel pool (SFP) from the Unit 1 reactor Building.

<u>ACTIONS FOLLOWING A LOAD DROP</u>

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

Immediate actions:

- Any load drop in the zone where the potential for piping damage exists, commence an orderly shutdown of Unit 2 in accordance with technical specifications.

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- The four alternate supply valves for the standby emergency diesel generator will be opened immediately to preclude damage to the diesel generators were a start signal generated due to some reason unrelated to the load drop. These valves are opened from the control room. These valves are being opened as a precautionary measure should a diesel generator be started.

- The other immediate operator action is to ensure that there are at least two running ERCW pumps per train. The operator verification consists of observing the pump handswitches. If the required number of pumps is not running then these will be started from the control room handswitch.

The control room actions will be completed within 10 minutes of control room notification of a heavy load drop.

Other actions

- Rupture or crushing of the 1A and/or 1B ERCW supply header

  If flows to the equipment on the 1A and/or 1B ERCW supply header drop to less than that needed to support equipment operation, alternate water supplies will be used for that equipment. The time required to place the alternate water supply in service is less than the time available to restore this equipment to service. If the alternate supplies are used, then the leak will be isolated by available valves to increase margins. Isolation is not required to support any safety function.

  If the incident involves a loss of function of the main control room chiller or the space cooler for the component cooling pumps, then alternate cooling water supplies will be used. Criteria in terms of actual flow rates and pressures to monitor for indications of damage has been determined and placed in the abnormal operating procedure that will be used during this event. The instruments used are not in the control room, but are easily accessible and can be quickly read and compared with the pre-established criteria. In the event of a heavy load drop, these instruments will be monitored and the readings communicated to the control room. If the readings indicate that the function of equipment important to Unit 2 safe shutdown is in jeopardy, then actions will be taken to ensure full functionality of the equipment.

  In order to supply cooling water to the components normally fed from the 1A ERCW supply header, the following actions will be performed in sequence:

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- Evaluate whether to supply the equipment from the B-train ERCW or from the raw cooling water system. The evaluation criteria consists of determining if there is detectable damage to the B-train ERCW. If there is no damage to Train B ERCW components then raw cooling water is recommended to be used. Time required to reach decision point will be less than one hour from the load drop.

- In order to supply raw cooling water to the ERCW headers, a flood mode spool piece is installed in the piping between the two systems. When the spool piece is installed, the manual valves on either side of the spool piece are opened. The time required to install the spool piece and open the required valves is less than four hours.

- In order to supply the 1A ERCW header from the 2B ERCW header, valve 1-FCV-67-81 is closed, then 1-FCV-67-147 is opened. These valves are motor operated valves manipulated either locally or from the valve breaker. The time required to manipulate these valves is less than one hour.

In order to supply cooling water to the components normally fed from the 1B ERCW supply header, the following actions will be performed in sequence:

- Evaluate whether to supply the equipment from the A-train ERCW or from the raw cooling water system. The evaluation criteria consists of determining if there is detectable damage to the A-train ERCW. If there is no damage to Train A ERCW components then raw cooling water is recommended to be used. Time required to reach decision point will be less than one hour from the load drop.

- In order to supply raw cooling water to the ERCW headers, a flood mode spool piece is installed in the piping between the two systems. When the spool piece is installed, the manual valves on either side of the spool piece are opened. The time required to install the spool piece and valve it in will be less than four hours.

- In order to supply the 1B ERCW header from the 2A ERCW header, valve 1-FCV-67-82 is closed, then 1-FCV-67-424 is opened. These valves are motor-operated valves manipulated either locally or from the valve breaker. The time required to manipulate these valves is less than one hour.

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The flood mode spool piece described above for use is an existing component used to tie the ERCW system and the raw cooling water system together in various plant conditions including loss of downstream dam and floods above plant grade, as described in UFSAR Sections 2.4 and 9.2.2. Existing plant procedures will be used to install the spool piece. For the location of the flood mode spool piece connections on the ERCW system refer to Figure 6-1 in Topical Report 24370-TR-C-002.

Of the plant equipment that might not receive adequate cooling water flow until required actions are complete the most limiting component is the control room chillers. The timeframe available to restore cooling water to the control room chillers is greater than 18 hours until the control room temperatures will reach the value at which equipment operation is affected (Reference TVA Calculation MDQ00003120020119). The time required to complete the required actions is substantially less than the time available to complete them.

- Rupture of the A ERCW discharge header

  No immediate actions are necessary, monitor conditions at the temporary wall in the pipe tunnel.

- Crimping of the A ERCW discharge header

  No specific actions are required, monitor conditions at the temporary wall in the pipe tunnel.

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ATTACHMENT 2

DENSIFICATION OF LOOSE DEPOSITS BY POUNDING

(JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION)

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JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Densification of Loose Deposits by Pounding

By Robert G. Lukas,[1] M. ASCE

Introduction

Certain types of marginal sites can be improved to the point where one-stoi to four-story structures can be supported by conventional spread footing foundations. The improvement consists of densifying loose soil or fill deposits by means of pounding. The pounding process used to date has consisted of dropping a weight of 2 tons–6 tons (1.8 metric tons–5.4 metric tons) from heights of 30 ft–35 ft (9.2 m–10.7 m) to impact into the soil thereby causing densification to depths ranging from 10 ft–20 ft (3.1 m–6.1 m).

This process has been successfully used on eight project sites under the writer's supervision. At five of the sites, the subsurface conditions consisted of building rubble and miscellaneous fill overlying a medium strength natural clay or clayey silt deposit. A natural loose fine sand was present at two sites and one project consisted of a shopping center constructed on a former garbage dump. This paper examines the ground subsidence observed during the pounding process, the degree of densification achieved with depth below grade, ground vibrations associated with pounding and the performance of the structures which were supported upon the densified soils.

Description of Projects

The pertinent features of each of the eight projects are summarized in Tab. 1. Typically, the pounding was used to densify the upper loose deposit thereby enabling the structural loads to be supported at grade. In the case of buildings, spread footings were normally supported within the densified deposit utilizing a bearing pressure of 3,000 psf (144 kPa). In Projects 1 and 3, the site densification was undertaken to minimize future maintenance problems associated with constructing a pavement or slab-on-grade over a loose deposit.

Note.—Discussion open until September 1, 1980. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 106, No. GT4, April, 1980. Manuscript was submitted for review for possible publication on July 17, 1979.

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The earliest densification projects were undertaken with whatever weight was available and in most cases this consisted of a 2-ton (1.8-metric tons) wrecking

TABLE 1.—Description of Projects

Project number (1)	Project description (2)	Date (3)	Substrate densified (4)	Weight, in tons (5)	Drop, in feet (6)	Average ground depressions, in inches (7)	Coverage (8)
1	Freight terminal truck parking area	1971	10 ft of rubble with large voids	2	30	Tight areas—6 Loose areas—12	Grid at 3 ft center to center
2	Four-Story building	1975	9 ft of cinders and sand over 3 ft of rubble	2	25	3–9	Entire footing areas
3	Floor slab loaded to 2,300 pounds per square foot	1976	13 ft of loose fine sand	4.8	12	4–6	Grid at 10 ft center to center
4	Two-Story shopping center	1976–1978	Upper 15 ft–20 ft of 60 ft of miscellaneous fill	6	35	Typical—12 to 18 Occasional—18–36	Grid at 7 ft Spacing plus footings
5	Two-Story parking garage	1977	15 ft of building rubble	3.4	25	12–15	Grid at 7 ft center to center
6	Refinery racks	1978	Loose fine sand from 5 ft–13.5 ft	6	30	Typical—12 Occasional—18	Grid at 6 ft center to center
7	Two-Story parking garage	1978	7 ft of rubble fill over 6 ft of sandy silt	4	25	Typical—12 Occasional—32–47	Grid at 5 ft center to center
8	One-Story truck terminal building	1979	5 ft–16 ft of miscellaneous fill over saturated clayey silt	6	40	Typical—12–15 Occasional—24–36	Grid at 7 ft plus footings

Note: Sites 1, 2, 5, 7, and 8 underlain by medium strength clay; and sites 3 and 6 underlain by medium dense to dense sand. 1 ft = 0.305 m; 1 ton = 907 kg; 1 in. = 25.4 mm.

ball. This weight worked reasonably well on building rubble formations but the rounded shape was not suited for densification of other fill or natural soil deposits. The densification conducted after 1976 was generally done with a heavier weight such as a 3.5 ton–6 ton (3.2 metric tons–5.4 metric tons) size which has a flat bottom.

The amount of ground depression as a result of pounding is listed in Col. 7 of Table 1. This value represents an average range throughout the project and the individual craters were many times this amount. The depth of ground displacement by itself does not indicate the degree of improvement being attained but it does serve as a practical field guide.

The coverage applied to each site is shown in Col. 8 of Table 1. At some projects, the primary concern was improvement only at concentrated load points so pounding was undertaken at the footings. The weight was dropped on a grid basis throughout the footing area plus a short distance beyond the edges. On other projects, there was concern that the floor slab as well as the footings could settle so the entire building area plus a short distance beyond the edges was pounded on a grid basis with the distance between impact points being about 4 ft–10 ft (1.2 m–3.1 m). At two projects, the grid pounding was followed by pounding at individual footing locations thereby effecting double coverage at these locations. At each pounding location, the weight was dropped seven to nine times.

Improvement versus Depth

The primary purpose for using the pounding procedure is to achieve a significant densification at depths greater than can normally be achieved by compaction equipment or heavily-loaded proofrolling devices. Menard and Broise (1) have proposed the following formula as the first-step indicator of the required energy to achieve densification to a predetermined depth:

$$W \times H = D^2 \qquad (1)$$

in which W = weight, in metric tons; H = height of drop, in meters; and D = depth of improvement, in meters.

To investigate the amount of densification, borings were made at the project sites before and after the pounding process had been undertaken. These borings included Standard Penetration Resistance tests or pressuremeter tests, or both. The Standard Penetration Tests were generally used in the relatively uniform deposits such as natural sandy soils and the pressuremeter tests in the nonhomogeneous deposits such as miscellaneous fill materials. At four of the project sites, sufficient borings and tests were made to measure the degree of improvement as a function of depth. The results of these tests performed before and after pounding are shown on Figs. 1–3.

For the hammers in the range of 3.5 tons–6 tons (3.2 metric tons–5.4 metric tons) falling through a distance of 25 ft–40 ft (7.6 m–12.2 m), the improvement in soil properties was found to extend to levels on the order of 15 ft–20 ft (4.6 m–6.1 m) below grade. At and below this level, either the Standard Penetration Resistance Value or the limit pressure was found to be approximately the same after densification as it was initially. A comparison of the depth of improvement by Eq. 1 to the depth of improvement as measured from Standard Penetration or pressuremeter tests is presented in Table 2. These data indicate that the depth to which improvement occurs is only on the order of 65%–80% of the depth predicted by Eq. 1. The improvement of the soil properties was not

uniform throughout and was greater at the upper levels diminishing to slight improvements at the deeper level.

One of the most beneficial effects of pounding is to collapse voids or to

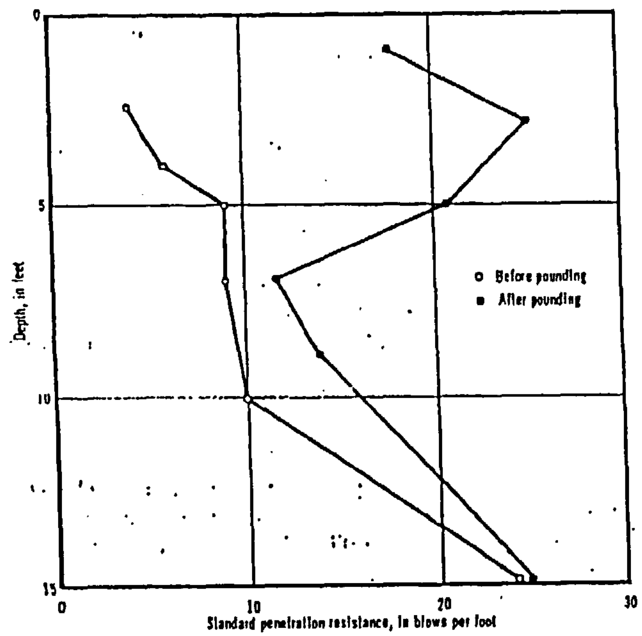

FIG. 1.—Standard Penetration Resistance Versus Depth: Site 3 (1 ft = 0.305 m)

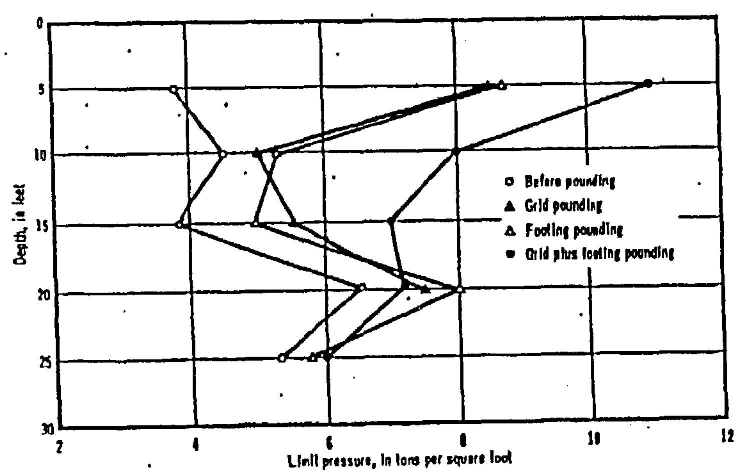

FIG. 2.—Limit Pressure versus Depth: Site 4 (1 ft = 0.305 m; 1 ton/sq ft = 95.8 kPa)

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densify very loose layers. This is shown in Fig. 3(*a*). Within the depth range of 8 ft–14 ft (2.4 m–4.3 m) below ground surface, the initial investigation indicated an extremely loose deposit of sand with the Standard Penetration Resistance Value on the order of one blow per foot. After site densification, the Standard Penetration Resistance Value near the center and edge of this deposit increased

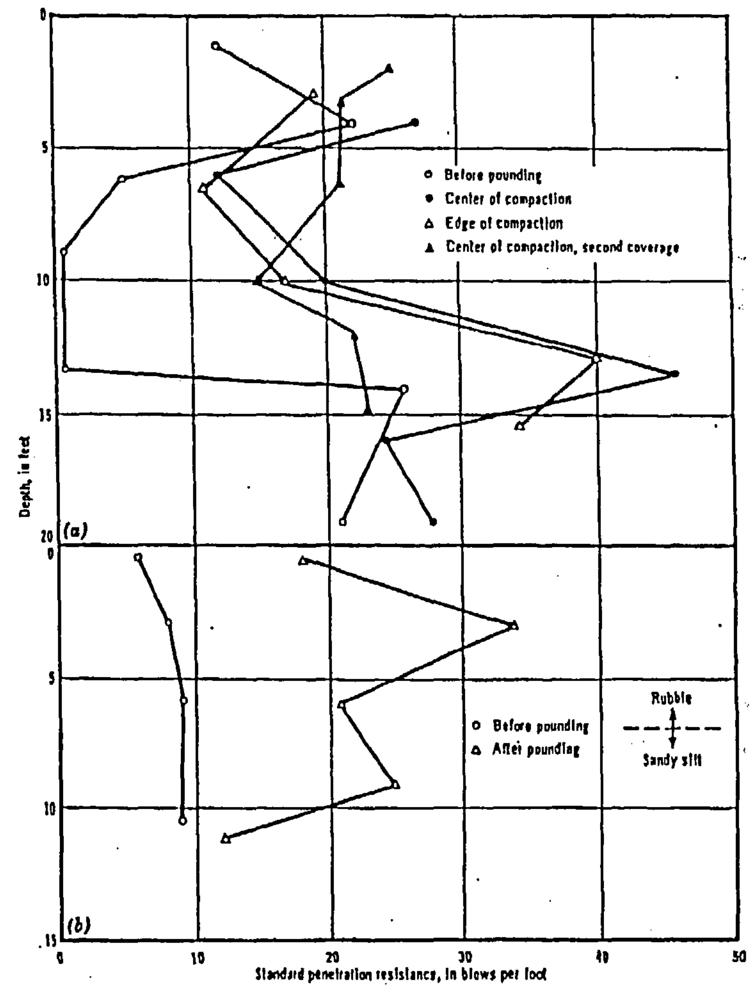

FIG. 3.—Standard Penetration Resistance Versus Depth: (*a*) Site 6; (*b*) Site 7 (1 ft = 0.305 m)

to 15 blows per ft–20 blows per ft. Within the upper portion of this deposit where the Standard Penetration Resistance Value was initially on the order of 22 blows per foot, the Standard Penetration Resistance Values after pounding were still only on the order of 20 blows per ft–27 blows per ft.

This data also indicates that the depth of improvement does not appear to

increase with additional coverages. At Site 6, which is represented by Fig. 3(*a*), the depth of improvement was approx 15 ft (4.6 m) for single and double coverage. A similar phenomena occurred at Site 4 as represented by Fig. 2. In both areas the additional coverage improved the degree of compaction achieved in the upper levels.

Densification Mechanism

At seven project sites, the densification was undertaken in materials that were relatively free draining and not fully saturated. Densification of the deposit was due to compaction wherein the air within the void spaces was compressed

TABLE 2.—Improvement with Depth

Site (1)	Depth of improvement, in feet: Predicted from Eq. 1 (2)	Depth of improvement, in feet: Measured (3)	Ratio: measured/predicted (4)
3	13	10	0.77
4	25	20	0.80
6	24	16	0.67
7	17	11	0.65

Note: 1 ft = 0.305 m.

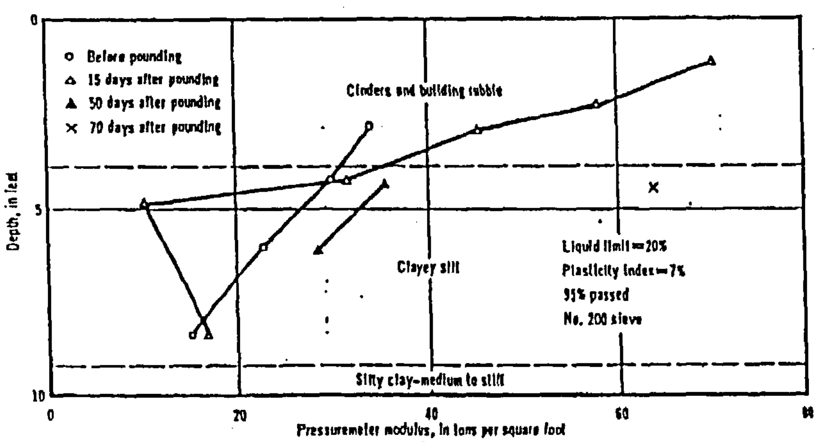

FIG. 4.—Pressuremeter Tests in Clayey Silt: Site 8 (1 ft = 0.305 m; 1 ton/sq ft = 95.8 kPa)

or expelled and large voids were collapsed. The improvement was immediate and the process could be described as dynamic compaction.

At two projects, double coverage was applied with a 6-ton (5.4 metric tons) weight dropping 35 ft–40 ft (10.7 m–12.2 m) with 9 tamps per impact point.

The data obtained from this study indicate that the depth of soil affected by pounding was approx 75% of the value computed from Eq. 1. The energy applied was assumed to be a maximum at ground surface and decrease hyperbolically to zero at this depth. Dividing the total energy applied at ground surface by the volume of soil computed as described, the unit energy applied at ground surface computes to 12,343 ft-lb/cu ft (60,311 kg-m/m^3) which is almost identical to Standard Proctor ASTM D-698 energy.

At Site 8, densification was required in areas where the saturated clayey silt soils were present below 5 ft (1.5 m) of rubble fill. Ground depressions occurred after impact but the ground surface behaved in a spongy manner and the soils liquefied. Shortly after pounding, water was observed to partially fill the craters. Measurements indicated that the ground water rose to a level 3-ft higher than normal and a period of 6 weeks elapsed before the water returned to the original position.

The liquefaction that occurred at Site 8 within the saturated clayey silt, is illustrated in Fig. 4. The pressuremeter modulus 15 days after pounding was lower than tests performed before pounding. After 50 days, the pressuremeter modulus improved to about 25% higher than the initial value. After 70 days, the modulus at the surface of the clayey silt was about double the original value. Samples taken 50 days after pounding indicated that the average water content of the silt dropped from 22%–19%. At Site 8, densification of the clayey silt deposit appears to be due to consolidation following liquefaction.

Ground Induced Vibrations

During the pounding process, a considerable amount of the energy is transmitted into the ground directly below the point of impact to densify the soil. However, some of the energy is transmitted through the ground to locations off the site. One of the concerns with regard to the pounding process is whether any damage could occur to buildings or utilities located beyond the edges of the site being densified.

At Sites 2, 4, and 5, the densification process took place in a relatively congested area adjacent to occupied structures. At Site 2, densification took place immediately adjacent to a one-story auto repair building and within 40 ft (12.2 m) of a 20-story high rise structure. The ground vibrations could be felt in both of these buildings but they were not of significant magnitude to cause damage even though the auto structure was a 50-yr old building. At Site 4, densification took place 20 ft (6.1 m) behind the retaining wall. The wall was measured to laterally deflect 1/8 in.–1/4 in. (3.2 mm–6.3 mm) at the top, but rebounded after each impact. In a restaurant building located about 75 ft–100 ft (22.8 m–30.5 m) away, the chandeliers were observed to swing for a period of about 5 sec–10 sec after impact, but no other adverse conditions were observed.

Site 5 is located in a downtown business district area adjacent to a 40-story high rise and across a city street from a three-story old railroad terminal building. For this project, the particle velocity was measured with two Sprengnether seismograph units. One unit was stationed on the sidewalk at the property line at a point 30-ft (9.2-m) distant from the point of impact and the second unit was moved to different locations around the site and included readings taken within the three-story railroad building where the tenants were complaining of

vibrations. The results of the readings obtained and the distances from the point of impact are summarized in Table 3. All of the readings are below 2 in./sec (51 mm/s) which for low frequencies has been generally accepted as the level above which damage to residential structures could occur (2). After each impact, the wave frequency was measured in the range of 10 Hz–20 Hz but there was a complete decay before the next impact.

The seismic velocity readings have been plotted on Fig. 5 which relates scaled energy factor to particle velocity. The scaled energy factor is defined as the square root of the energy applied to the ground in foot pounds divided by the distance from the point of measurement to the point of impact. The chart was prepared by Wiss (2) to predict particle velocity resulting from pile driving operations when the subsoils consisted of wet sand, dry sand, and clay. At

TABLE 3.—Record of Vibration Measurements

Location (1)	Distance from point of impact, in feet (2)	Particle velocity, in inches per second (3)	√Energy, in foot-pounds/ Distance, in feet (4)
1. Sidewalk at north property line	30	0.696	13.8
2. Sidewalk adjacent to railroad building	108	0.174	3.8
3. Street on west side of project	70	0.270	5.9
4. At grade beyond south edge of property	200	0.085	2.0
5. First floor of railroad building	155	0.051	2.7
6. Basement of railroad building	120	0.070	3.4
7. Second floor of railroad building	115	0.040	3.6
8. Roof of railroad building	115	0.055	3.6

Note: 1 ft = 0.305 m.; 1 in. = 25.4 mm; 1 lbf = 4.45 N.

Site 5, the subsurface profile consists of building rubble. Points 1 to 4 of Table 3 define a new line on Fig. 5 which can be labeled as rubble fill. These 4 points were observed measurements made at ground surface at various distances from the point of impact. All the measurements taken within the building fall within the range that indicates perceptible vibration and this agrees with the reaction of the tenants. No damage occurred within this building even though the site densification took place over a period of about 3 weeks. The readings taken immediately adjacent to the area being densified indicates an objectionable range of ground vibrations but this instrument was located at the property line where no structures or permanent facilities were located.

One of the advantages of plotting the seismograph readings on a plot like Fig. 5 is that the data can be extrapolated to determine the distance from the

point of impact where damage to a structure could occur. The generally accepted safe level for particle velocity to prevent damages to residences is 2 in./sec (51 mm/s) (2). Extrapolating the line labeled building rubble to an intersection with 2 in./sec (51 mm/s) would result in a scaled energy factor of about 41. For the amount of energy applied at this site, the anticipated distance at which the particle velocity would be 2 in./sec (51 mm/s) computes to be 10 ft (3.1 m). If the impact energy were changed to 6 tons (5.4 metric tons) and a drop

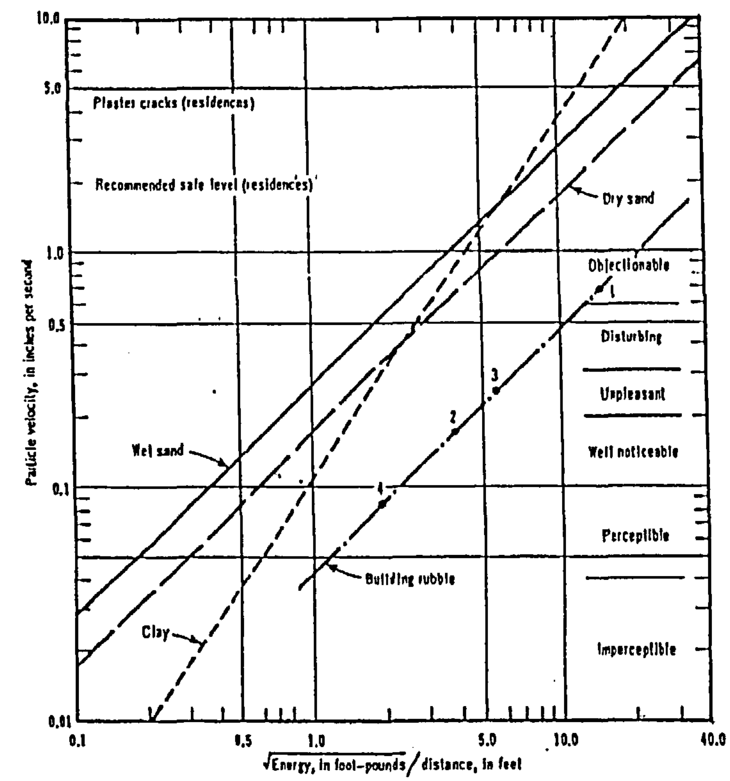

FIG. 5.—Scaled Energy Factor Versus Particle Velocity: Site 5

height of 40 ft (12.2 m), the distance beyond which the particle velocity is predicted to be less than 2 in./sec (51 mm/s) computes to be 17 ft (5.2 m).

For future projects, it would appear worthwhile to take measurements with a portable seismograph at varying distances from the point of impact during driving and then to plot the data on a chart such as Fig. 5 to develop the relationship between particle velocity and scaled energy for that particular site. This data could then be extrapolated to determine the appropriate distances that the points of impact should be kept from nearby structures to prevent damage.

Limitations

Some fill deposits were found to resist densification and had to be removed and replaced with a better material. This included fill containing a high proportion of wood, pockets of sawdust, and high water content organic soils.

Most materials respond to the pounding process by an improvement of the properties. However, there is a limitation as to how much property improvement can be achieved and this must be kept in mind when designing structures to be supported on these deposits. As an example, a localized area of Site 4 consisted of cinders, glass and clay fill. This deposit was densified by pounding and achieved a pressuremeter modulus of only 40 tsf–50 tsf (3.8 MPa–4.8 MPa), whereas, a pressuremeter modulus of 70 tsf–90 tsf (6.7 MPa–8.6 MPa) was typically achieved in other areas of the project. Additional pounding was undertaken in this area to further improve the properties of the soils but the modulus could not be improved. At Site 6, represented by Fig. 3(*a*), the Standard Penetration Resistance of the sandy soil at the 4 ft (1.2 m) level remained at about 20 blows/ft after single and double coverage.

Pounding of fine grained saturated soils should be approached with caution. The experience gained at one project site indicates that the degree of improvement attained is limited and occurs at a slow rate.

Structural Performance

The performance of Sites 1–3 and Sites 5–7 have been checked only by visual inspections. In all of these sites, no visual effects of settlement were observed. At Site 4, two of the shopping center buildings were monitored for settlement. At this site, there is 60 ft (18.3 m) of fill consisting primarily of an old refuse dump and miscellaneous fill that had been dumped over the years. Within Building A, 23 columns were monitored and within a period of 6 months after completion of the structure, the maximum observed settlement was 1/2 in. (13 mm). The typical column settlement was 1/4 in. (6.3 mm) or less. Within Building B, 67 columns were monitored and within 2 months after completion of the building, the average settlement ranged from 1/4 in.–9/16 in. (6.3 mm–14.5 mm). The settlement occurred as the column loads were applied and stopped when the structures were completed. The buildings were designed to take additional settlement due to potential future decomposition of the underlying organic matter within the fill deposits. Fortunately, the refuse which had been placed at this site was deposited 30 or more years ago and most of the organic decomposition has already occurred. In addition, open burning was undertaken at this pit when the refuse was dumped so a large part of the fill consisted of ashes and decomposed material. The column loads for both of these buildings are on the order of 300 kips (1,335 kN). Building A was completed in 1977 and Building B in 1978 and to date, the performance has been satisfactory.

At Site 8, footing settlement of 1 in.–2 in. (25 mm–51 mm) was recorded in an area where saturated clayey silt soils were present at footing level. This settlement occurred before any structural loads were applied to the footings and the settlement is attributed to dissipation of pore pressures following pounding. The footings were constructed 2 weeks–4 weeks after pounding and settlement continued until 6 weeks–8 weeks after pounding. The structural loads were applied 12 weeks after pounding and no measureable settlement occurred when these loads were applied.

Practical Applications

There are many marginal sites especially in urban areas. These marginal sites frequently consist of land that has been filled to raise the grade over soft ground deposits or where buildings have been wrecked and the rubble has been left in place to fill the former basement areas. The pounding process has proven to be practical and economical for improving these sites to support structures of one story–four stories in height. The costs of pounding wherein the site was improved to levels of 10 ft–15 ft (3.1 m–5.3 m) below grade have ranged from \$0.50/sq ft–\$1.00/sq ft. Alternative designs have been priced as more expensive. Removal and replacement with compacted fill assuming 10 ft (3.1 m) of existing fill depth has been priced at three to five times site improvement costs. The cost for extended foundations depends upon the length of foundation required. At Site 4, deep foundations were priced ten times site improvement costs while at Site 7, the cost ratio was 3.5.

Conclusions

On the basis of the data obtained in conjunction with the construction of eight projects, plus the performance of the structures afterwards, it can be concluded that:

1. The pounding process which consists of dropping a heavy weight through a predetermined distance to impact into the soil is a practical way of densifying certain marginal sites. In partly saturated materials above the water table densification is due to compaction plus a collapse of any large voids which may be present therein. In saturated clayey silt, densification is due to consolidation following liquefaction.
2. The depth of improvement was observed to levels of 10 ft–20 ft (3.1 m–6.1 m) below ground surface for weights on the order of 2 tons–6 tons (1.8 metric tons–5.4 metric tons) which were dropped through distances of 30 ft–35 ft (9.2 m–10.7 m) with 7 tamps per location–9 tamps per location. The depth of improvement was approx 65%–80% of the square root of the product of the weight in metric tons and the drop in meters. The number of coverages applied to the area does not appear to affect the depth of improvement.
3. The energy imparted to the improved zone is approximately equal to the Standard Proctor (ASTM D-698) energy when a 6-ton (5.4-metric ton) weight is dropped a height of 35 ft–40 ft (10.7 m–12.2 m) with 9 tamps per impact point.
4. When the weight impacts into the soil, ground vibrations are transmitted off the site. A method of estimating the particle velocity at a distance from the point of impact is described in the main text. For building rubble, the distance from the point of impact to the location where the particle velocity will be 2 in./sec (51 mm/s) computes to 17 ft (5.2 m) for a weight of 6 tons (5.4 metric tons) falling 40 ft (12.2 m).
5. Each material that is densified achieves a maximum or limited improvement.

This must be kept in mind when designing structures to be supported on these deposits.

Acknowledgments

The writer is indebted to the field engineering personnel who monitored the pounding operations and assisted in making field adjustments as this method of densification was developed and improved. In particular, Norman Seiler who was involved with five projects and Sylvio Pollici who was involved with one project, are deserving of special mention.

Appendix I.—References

1. Menard, L., and Broise, Y., "Theoretical and Practical Aspects of Dynamic Consolidation," *Geotechnique*, Vol. 25, No. 1, 1975.
2. Wiss, J., "Damage Effects of Pile Driving Vibration," *Highway Research Record Number 155*, 1967.

Appendix II.—Notation

The following symbols are used in this paper:

D = depth of improvement, in meters;
H = height of drop, in meters; and
W = weight of hammer, in metric tons.

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Cone Penetration in Soil Profiling

By Mohsen M. Baligh,[1] M. ASCE, Vitoon Vivatrat,[2] and Charles C. Ladd,[3] F. ASCE

Introduction

The primary objectives of a soil exploration program are to determine the nature of the subsurface stratigraphy (i.e., the extent, thickness and location of the different soil layers) and those engineering properties of the layers pertinent to foundation design. Accurate information on soil stratification and variability in properties is necessary for the geotechnical engineer to interpolate reliably between data obtained from widely-spaced boreholes, plan the final testing program, select adequate spatial distributions of soil parameters and hopefully reduce the risk of inadequate foundation performances.

The most common U.S. practice for soil investigations involves the Standard Penetration Test for soil identification and empirical correlations with engineering properties and undisturbed sampling for various laboratory strength and consolidation testing. The quasi-static (Dutch) cone test is widely used in Europe and many parts of the world for soil exploration (Sanglerat, 1972; ESOPT, 1974), but has had relatively limited acceptance in the U.S. The Dutch cone test enables essentially continuous measurements of cone resistance, q_c, and sleeve friction, f_s, as the cone is pushed into the soil. These data are generally interpreted, with varying degrees of reliability, on the basis of empirical correlations to: (1) Identify soil types; (2) estimate the undrained shear strength of clays and the friction angle and compressibility of sands; and (3) predict the point and shaft resistances of piles (Schmertmann, 1975; Mitchell and Gardner, 1975; Lunne, et al., 1976; and Meyerhof, 1976). Another type of cone penetration test, the piezometer probe, independently developed by Wissa, et al. (1975)

Note.—Discussion open until September 1, 1980. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 106, No. GT4, April, 1980. Manuscript was submitted for review for possible publication on January 2, 1979.

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ATTACHMENT 3

DAMAGE EFFECTS OF PILE DRIVING VIBRATION

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Damage Effects of Pile Driving Vibration

JOHN F. WISS, Wiss, Janney, Elstner and Assoc.

•PILE DRIVING, like dynamite blasts, nuclear blasts, and sonic booms, is a source of vibration which is frequently alleged to cause damage to structures. Unlike blasts, however, pile driving vibrations are produced by mechanical energy that is limited by the capabilities of the mechanical system. For example, a 5,000-lb ram falling freely from a height of 3 ft cannot deliver more than 15,000 ft-lb of energy on impact. Similarly, the maximum energy available from a double-acting steam hammer is limited by the steam pressure, the area of the piston, and the stroke.

On impact, the energy of the ram is imparted to the pile. It is distributed between rebound of the ram, elastic distortion of the pile, elastic and plastic deformation of the cushioning material, penetration of the pile, and elastic and plastic deformation of the earth surrounding the pile. The elastic deformation of the soil is propagated through the earth materials as elastic waves. The distribution of the available impact energy to the sources previously mentioned consists of interrelated functions, but the most important factor is the resistance of the soil to penetration by the pile. In a soft, easily penetrated soil, most of the energy is used in advancing the pile, and the least amount in the elastic deformation of the soil. In very hard, resistant soil the converse is true.

It is convenient to visualize the wave motion at the surface of the earth as being similar to the ripples produced on a smooth surface of water when a stone is thrown in. The wave length of the earth waves from pile driving is approximately 200 ft; this is the distance from the crest of one wave to the crest of the succeeding wave. Structures supported on the surface ride such waves in the same manner as a cork or box floating on the ripples of the water. Deeply embedded structures respond to a lesser degree in proportion to the orbital diameter of the earth particle motion which decreases exponentially with depth. For example, a structure embedded 200 ft below the surface would receive virtually no vibration. One at 100 ft would receive $\frac{1}{32}$th of the vibration experienced by a point on the surface. Regardless of depth, the magnitude of vibration intensity varies with the amount of energy transmitted to the soil, the physical properties of the soil, and the distance that the wave has traveled from the source.

Many instruments are capable of measuring the vibration intensities resulting from pile driving. Basically, such systems consist of a vibration sensor which converts the physical motion of the earth or structure into electrical signals. These in turn are sufficiently strengthened by an electronic amplifier to drive a galvaniometer and produce a recording of vibration vs time. It is essential to record the vibration, because the impulses are transient, and the response of meters is not fast enough to follow the vibrations accurately. It is also important to record simultaneously the vibratory motion in three mutually perpendicular directions. Although the impact force is generally in the vertical direction, the maximum earth or structural vibration is not necessarily vertical.

The instrument most commonly used for measurement of earth or structural vibration resulting from pile driving is the portable three-component seismograph. This unit is a mechanical optical system which utilizes seismic principles, is portable and battery operated, and produces a recording of displacement in three mutually perpendicular directions vs time. It is ideally suited for field recordings of the vibrations associated with pile driving.

Paper sponsored by Committee on Construction Practices—Structures and presented at the 45th Annual Meeting.

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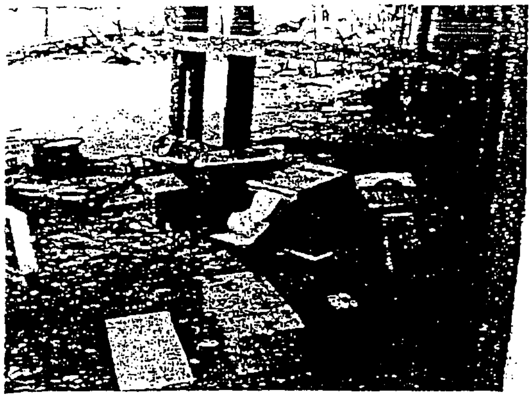

Figure 1. Sprengnether portable seismograph and equivalent complement of electronic equipment and transducers.

Figure 1 shows the Sprengnether portable seismograph which we have used extensively, and an equivalent complement of electronic equipment and transducers. Typical recordings of vibration from pile-driving operations are shown in Figure 2.

The damage potential of pile-driving vibrations depends on the displacement and the frequency of the vibration. Neither of these two characteristics alone will damage a structure. Concerning displacement, it is common knowledge that a structure can be uniformly jacked through several feet without causing damage. Likewise, with regard to frequency, normal sound, in passing through a wall, can vibrate the wall at high frequencies (several thousand cycles per second) without

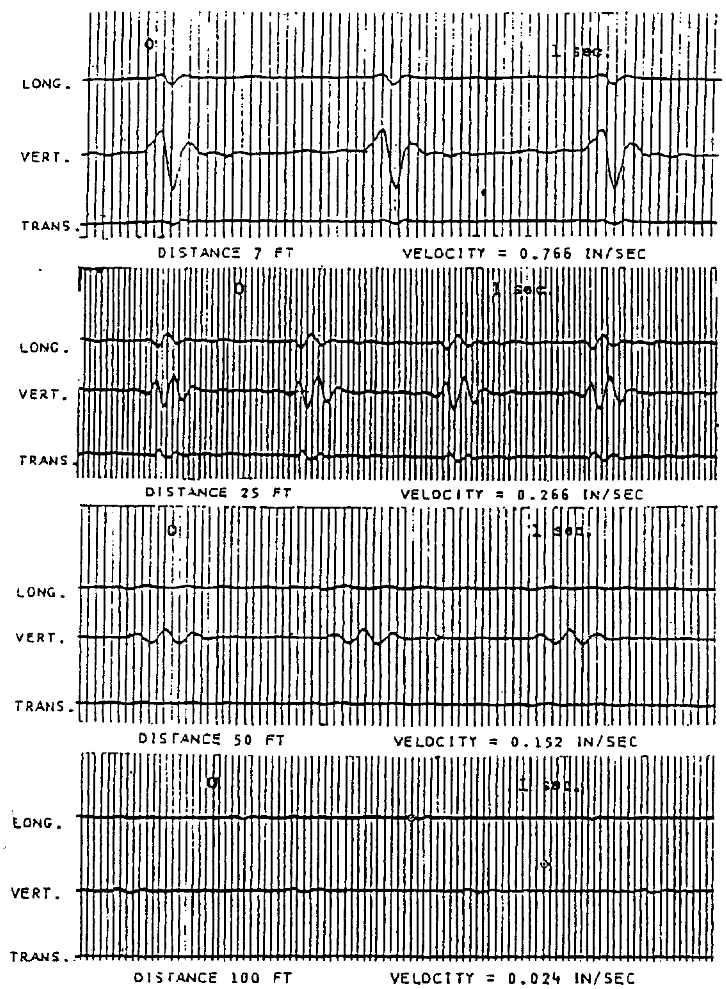

Figure 2. Typical earth vibrations from pile driving.

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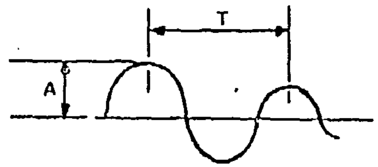

A = AMPLITUDE OR DISPLACEMENT (INCHES)

T = TIME (SECONDS) FOR ONE CYCLE

f = FREQUENCY (CPS) = 1/T

V = PARTICLE VELOCITY (INCHES PER SECOND)

$$V = 2\pi f A$$

TYPICAL SOIL FREQUENCIES

(PILE DRIVING)

ALLUVIAL FILL	5-10 CPS
CLAY	15-25 CPS
SAND	30-40 CPS

Figure 3. Particle velocity in alluvial fill, clay, and sand.

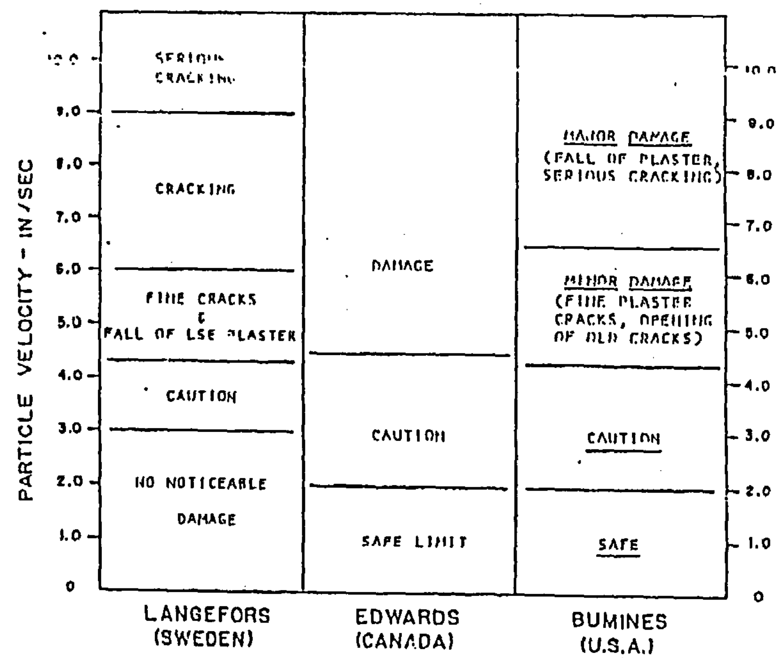

Figure 4. Comparison of damage criteria, residential-type structures.

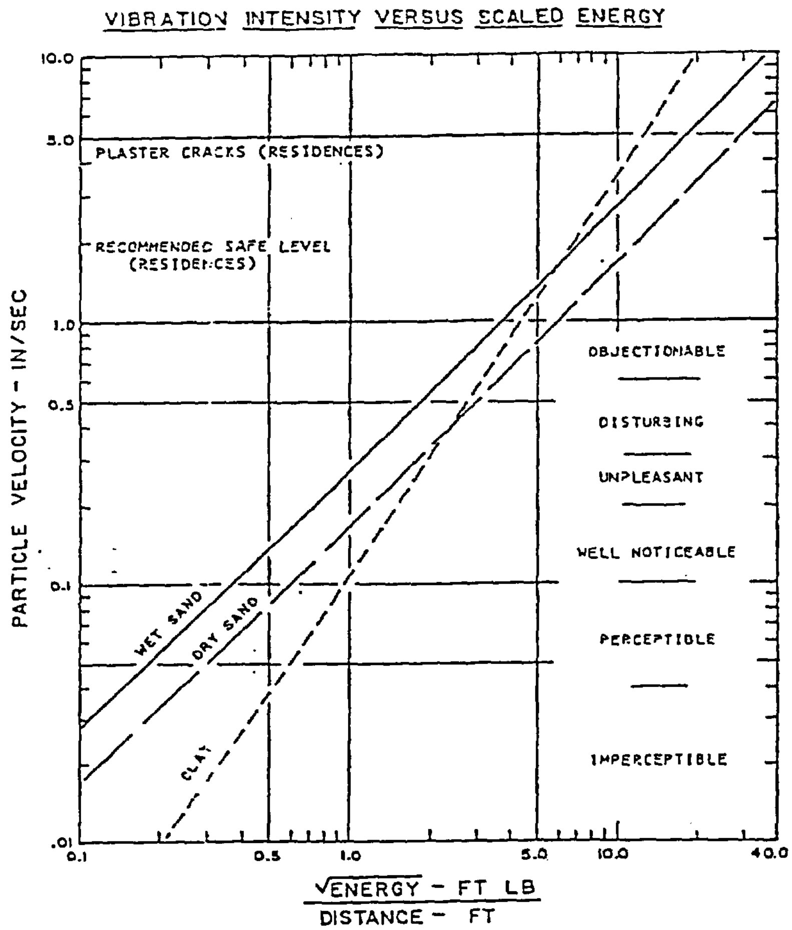

Figure 5. Maximum vibration intensities expected from pile driving on wet sand, dry sand, and clay.

causing damage. It is a combination of displacement (amount of motion) and frequency which causes damage. The particle velocity of earthborne vibration is the best measure of damage potential because it combines displacement and frequency in the most significant manner. Particle velocity (Fig. 3) can be expressed as $2\pi fA$, in which f is frequency (cps) and A is amplitude (displacement). Impact vibrations produced by pile driving have characteristic frequencies depending on the type of soil. A loose alluvial fill has natural frequencies of about 5 to 10 cps, clay soils vary between 15 and 25 cps, sand between 30 and 40 cps.

Several investigators in this country and abroad (including the U.S. Bureau of Mines) have found that particle velocities in excess of 4.0 in./sec are required to cause plaster cracks in dwellings. Figure 4 shows a comparison of the results of several of the investigations. With appropriate conservatism, the investigators agree that a vibration level of 2.0 in./sec (particle velocity) is safe with regard to plaster cracks in residential-type structures.

The effect of ground motion on an engineered structure can be computed by commonly used methods in the earthquake engineering field. The structure is considered a lumped mass-spring dashpot system, and its response to a series of impacts can be calculated. Based on observation and experience, it can be stated that ground motion particle velocities below 4.0 in./sec are well within the safe range for engineer structures.

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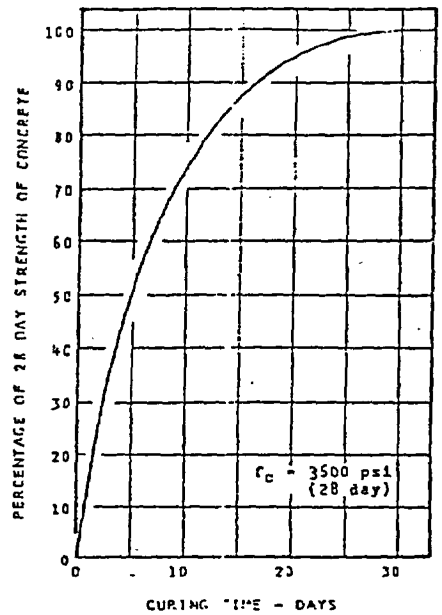

Figure 6. Strength of concrete vs curing time.

Figure 5 shows the maximum vibration intensities to be expected from pile driving in several soils on which extensive data have been obtained by the author. The data are plotted on log-log paper in which the abscissa is $\sqrt{E/D}$. This scaled energy factor permits use of the graphs with any size of pile driver; E is the foot-pounds of energy delivered by the hammer, and D is the seismic distance, in feet, from the pile tip to the location of interest. The vibration intensity (particle velocity) varies as the square root of the energy of the hammer. Figure 5 also indicates the levels at which vibration damage may be expected and the normal human evaluation of pile driving vibration. In several investigations, vibrations resulting from the driving of sheet piling, wood piles, and H piles were measured. For all practical purposes there is no difference in the vibration produced, all other variables being constant.

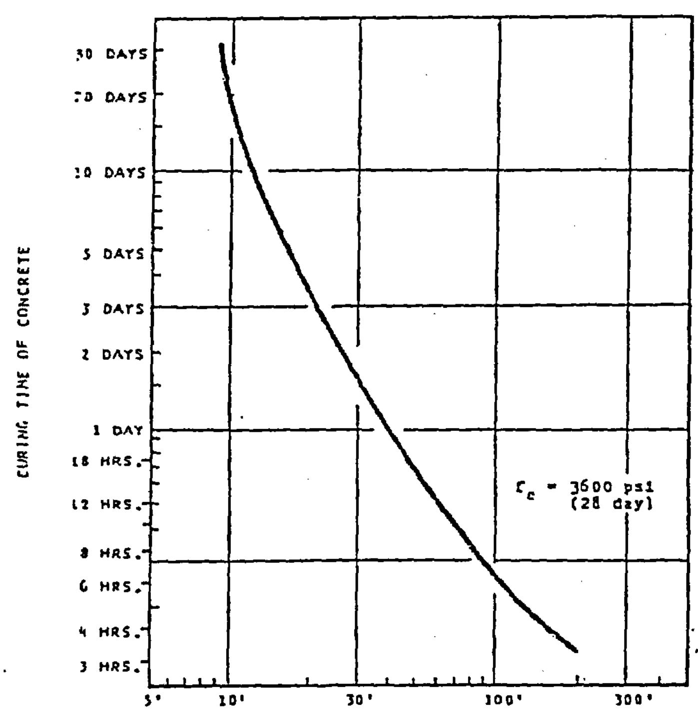

Figure 7. Limiting safe distance vs curing time of concrete for pile driver rated at 15,000 ft-lb of energy.

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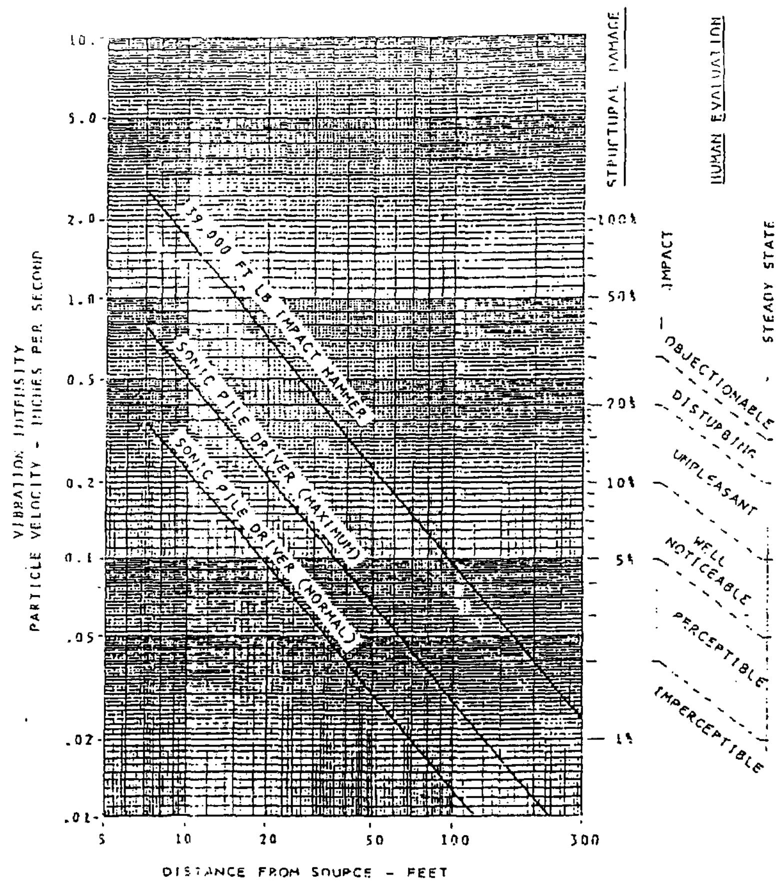

Figure 8. Vibration intensity vs distance.

Another problem of common interest on a construction project involves the case in which piles are to be driven at the same time that concrete is being placed. The question has frequently been raised as to whether the pile driving might have a detrimental effect on "green" concrete. Until more is known about concrete technology, it is doubtful that a rigorous analysis of such effects can be made.

As a practical matter, the following reasoning has been used by the author in the past, and although the magnitude of the safety factor has not been determined, there has been no evidence to indicate that the approach is not conservative.

Assuming that 5.0 in./sec (particle velocity) is conservatively a safe vibration level for cured concrete, and recognizing the rate at which green concrete attains strength, it is then possible to relate the permissible safe vibration level to the time since placing the concrete. When the decrease of vibration with distance has been evaluated for a particular site, the distance at which pile driving may be permitted (for a certain size hammer) can then be determined as a function of curing time.

Assume, for purposes of illustration, that a 3,600-psi concrete (28-day strength) has been specified on a particular project. The percentage of 28-day strength of concrete vs curing time is shown in Figure 6. This curve shows that the concrete has approximately 5 percent of its strength in 12 hr (one-half day), or 10 percent in 24 hr (one day). Thus, the vibration intensity should not exceed this same percentage of a particle velocity of 5.0 in./sec; limiting values of vibration are, therefore, 0.25 and 0.5 in./sec, respectively. If the soil is basically a clay, these vibration particle velocities correspond to an $\sqrt{E/D}$ of 2 and 3, respectively (Fig. 5). If a 15,000-ft-lb

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pile driver is used, the closest permissible distances are 61.5 and 41.0 ft, respectively.

By the foregoing method a curve can be developed in which the limiting safe distance for pile driving vs curing time of concrete can be determined (Fig. 7). This curve is represented as a typical evaluation. For a specific site, pile driver, and concrete, limiting distances and curing time should be based on measured vibration intensities and concrete strength determinations, especially where short curing times are involved.

In closing, some brief comments on the vibration produced by the sonic pile driver and by other vibratory pile drivers are pertinent. In contrast to the impact hammer, which excites the soil at its natural frequency and the vibrations die out before the next blow, the sonic and vibratory pile drivers force the soil to vibrate at the continuous frequency (rpm) of the driver. These units can be speed controlled over a limited frequency range. Investigations of a sonic pile driver driven at frequencies between 90 and 120 cps, and a vibratory pile driver adjustable between 16 and 21 cps resulted in the following observations.

The normal vibration levels from the sonic driver may by one order of magnitude lower than those of an impact pile driver. However, the vibration varies continuously and occasionally attains intensities approximately one-half of the levels produced by a comparable impact hammer. Further, because the vibration is of a steady-state rather than a transient character, the human evaluation is usually more pronounced—by a factor of 2 (Fig. 8). The other vibratory pile driver investigated produced vibration levels of the same order of magnitude as a comparable impact pile driver.

With a steady-state excitation the possibility of resonance response in building components (especially panels) may become of significant importance. In the case of the transient vibrations produced by an impact pile driver, the duration of the transient is sufficiently short (0.2-0.3 sec) that a resonance buildup of structural components is not likely. The safe level of intensity for a steady-state vibration could conceivably be between one-half and one-fifth of the safe level for transient excitation; this is due to the possible magnification at resonance which depends primarily on the inherent damping characteristics of the structure.

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ENCLOSURE 2

TENNESSEE VALLEY AUTHORITY
SEQUOYAH NUCLEAR PLANT (SQN)
UNIT 1
DOCKET NO. 327

TOPICAL REPORT NO. 24370-TR-C-002,REVISION 3
"RIGGING AND HEAVY LOAD HANDLING"

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SEQUOYAH UNIT 1 STEAM GENERATOR REPLACEMENT

RIGGING AND HEAVY LOAD HANDLING TOPICAL REPORT

REV.	DATE	REASON FOR REVISION	BY	EGS	PE
3	12-24-02	Updated to Reflect Responses to NRC RAIs		MRA	
2	4/12/02	Incorporated TVA Comments	SWK	MRA	CCG for JVS
1	3/6/02	Incorporated TVA Comments	SWK	MRA	JVS
0	2/14/02	Issued for TVA use	SWK	DLK	JVS

BECHTEL

JOB NO.: 24370

DOCUMENT NO.:
24370-TR-C-002

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SEQUOYAH UNIT 1

STEAM GENERATOR REPLACEMENT

RIGGING and HEAVY LOAD HANDLING

TOPICAL REPORT

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1.0 Abstract

In response to NRC Generic Letters 80-113 (Reference 18) and 81-07 (Reference 19), TVA established a program for the control of heavy loads at Sequoyah. This program, which addresses the guidance provided in NUREG-0612 (Reference 12), was reviewed by the NRC and incorporated into plant Procedure 0-MI-MXX-000-026.0 (Reference 8). During the upcoming Steam Generator Replacement (SGR) at Sequoyah Unit 1, which will occur during the Unit 1 Cycle 12 refueling outage, heavy loads exceeding those anticipated by Reference 8 will be handled using new safe load paths. In some cases, the load paths traverse over safety-related equipment supporting operation and safe-shutdown capability for Unit 2, which will remain in operation during the Unit 1 Steam Generator Replacement Outage (SGRO).

As defined in NRC Bulletin 96-02 (Reference 13), licensees planning to perform activities involving the handling of heavy loads over safety-related equipment while the reactor is at power and involving a potential load drop accident that has not previously been addressed in the FSAR should submit a license amendment request to the NRC. Following recent revisions to 10CFR50.59, the Bulletin's guidance was supplemented by NRC Regulatory Issue Summary 2001-03 (Reference 20), which states that, "The fact that the load is larger or is moving in a different load path than previously evaluated would enter into the risk assessment required by 10CFR50.65(a)(4) and determine under what plant conditions the load lift should occur."

This Topical Report documents the provisions made to ensure that heavy load handling activities associated with the Unit 1 SGRO can be accomplished without impacting the safe operation of Unit 2. These provisions support the risk assessment required by 10CFR50.65(a)(4) and an application for a one-time license amendment associated with the operability of the Essential Raw Cooling Water (ERCW) System. As concluded in Appendix B, these provisions and one-time license amendment do not involve a significant hazards consideration. Actions required to support the conclusions of this Topical Report are detailed in Appendix A.

2.0 Introduction

This Topical Report provides a description of and technical justification for the use of cranes and rigging of heavy loads over safety-related structures, systems, and components (SSCs) in support of the Sequoyah Unit 1 SGR. The cranes and the heavy loads addressed in this Topical Report are:

- Outside Lift System (OLS) (i.e., Mammoet PTC Heavy Lift Crane)
- Mobile (lattice boom and truck) cranes
- Large crane components
- Old and replacement steam generators
- Reactor shield building dome and steam generator compartment roof concrete sections
- Containment vessel dome steel sections

The activities addressed in this Topical Report are:

- Assembly, use, and disassembly of the OLS.
- Use of the mobile cranes for assembly/disassembly of the OLS.

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- Removal of reactor shield building dome and steam generator compartment roof concrete sections and containment vessel dome steel sections.
- Removal of the old steam generators (OSGs) and installation of the replacement steam generators (RSGs).
- SSC protection from external events and postulated load drops.

The OLS is commercially designed and, therefore, is considered as non-safety related. The OLS was not specifically designed to withstand the external events addressed by 10CFR50, Appendix A, General Design Criterion (GDC) 2 that are a part of the Sequoyah design and licensing basis. However, due to the size of the OLS and because of the OLS location and proximity to the Containment, Auxiliary Building, Essential Raw Cooling Water (ERCW) piping, Refueling Water Storage Tank (RWST), Main Steam (MS) piping, and Feedwater (FW) piping, the OLS was evaluated as indicated below for those external events that might cause it to collapse when these SSCs are required to be operable.

The OLS was analyzed for both loaded and unloaded configurations for structural adequacy with design basis earthquake (DBE) loads imposed. Details of this seismic evaluation are provided in Section 5.1. As also discussed in Section 5.1, administrative controls will be imposed to restrict crane use and orientation under high winds or severe weather conditions.

This Topical Report also documents the load path provisions, equipment protection techniques, operator training, and compensatory measures that will ensure that OLS assembly/disassembly and load handling with the OLS is performed safely.

3.0 Objectives

This Topical Report provides the technical basis for a one-time Operating License (OL) change applicable to the Unit 1 Cycle 12 refueling outage that establishes that lifting of heavy loads will not affect ERCW system operability provided that the load movements are performed in accordance with this Topical Report and prescribed compensatory measures.

4.0 Regulatory Requirements/Criteria for Handling of Heavy Loads

Detailed below are regulatory requirements/criteria that are relevant to the handling of heavy loads over safety-related equipment. Since the load handling activities described in this topical report do not involve handling of loads over or near spent fuel, requirements related specifically to load handling over fuel are not addressed. Following each requirement/criteria is an *italicized* reference to where the requirement/criteria is addressed within this topical report.

4.1 SRP Section 9.1.5 – Overhead Heavy Load Handling Systems

Standard Review Plan (SRP) Section 9.1.5 addresses the reviews of overhead heavy loads handling systems performed by the NRC to assure conformance with the requirements of 10CFR50, Appendix A, GDC 2, 4, 5, and 61. The heavy load handling system is considered acceptable if the integrated design of the structural, mechanical, and electrical elements, the manual and automatic operating controls, the safety interlocks and devices, and the load handling instructions, inspections, maintenance and testing, provide adequate system control for the specific procedures of handling

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operations, if the redundancy and diversity needed to protect against malfunctions or failures are provided, and if the design conforms to the relevant requirements of the following regulations:

1) GDC 2, as related to the ability of structures, equipment, and mechanisms to withstand the effects of earthquakes. Acceptance is based in part on meeting position C.1 of Regulatory Guide 1.29 for safety-related equipment and position C.2 for non-safety related equipment, and positions C.1 and C.6 of Regulatory Guide 1.13.

   As detailed in Section 5.1, the OLS has been evaluated for seismic loads while unloaded and while loaded with its heaviest load (a steam generator). This seismic evaluation determined that the OLS will not collapse or result in a drop of the load during a seismic design basis safe shutdown earthquake event for the lift configurations to be used for the Sequoyah Unit 1 SGR.

   Per Section 5.2, use of the mobile cranes for OLS assembly/disassembly is limited to an area within 60 ft. of the OLS boom location shown on Figure 5-2. However, mobile crane usage beyond 60 ft. from the OLS boom location may be allowed if Engineering evaluation shows no adverse impact to nearby safety-related SSCs. Protection (see Figure 5-2) for safety-related SSCs is provided, as necessary, to ensure that Unit 1 and Unit 2 can be safely shut down and/or maintained in a safe condition in the unlikely event of a seismically induced load drop during use of these cranes for assembly/disassembly of the OLS.

2) GDC 4, as it relates to protection of safety-related equipment from the effects of internally generated missiles (i.e., dropped loads). Acceptance is based in part on meeting positions C.3 and C.5 of Regulatory Guide 1.13.

   Safety-related SSCs that may be affected by a load drop from the OLS or mobile cranes are described in Section 6. As detailed in Section 8, these SSCs have been evaluated and where necessary, protective or compensatory measures have been determined to mitigate the effects of a load drop induced SSC failure.

3) GDC 5, as related to the sharing of equipment and components important to safety, between Units 1 and 2.

   As detailed in Section 6.3, ERCW is the only shared system that could be affected by load drops from the OLS. Equipment that may be affected by a load drop is detailed in Section 8.3. As indicated in Appendix A, plant procedures will be developed to delineate specific actions required in case of a heavy load drop.

4) GDC 61, as related to the safe handling and storage of fuel.

   Conformance to this GDC is not applicable, as the load handling detailed herein will not involve moving fuel or moving loads over fuel.

Other specific criteria necessary to meet the relevant requirements of GDC 2, 4, and 61 are detailed in NUREG-0612.

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4.2 NUREG-0612 – Control of Heavy Loads at Nuclear Power Plants

Section 5.1 of NUREG-0612 provides guidelines for the control of heavy loads. The objectives of these guidelines, in part, are 1) to assure that the potential for a load drop is extremely small or 2) radioactive releases resulting from damage caused by the load drop are less than 1/4 of 10CFR100 limits (i.e., less than 75 rem thyroid and 6.25 rem whole body) and to ensure that damage to equipment in redundant safe shutdown paths is not sufficient to preclude safe shutdown.

The evaluation of the radiological consequences of dropping an OSG is described in Section 7.1.

The NUREG reflects an overall philosophy that provides a defense-in-depth approach for controlling the handling of heavy loads; i.e., prevent as well as mitigate the consequences of postulated accidental drops. Part of this defense-in-depth approach involves 1) providing sufficient operator training, handling system design, load handling instructions, and equipment inspections to assure reliable operation of the handling system and 2) defining safe load paths through procedures and operator training so that to the extent practical heavy loads being carried over or near safe shutdown equipment are avoided. Where a load path that avoids safe shutdown equipment cannot be defined, alternative measures may be taken to compensate for this situation.

As detailed in Section 7, for the large equipment lifts discussed in this Topical Report, a safe load path has been chosen that minimizes potential interactions with critical equipment. For the lifts that must traverse safe shutdown equipment, compensatory measures will be implemented in the unlikely event of a load drop.

Section 5.1.1 of NUREG-0612 states that all plants should satisfy the following for handling heavy loads that could be brought in proximity to or over safe shutdown equipment:

1) Load paths should be defined for the movement of heavy loads to minimize the potential for heavy loads to impact safe shutdown equipment. These load paths should be defined in procedures, shown on equipment layout drawings, and clearly marked in the area where the load is to be handled.

   Safe load paths for the loads to be handled by the OLS have been identified as shown on Figure 5-2. Criteria for operation of mobile cranes used in the assembly/disassembly of the OLS have been developed as detailed in Section 5.2.

2) Procedures should be developed to cover load handling operations for heavy loads to be handled in proximity to safe shutdown equipment. These procedures should include identification of required equipment, inspections and acceptance criteria required before movement of the load, the steps and proper sequence to be followed in handling the load, the safe load path, and other special precautions.

   Rigging operations using the OLS and mobile cranes will be controlled and conducted by highly trained and qualified personnel in accordance with approved procedures. The entire operation has been evaluated by engineering personnel and documented by calculations, engineering drawings, and procedures.

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Drawings showing the safe load paths have been developed. Assembly and disassembly of the OLS will be performed in accordance with the crane manufacturer's procedures and drawings. Tornado initiated crane failures or load drops will be precluded through implementation of procedures to suspend load handling when high winds or severe weather/tornado conditions are anticipated. As indicated in Appendix A, procedures to implement compensatory measures required to mitigate the effects on ERCW system operation of a postulated load drop will be developed and personnel will be trained in their use.

3) Crane operators should be trained, qualified, and conduct themselves in accordance with ANSI B30.2 Chapter 2-3 guidelines.

   ANSI B30.2 is applicable to overhead gantry cranes. The appropriate guidance for the mobile cranes is ANSI B30.5. The operator training detailed in Sections 5.1 and 5.2 of this topical report conforms to the guidelines of ANSI B30.5, Chapter 5-3.

4) Special lifting devices should satisfy the guidelines of ANSI N14.6, as modified by NUREG-0612.

   The rigging operations addressed in this Topical Report do not use special lifting devices as defined by ANSI N14.6.

   As described in Section 5.1, the OLS attachments and rigging meet the requirements of ASME NQA-1-1997, Subpart 2.15 and the applicable ASME B30 series standards. The attachments and rigging used to attach the OLS to the SGs have been previously load tested in accordance ASME NQA-1, Subpart 2.15 or have a previous load history that exceeds the loads to be lifted.

5) Lifting devices that are not specially designed should be installed and used in accordance with the guidelines of ANSI B30.9, as modified by NUREG-0612.

   As described in Section 5.1, the OLS attachments and rigging meet the requirements of ASME NQA-1-1997, Subpart 2.15 and the applicable ASME B30 series standards. This includes ANSI B30.9 as modified by NUREG-0612.

6) The crane should be inspected, tested, and maintained in accordance with ANSI B30.2, as modified by NUREG-0612.

   ANSI B30.2 is applicable to overhead gantry cranes. The appropriate guidance for the mobile cranes is ANSI B30.5. The crane inspections, testing, and maintenance detailed in Sections 5.1 and 5.2 of this topical report conform to the guidelines of ANSI B30.5, as modified by NUREG-0612.

7) The crane should be designed to meet the applicable criteria and guidelines of Chapter 2-1 of ANSI B30.2 and of CMAA-70.

   ANSI B30.2 is applicable to overhead gantry cranes. The appropriate guidance for the mobile cranes is ANSI B30.5. The manufacturer's user manual for the OLS also refers to ANSI B30.5. The crane designs detailed in Sections 5.1 and 5.2 of this topical report conform to the guidelines of ANSI B30.5, which meets the intent of ANSI B30.2 and CMAA-70.

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Section 5.1.5 of NUREG-0612 states that in addition to the above requirements from Section 5.1.1, the effects of load drops should be analyzed (in accordance with the guidelines of Appendix A to NUREG-0612) and the results should indicate that damage to safe shutdown equipment is not sufficient to preclude safe shutdown.

Appendix A of NUREG-0612 states, in part, that analyses of postulated load drops should as a minimum include the following considerations:

1) The load is dropped in an orientation that causes the most severe consequences.

   The consequences of a postulated load drop from the OLS or the mobile cranes are detailed in Section 7. Where it was not possible to protect SSCs in the vicinity of the load drop, the worse case failure of these SSCs was postulated.

2) The load may be dropped at any location in the crane travel area where movement is not restricted by mechanical stops or electrical interlocks.

   As detailed in Section 7.1, loads drops along the entire load path have been postulated and evaluated. The load path is maintained by strict administrative controls. These administrative controls will be in the form of notes on drawings and procedural steps contained in controlled work packages.

3) X/Q values for determining the radiological consequences of a heavy load drop should be derived from analysis of onsite meteorological measurements based on 5% worst meteorological conditions.

   The meteorological conditions and the X/Q values used to determine the doses resulting from a postulated drop of an OSG are detailed in Section 7.1.

4) Analyses should be based on an elastic-plastic curve that represents a true stress-strain relationship.

   As detailed in Sections 7 and 8, when appropriate, the analyses are based on the true material characteristics.

5) The analysis should postulate the "maximum damage" that could result (i.e., the analysis should consider that all energy is absorbed by the structure and/or equipment that is impacted).

   Where it was not possible to analytically show that a SSC would survive the impact of a postulated load drop, the SSC was assumed to fail to the point where it could no longer perform its design function. If this failure could result in an adverse impact on other SSCs, this impact was accounted for in assessing whether compensatory measures were required to restore the affected functions.

6) Credit may not be taken for equipment to operate that may mitigate the effects of the load drop if the equipment is not required to be operable by the Technical Specifications when the load could be dropped.

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No credit has been taken for equipment not required to be operable by the Technical Specifications.

4.3 NRC Bulletin 96-02 – Movement of Heavy Loads Over Spent Fuel, Over Fuel in the Reactor Core, or Over Safety-Related Equipment

This bulletin requires licensees planning to handle heavy loads over safety-related equipment while the reactor is at power that involve a potential load drop accident that has not been previously evaluated in the FSAR or a change to the Technical Specifications, to submit a license amendment request in advance of the planned load movement so as to afford the NRC sufficient time for review and approval.

Since the postulated load drops could adversely affect safety-related components that are addressed in the Technical Specifications, this Topical Report has been prepared to support NRC review and approval of an amendment to the Unit 2 Operating License to add a one-time condition for conduct of heavy load lifts associated with the Unit 1 steam generator replacement (Reference Section 8.3).

4.4 NRC Regulatory Issue Summary 2001-03 – Changes, Tests, and Experiments

Attachment 1 to Regulatory Issue Summary 2001-03, Issue 7, states, "With respect to [Bulletin] 96-02, if a heavy load movement is part of a maintenance activity, there is no 10CFR50.59 evaluation needed. The fact that the load is larger or is moving in a different load path than previously evaluated would enter into the risk assessment required by 10CFR50.65(a)(4) and determine under what plant conditions the load lift should occur. If the heavy load lift is not maintenance related, and so requires a 10CFR50.59 evaluation, the licensee should follow the requirements of the revised rule to determine whether prior NRC approval is needed.

This Topical Report documents the provisions made to minimize and control the risks associated with the subject lifts. While the lifts are associated with a maintenance activity for Unit 1, it is TVA's intent that Unit 2 continues normal operation during the Unit 1 SGRO. Because of the potential interactions of the Unit 1 activities upon Unit 2 safety, and to clarify the operational issues associated with the plant Technical Specifications, a license amendment based upon this Topical Report will be requested.

5.0 Description of Cranes and Heavy Loads

The cranes described herein are commercially available equipment and are not specifically designed as single failure proof, nor are they specifically designed to withstand the external events that are a part of the plant licensing basis. Since this rigging equipment will carry large and heavy loads in the vicinity of safety-related SSCs, it must be demonstrated that the installation, use, and removal of this rigging equipment does not adversely affect the safety function of these SSCs or that alternative means of performing the SSC safety function are available.

5.1 Outside Lift System

The OLS will consist of a Mammoet Platform Twin-Ring Containerized (PTC) Heavy Lift Crane (see Figure 5-1), which is a commercially designed crane. The maximum rated load for this crane is 1763.2 tons (1600 metric tons), however this will vary with crane

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configuration and lift radius. The rated load for the crane configuration proposed for the Sequoyah SGR ranges from 440.8 tons (400 metric tons) to 517.9 tons (470 metric tons), depending on the lift radius. The OLS meets or exceeds ASME NQA-1 Subpart 2.15 design requirements, and its load charts and operating restrictions consider applicable dead, live, wind, impact, and out-of-plumb lift loads. The OLS, supplied with standard load charts for its various boom configurations, has a rated load capacity certified by the manufacturer and has been load tested during its production; this meets the load test requirements of ASME NQA-1, 1997 Edition, Subpart 2.15, Section 601.2. In addition, after the OLS has been erected it will be load tested by lifting a 275 ton (550 kip) test load assembly with the OLS boomed out to a radius where the test load represents 110% of the OLS rated capacity at this radius. OLS lifts of the loads described in this topical report will be performed after Unit 1 is defueled and will be completed prior to the start of refueling. The OLS load test may be performed with Units 1 and 2 in any mode (Reference 24). The OLS will be located in an area between the Service Building, the Unit 1 RWST and the Unit 1 Containment, as shown on Figure 5-2.

The OLS consists of a main A-frame boom, which is pinned to and rides on wheel trucks at its base, and has a jib boom and 2 stay beams pinned to its top end. The main boom is stabilized by a counterweight system including a backmast boom that also rides on wheel trucks. The OLS wheel trucks ride on the base ring supported by built-in outrigger support rings/plates, which enable the OLS to be self-leveling, as shown on Figure 5-3.

OLS attachments that have been specially designed for SG rigging purposes will be connected to the steam generator during Modes 5 or 6 or the defueled condition while the OSG is still within its compartment. The OLS will be attached to the SGs, Shield Building concrete sections, steel Containment vessel sections, and SG compartment concrete sections using slings, cables, spreader beams, etc. attached to the OLS load block. The OLS attachments and rigging meet the requirements of ASME NQA-1-1997, Subpart 2.15 and the applicable ASME B30 series standards. The attachments and rigging used to attach the OLS to the SGs have been previously load tested in accordance ASME NQA-1, Subpart 2.15 or have a previous load history that exceeds the loads to be lifted. Rigging will be inspected prior to use in accordance with approved procedures and rigging operations will be controlled and conducted by highly trained and qualified personnel in accordance with approved procedures.

Personnel involved in operating the OLS will receive the following instruction:

- Operators will receive the applicable Sequoyah site-specific training specified in Appendix C of MMDP-2, "Safe Practices for Overhead Handling Equipment" (Reference 9).
- Personnel will undergo hands on training with the equipment before a load is attached to the equipment.
- Prior to a lift, detailed pre-lift meetings will be conducted.
- Direction to the operators during each OLS lift will be given by technical representatives of the equipment owner and the SGR contractor rigging specialist.

During the lifting operation, the exact location of boom tip and load block will be monitored by two independent methods. Instrumentation internal to the crane provides continuous readout of crane and boom orientation and the location of the boom tip and load block. In addition, the boom tip will be continuously monitored from a remote survey station independent from the crane instrumentation. This survey station will have the necessary data input to monitor and calculate the boom tip location relative to the

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interfacing structures and components. The individual directing the rigging operations will be in constant communication with both the crane operator and the surveyor manning the remote survey station. These controls will be utilized to ensure that the exact location of the load is known and compliance with design requirements is maintained.

Assembly and disassembly of the OLS will be performed in accordance with the crane manufacturer's procedures and drawings and may be performed with Unit 1 and Unit 2 in Modes 1-6 or defueled. The assembly/disassembly process will require the use of mobile cranes and other equipment as detailed in 5.2. During assembly and disassembly of the OLS, the main boom will lay in an area to the north of the Unit 1 Containment as shown on Figure 5-2. The orientation of the main boom during assembly/disassembly along with the restrictions on mobile crane usage and SSC protection provisions in 5.2 ensure that Unit 1 and Unit 2 can be safely shut down and/or maintained in a safe condition in the unlikely event of a load drop during assembly/disassembly of the OLS.

The OLS has been evaluated for seismic loads while unloaded and while loaded with a steam generator (SG) as detailed in Reference 21. A SG is the heaviest load that will be handled by the OLS. This seismic evaluation determined that the OLS will not collapse or result in a drop of the load during a seismic design basis Safe Shutdown Earthquake (SSE) event for the lift configurations to be used during the Sequoyah Unit 1 SGR. Therefore, use of the crane for the Sequoyah Unit 1 SGR will not result in Seismic II/I interaction issues on the SSCs located in the vicinity of the OLS.

Reference 21 developed a GT-STRUDL 3-D lumped mass finite element model using beam/truss elements to analyze the critical lift configurations of the OLS for SSE loads. NRC Regulatory Guide (RG) 1.61 allows 7% damping for bolted steel structures for the SSE. However, this analysis conservatively used 5%, which is consistent with Table 3.7.1-1 of the UFSAR. The OLS seismic analysis was performed using the response spectrum method in both the loaded and unloaded conditions.

The seismic analysis of the OLS is based on an appropriate ground spectrum corresponding to the plant's minimum SSE design basis spectra. The OLS will be supported on a concrete ring foundation seated on a large number of battered piles anchored to bedrock. Based on soil borings the average depth of soil deposit at the location of the OLS is 30 ft. The 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. under the crane foundation. This amplified spectra was obtained by interpolation for a 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. It is noted that the amplified ground spectra documented in Reference 27 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 the UFSAR). A 10% broadened amplified SSE horizontal 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 thus developed from the 20 ft. and 40 ft. curves in Reference 27 and used as the input horizontal spectrum. Since the OLS will be supported on a concrete ring foundation seated on a large number of battered 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

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UFSAR. The vertical response spectrum used is 2/3rds (per Section 2.5.2.4 of UFSAR) of the horizontal minimum design spectrum.

Soil springs were calculated to simulate soil-structure interaction at the foundation. The response spectra loadings were applied simultaneously in two horizontal directions and the vertical direction. Modal responses were combined using the NRC Ten-Percent Method. Co-directional responses were combined using the Square Root of the Sum of the Squares (SRSS) method.

The seismic evaluation of the OLS determined that the calculated stresses are less than the maximum allowable stresses (0.9 F_y) and the minimum safety factor against overturning is 1.1.

To further demonstrate the capability of the OLS, Reference 21 also determined the "whip-lash" effect a loss of lifted load would have on the OLS. Reference 21 determined that the whip-lash effect resulting from a postulated drop of a load from the OLS will not cause instability of the boom masts in the reverse direction, i.e. the masts will not flip over backwards and impact SSCs (e.g., Auxiliary Building, Control Building, etc.) behind the OLS.

Rigging operations will not be performed when wind speeds exceed the maximum operating wind speed for the OLS. This wind speed will be measured using an anemometer on the crane boom tip. If wind speeds increase during a rigging operation such that the wind speed may exceed the maximum operating speed, rigging operations will be suspended and the unloaded OLS will be secured by implementing administrative controls specified by the manufacturer in Reference 7. These administrative controls define the allowable mainmast and jib angles, and the slew drive and load block configurations, and are dependent on the wind speed.

To eliminate the effects of wind conditions beyond the maximum operating wind speed, a lift will not commence if analysis of weather data for the expected duration of the lift indicates the potential for wind conditions in excess of the maximum operating wind speed. Further, should there be an unexpected detrimental change in weather while the OLS is loaded, the lift will be completed and the OLS will be placed in its optimum safe configuration or the load will be grounded and the crane will be placed in a safe configuration.

Based on the above discussion, the conditions that could result in credible crane failure modes or load drops (i.e., operator errors, use of improper rigging or inappropriate slings, and crane component failures) have been minimized or eliminated through the training of rigging personnel, use of engineer developed procedures for the load lifts, performance of engineering evaluations of the OLS and rigging components, and inspection and testing of the OLS. In addition, an OLS failure or load drop due to a tornado or seismic event has been eliminated. The tornado initiated OLS failure or load drop will be eliminated through implementation of procedures to preclude load handling when high winds or severe weather/tornado conditions are anticipated. The seismic induced crane failure or load drop has been eliminated by showing that the OLS will not collapse or drop a load while loaded or unloaded during the SSE. Given the training, procedures, evaluations, inspections, and testing involved in use of the OLS, it is highly unlikely that the OLS will fail or drop a load. However, as required by NUREG-0612, load drops from the OLS have been postulated and the potential consequences of these postulated drops evaluated as detailed in Sections 7 and 8.

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5.2 Mobile Cranes

Mobile (lattice boom and/or truck) cranes will be used in the assembly/disassembly of the OLS. These cranes will be used when Unit 1 and 2 in are in any operating mode or when Unit 1 is defueled. The lattice boom and truck cranes are commercially designed, ruggedly constructed, cranes with a main boom. The crane with its main boom is stabilized using a counterweight system. The design of the lattice boom and truck cranes meets ASME/ANSI Standard B30.5-2000 design requirements and their rated capacity considers applicable loadings. The lattice boom and truck cranes have been load tested during their production and will have a current certification.

Use of the mobile cranes for OLS assembly/disassembly is limited to an area within 60 ft. of the OLS boom location shown on Figure 5-2. Mobile crane usage beyond 60 ft. from the OLS boom location may be allowed if Engineering evaluation shows no adverse impact to nearby safety-related SSCs. Restrictions on the use of these cranes will also 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. These restrictions are designed to preclude adverse interactions with safety-related SSCs. Protection (see Figure 5-2) for safety-related SSCs is provided, as necessary, to ensure that Unit 1 and Unit 2 can be safely shut down and/or maintained in a safe condition in the unlikely event of a load drop during use of these cranes for assembly/disassembly of the OLS.

Personnel involved in operating the mobile cranes will receive the following instruction:

- Operators will receive Sequoyah site-specific training as specified in Reference 9.
- Personnel will undergo hands on training with the equipment before a load is attached to the equipment.
- Prior to lifts over safety-related SSCs, detailed pre-lift meetings will be conducted.
- Direction to the operators will be given by technical representatives of the equipment owner, as required.

The mobile cranes will not be operated in high winds or weather conducive to tornadoes and will be relocated away from safety-related SSCs under these conditions. The mobile cranes are not designed to withstand seismic events.

Based on the above discussion, it is highly unlikely that a load will be dropped from a mobile crane. However, as required by NUREG-0612, load drops from a mobile crane has been postulated and the potential consequences of a postulated drop evaluated as described in Sections 7 and 8. None of these consequences lead to the need to invoke the one-time Operating License change or impose the ERCW compensatory measures during lifts by the mobile cranes.

5.3 Outside Lift System Components

The OLS will arrive at the Sequoyah site in standard containers. These containers will be moved to the OLS assembly/disassembly area (see Figure 5-2) on tractor-trailers. The OLS will be assembled/disassembled in accordance with Reference 7 while both units are in Modes 1-6 or defueled. As described in Reference 7, the heaviest individual component is the lower counterweight tray at 27.8 tons (55.6 kips). The heaviest

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assembled component lifted during the erection process is the main mast at 135 tons (270 kips). The largest ballast blocks used are 10.9 tons (21.8 kips).

The crane components will be off-loaded from the tractor-trailers using the lattice boom and truck cranes discussed in Section 5.2, and forklifts. During the offload process, the components will be lifted slightly higher than the trailer bed and lowered to the ground. Offloading locations will be picked to minimize the potential for impacting ERCW piping. When that is not possible, timber mats (as shown on Figure 5-2) will be used to distribute the impact from a load drop such that the ERCW piping will not be affected. None of these consequences lead to the need to invoke the one-time Operating License change or impose the ERCW compensatory measures during lifts by the mobile cranes.

5.4 Old & Replacement Steam Generators

The existing Westinghouse Model 51 OSGs will be removed and new RSGs furnished by CENP-Westinghouse will be installed. The RSGs are form, fit and function replacements of the OSGs and are similar in orientation and overall physical dimensions to the OSGs. The enveloping weight for the steam generator lifts has been determined to be 424.6 tons (849.2 kips). This enveloping weight includes the steam generator, rigging, attached upper lateral restraint, and attached insulation.

Movement of the OSGs/RSGs out of/into Containment will be performed with Unit 1 in the defueled condition and Unit 2 at power. Coordination with Operations is required prior to commencement of SG movement activities.

Once lifted clear of the Containment roof, the OSGs will follow designated load paths over the top of the Containment roof as shown on Figure 5-2. Rigging and lifting of the OSGs will be performed by trained personnel, will be strictly controlled and conducted in accordance with approved procedures, and will be restricted to the load paths described herein. Once the bottom of the OSG reaches a suitable height above the ground the OLS will rotate and move the OSG to the downending area where footpad attachments will be attached to the lower portion of the OSG. The OSGs will be maneuvered to a downending device designed to receive the footpad attachment and facilitate the downending operation. This downending device allows each OSG to be pivoted and downended directly onto its transport/storage saddles, which will be staged on the transporter. Downending equipment and the downending foundation area have been designed for the applicable loads in accordance with ASME NQA-1, Subpart 2.15. Reference 22 determined that the loads on the downending foundation are less than 150 tons (300 kips) and the soil bearing pressure from the foundation is less than the allowable pressure. Once the OSG has been set on the saddles located on the transporter, the footpad attachments will be removed with the assistance of a construction crane. Each OSG will be handled in an identical manner (but with slightly different load paths over the Containment roof) and the RSGs will be handled in a similar manner, but reverse order. However, the OLS attachments, footpad attachments, and saddles will be different for the RSGs.

5.5 Reactor Shield Building Concrete, Steam Generator Enclosure Concrete, and Containment Vessel Steel

Two holes (approximately 20 ft. by 45 ft.) will be created by cutting through the Shield Building dome and Containment vessel dome to allow removal of the OSGs and installation of the RSGs. Rigging of the Shield Building concrete and Containment

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vessel steel will occur only during the defueled condition. The OLS will be used to remove/replace the cut concrete and steel sections. The Containment vessel steel sections will weigh no more than 15 tons (30 kips). The Shield Building concrete sections will weigh less than 132.5 tons (265 kips).

The SG compartment roof and the Main Steam whip restraint beams below the roof will be cut and removed as one piece in each of the four compartments. The diameter of the openings is 18 to 20 feet. The cut sections of concrete from the SG compartments weigh less than 65 tons (130 kips). The OLS will be used to remove/replace the cut sections of concrete. Removal/replacement of the cut sections of concrete will take place during the defueled condition.

Movement of the above loads will be performed with Unit 1 in the defueled condition and Unit 2 at power. Coordination with Operations is required prior to commencement of heavy load movement activities.

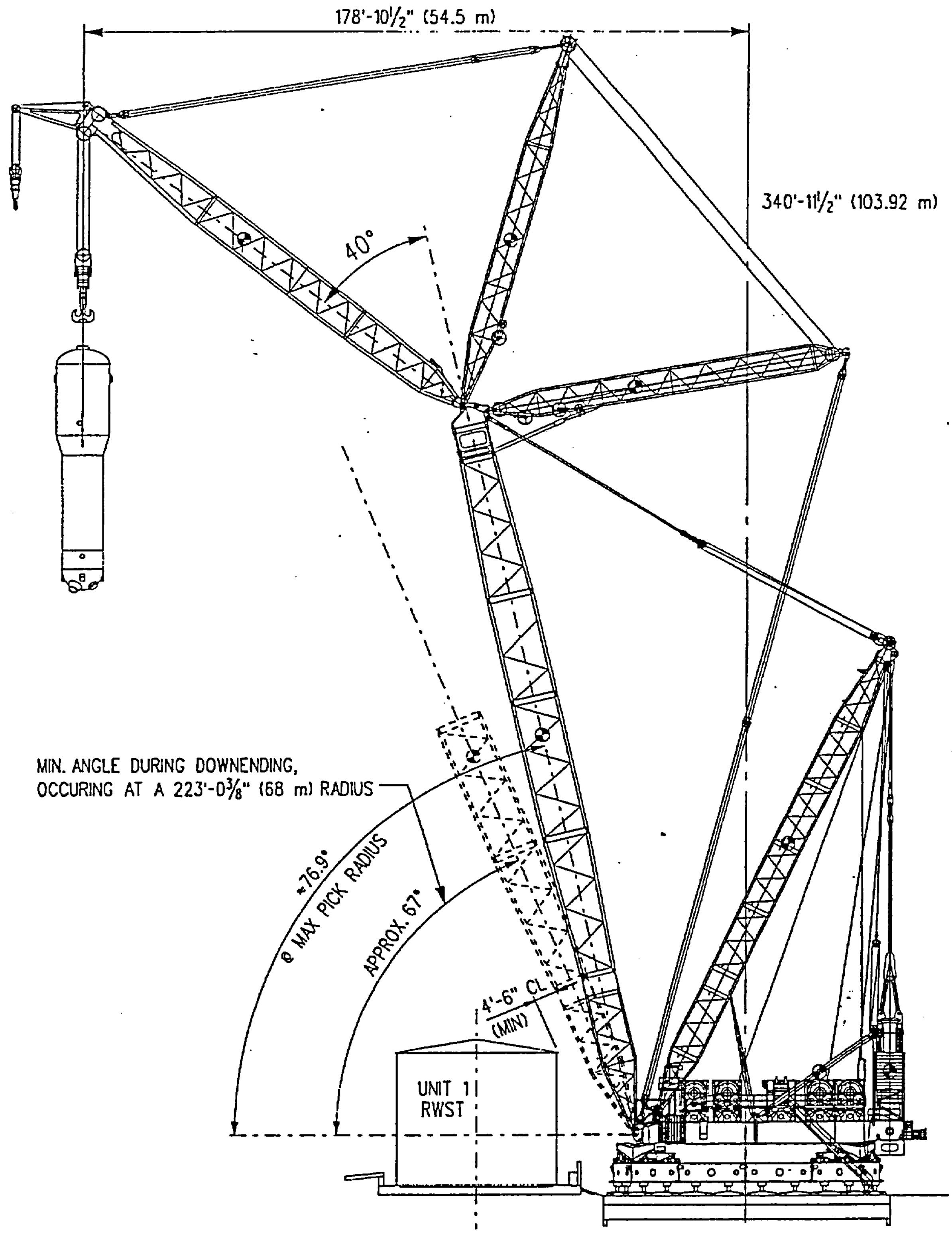

Figure 5-1 – Outside Lift System Elevation

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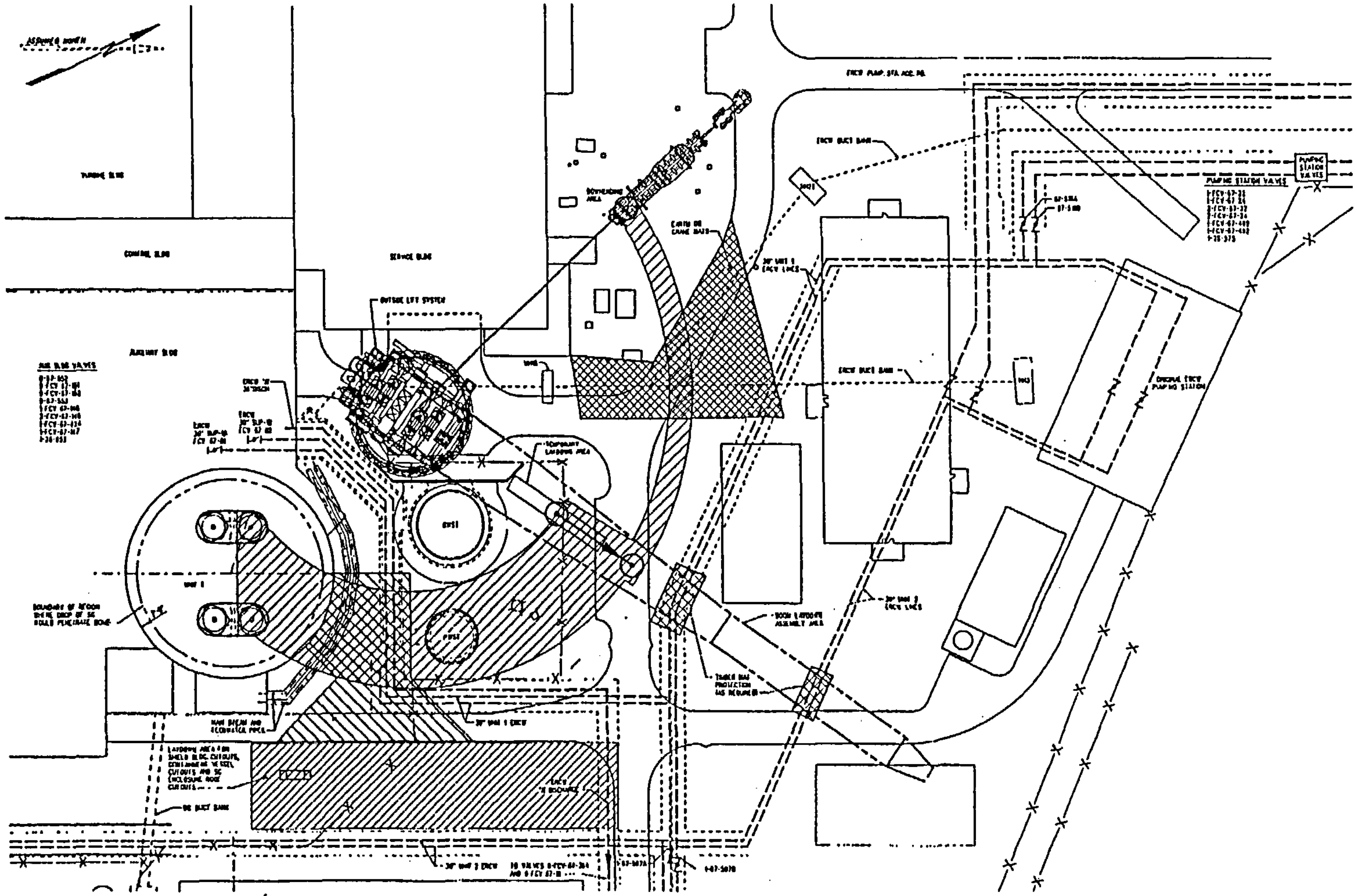

Figure 5-2 – Outside Lift System Location

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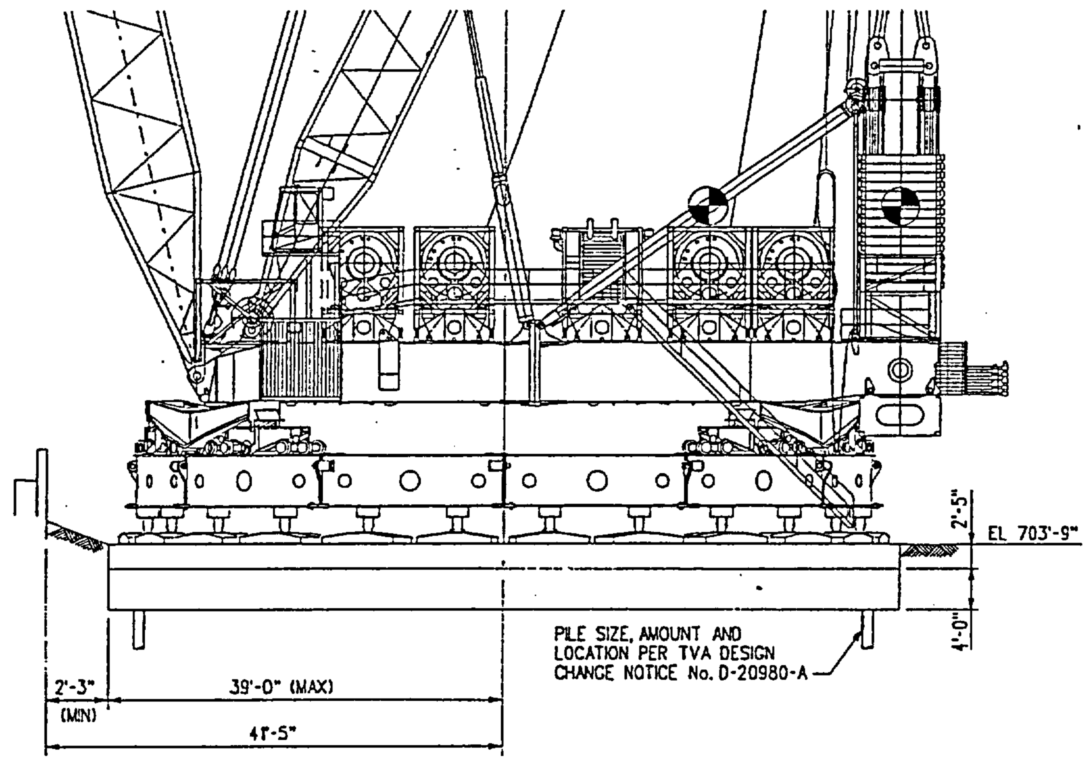

Figure 5-3 – Outside Lift System Base Elevation

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6.0 Description of SSCs Potentially Affected by a Postulated Load Drop

To support the Unit 1 SGR, movement of heavy loads in the vicinity of, and over, safety-related equipment required to support operation of both units is required in Modes 1 through 6 and while defueled. The SSCs that are potentially affected from either equipment impact or a heavy load drop impact are identified in this section. The potentially affected design functions and operability requirements of these SSCs are also addressed. As detailed in Sections 7 and 8, the movement of loads in the vicinity of and over these SSCs has been evaluated and found acceptable based on the capability of the SSC to withstand the impact, protection being provided, and/or compensatory measures being implemented.

6.1 Containment

The Sequoyah Unit 1 Containment consists of a free-standing Steel Containment Vessel (SCV) surrounded by a free-standing concrete Shield Building. The SCV and Shield Building are designed to Seismic Category I standards to remain functional during and after a SSE. The design function of the SCV, as indicated in UFSAR Sections 3.8.2.1, 6.1 and 6.2.4, is to provide an essentially leak-tight barrier to the release of fission products to the environment. As described in UFSAR Section 3.8.1.1, the Shield Building is a reinforced concrete structure. The design function of the Shield Building is to protect the SCV from external events and to act as the principal structure that limits doses from radioactivity inside the Containment. These design functions are not required while the reactor is defueled.

Unit 1 Technical Specifications (TSs) 3/4.6.1.1, 3/4.6.1.6, and 3/4.6.1.7 specify the integrity requirements for the SCV and Shield Building during Modes 1-4. The bases for TSs 3/4.6.1.1, 3/4.6.1.6, and 3/4.6.1.7 indicate that the safety design basis for Primary Containment is that the Containment must withstand the pressures and temperatures of the limiting design basis accident without exceeding design leakage rates.

Unit 1 TS 3/4.6.2.2 requires that two independent trains of lower Containment vent coolers be operable with two coolers in each train in Modes 1-4. The bases for TS 3/4.6.2.2 indicate that the operability of the lower Containment vent coolers ensures that adequate heat removal capacity is available to provide long-term cooling following a non-LOCA event.

Unit 1 TS 3/4.6.3 requires that each Containment isolation valve be operable in Modes 1-4. The bases for TS 3/4.6.3 indicate that operability of the Containment isolation valves ensures that the Containment atmosphere will be isolated from the outside environment in the event of a release of radioactive material to the Containment or pressurization of the Containment.

Unit 1 TS 3/4.9.4 defines the required status of Containment Building penetrations during movement of irradiated fuel within the Containment. The bases of TS 3/4.9.4 indicate that the requirements on Containment Building penetration closure and operability ensure that a release of radioactive material within Containment will be restricted from leakage to the environment.

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6.2 Auxiliary Building

The Auxiliary Building will not be directly impacted by the evaluated load drops. However, a potential effect from the postulated load drops is flooding of the Auxiliary Building through the ERCW tunnel. The impact of potential flooding on the Auxiliary Building is addressed in Section 7.1. Measures that mitigate this flooding are detailed in Section 8.

As described in UFSAR Section 3.8.4.1.1, the Auxiliary Building is a part of the Auxiliary Control Building. It is a multi-story reinforced concrete structure that provides housing for Unit 1 and 2 Engineered Safety Features equipment. The Spent Fuel Pit and Fuel Transfer Canal are also housed in the Auxiliary Building. The Auxiliary Building is designed to Seismic Category I standards and will remain functional during and after a SSE. The exterior concrete walls above grade are designed to resist the design basis tornado missiles. Since the Auxiliary Building is shared between Unit 1 and Unit 2, these design bases are required whenever either unit is in Modes 1-4 or fuel is stored in the Spent Fuel Pool.

6.3 Essential Raw Cooling Water System

As described in UFSAR Section 9.2.2.2, the ERCW system consists of eight pumps, four water traveling screens, four screen wash pumps, and four strainers located within the ERCW pumping station, and associated piping and valves. The safety-related portion of the ERCW system is designed to Seismic Category I standards and will remain functional following the SSE. Water is supplied to the Auxiliary Building from the ERCW pumping station through four independent sectionalized supply headers designated as 1A, 1B, 2A, and 2B. Four ERCW pumps are assigned to train A and four are assigned to train B. The two headers associated with the same train (i.e., 1A/2A or 1B/2B) may be cross-tied to provide greater flexibility. This allows one supply header to be out of service (e.g., for strainer maintenance), subject to the Ultimate Heat Sink limitations of TS 3/4.7.5. 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. A simplified flow diagram of the ERCW system is provided as Figure 6-1. Figure 6-1 also depicts the impact locations of the postulated load drop of an SG based on the load path indicated on Figure 5-2. Note that three ERCW lines run in parallel under the load path resulting in three impact locations on Figure 6-1.

The ERCW system piping is arranged in four headers (1A, 1B, 2A, and 2B) each serving certain components in each unit as follows:

1. Each header supplies ERCW to one of the two Containment Spray heat exchangers associated with each unit.
2. The primary cooling source for each of the Diesel Generator heat exchangers is from the Unit 1 headers. Each diesel also has an alternate supply from the Unit 2 headers of the opposite train.
3. The normal cooling water supply to Component Cooling System (CCS) heat exchangers 1A1 and 1A2, 2A1 and 2A2, and 0B1 and 0B2, is from ERCW headers 2A, 2A, and 2B, respectively.
4. Each A and B supply header in each unit header provides a backup source of Feedwater for the turbine-driven Auxiliary Feed Pumps in the respective unit.

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5. Each of the two discharge headers provides a backup source of Feedwater for the motor-driven Auxiliary Feedwater Pumps in each unit.
6. Headers 1A and 1B provide ERCW cooling water to the Control Room and Control Building electrical board room air-condition systems.
7. Each A and B header in each unit supplies ERCW cooling water to the Auxiliary Building ventilation coolers for safeguard equipment, the Containment ventilation system coolers, the Reactor Coolant Pump (RCP) motor coolers, the control rod drive vent coolers, and the Containment instrument room cooler's water chillers in the respective unit.
8. Headers 1A and 1B provide a normal and backup source of cooling water for the Station Air Compressors.
9. Headers 1A and 2B provide ERCW cooling water for the Shutdown Board room air-conditioners and Auxiliary Control Air Compressors.
10. Headers 2A and 2B provide ERCW cooling water for the Emergency Gas Treatment room coolers and boric acid transfer and Unit 2 Auxiliary Feedwater Pump space coolers.
11. Headers 1A and 1B provide ERCW cooling water for the CCS pumps and Unit 1 Auxiliary Feedwater Pump space coolers.
12. Under flood conditions, each header would provide water to the Spent Fuel Pit heat exchangers, Reactor Coolant Pump thermal barriers, ice machine refrigeration condensers, and sample heat exchangers, and the Residual Heat Removal heat exchangers as needed.

The headers are arranged and fitted with isolation valves such that a rupture in a header can be isolated and will not jeopardize the safety functions of the other headers. The layout of ERCW piping and key isolation valves relative to the heavy load paths is provided on Figure 5-2. The operation of two pumps on one plant train is sufficient to supply cooling water requirements for the 2-unit plant for unit cooldown, refueling, or post-accident operation. However, additional pumps may be started, if available, for unit cooldown or refueling. Two pumps per train operate during the hypothetical, combined accident and loss of normal power if each Diesel Generator is in operation. In an accident the Safety Injection signal automatically starts two pumps on each train, thus providing full redundancy. This arrangement assures adequate cooling water under both normal and emergency conditions.

TS 3/4.7.4 (both units have the same TS requirements) requires that at least two independent ERCW loops be operable in Modes 1-4. The bases of TS 3/4.7.4 indicate that the operability of the ERCW system ensures that sufficient cooling capacity is available for continued operation of safety-related equipment during normal and accident conditions. The Unit 1 systems that require ERCW are not required to be operable while the reactor is defueled.

6.4 Refueling Water Storage Tank

As discussed in UFSAR Section 3.8.4.1.4, the Refueling Water Storage Tank (RWST) is a Seismic Category I structure, but is not tornado missile protected. Pipes from the RWST to the Auxiliary Building are housed in reinforced concrete tunnels. A storage basin is provided around the RWST to retain a quantity of borated water in the event the RWST is ruptured by a tornado missile or other initiating event.

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The design function of the RWST, as indicated in UFSAR Sections 5.5.7.2.2, 6.3.2.2 and 6.3.3.12, is to provide borated water for (1) filling the Refueling Canal during refueling and (2) the Safety Injection, Residual Heat Removal, and Containment Spray pumps during the Emergency Core Cooling System (ECCS) function. These design functions are not required while the Reactor is defueled.

UFSAR Table 6.3.2-3 provides the minimum storage volume for the accumulators and RWST. As indicated in UFSAR Section 6.3.2.6, this minimum storage volume is sufficient to ensure that after a RCS break, sufficient water is injected and is available within the Containment to permit recirculation flow to the core, and to meet the net positive suction head requirements of the RHR pumps.

Unit 1 TS 3/4.5.5 requires the RWST to be operable in Modes 1-4. The bases for TS 3/4.5.5 indicate that the operability of the RWST as part of the ECCS ensures that a sufficient supply of borated water is available for injection by the ECCS in the event of a LOCA.

6.5 Primary Water Storage Tank

As indicated in UFSAR Section 12.1.2, the Primary Water Storage Tank (PWST) is one of the outside tanks used to store low-level radioactive liquids. It is a non-seismic, non-tornado missile protected, non-safety related tank. Section 11.2.3 of the UFSAR indicates that the PWST has a high level alarm and an overflow line that discharges to the ERCW pipe tunnel.

6.6 Main Steam Lines

UFSAR Section 10.3 describes the Main Steam supply system. The system is designed to conduct steam from the Steam Generator outlets to the High Pressure Turbine, the Condenser Steam Dump system, and to other components. Downstream of the Main Steam Isolation Valves (MSIVs), the steam lines follow the outside perimeter of the Shield Building until they enter the Turbine Building.

As described in UFSAR Section 10.3.2.1, the MSIVs and Main Steam Bypass Isolation Valves are provided to protect the plant following a break in the steam header downstream of the MSIVs. UFSAR Section 3.5.5 states that tornado missile protection is not required for the portion of the Main Steam piping downstream of the MSIVs.

Unit 1 TS 3/4.7.1.5 requires that four MSIVs be operable in Modes 1-3. The bases for TS 3/4.7.1.5 indicate that the operability of the MSIVs ensures that no more than one Steam Generator will blowdown in the event of a steam line rupture.

6.7 Feedwater Lines

As described in UFSAR Section 10.4.7.1, the Condensate Feedwater system is designed to supply a sufficient quantity of feedwater to the Steam Generator secondary side inlet during normal operating conditions and to guarantee that feedwater will not be delivered to the Steam Generators when feedwater isolation is required. The portion of the system from the Steam Generators back through the check valve and isolation valve is designed as TVA Class B.

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Unit 1 TS 3/4.7.1.6 requires that four Main Feedwater Isolation Valves (MFIVs), four Main Feedwater Regulating Valves (MFRVs), and four MFRV Bypass Valves be operable in Modes 1-3. The bases for TS 3/4.7.1.6 indicate that isolation of the Main Feedwater system is provided when required to mitigate the consequences of a steam line break, feedwater line break, excessive feedwater flow, and loss of normal feedwater (and station blackout) accident.

6.8 Fire Protection System Piping

Section 12.1 of Part II of the Sequoyah Fire Protection Report (FPR) indicates that the High Pressure Fire Protection (HPFP) system water supply is common to both units and consists of one electric motor driven fire pump.and one diesel engine driven fire pump. Each pump takes suction from its own 300,000 gallon potable water storage tank which is supplied by the local municipal utility. Each pump is connected to the HPFP system looped yard main by a separate supply line that can be isolated.

A fire protection water distribution system is provided to serve both units and is cross-tied between the units. Sectional isolation valves are provided so that maintenance may be performed on portions of the loop while maintaining fire fighting capability. The sectional isolation valves in the underground loop are locked or sealed in position and surveillance is performed to ensure proper system alignment.

The HPFP system is also connected to the two fire/flood mode pumps (old fire pumps) which can be utilized by opening the normally closed valves which isolate them from the system. These pumps are not required for the HPFP system to fulfill its design bases.

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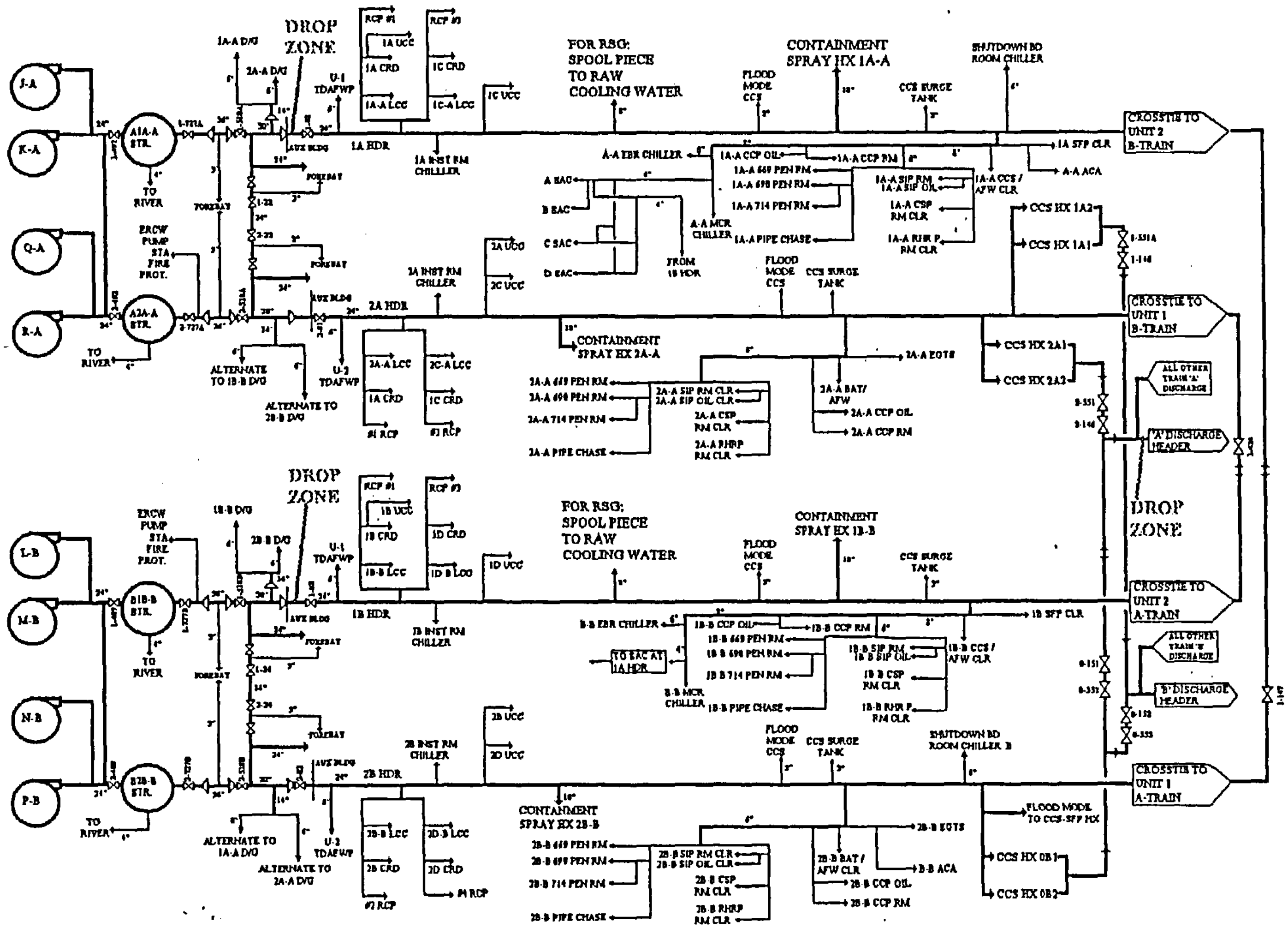

Figure 6-1 – Simplified ERCW Flow Diagram

7.0 Postulated Load Drops

Details of the design of the cranes being used (including their seismic capability), inspections and load testing performed on these cranes, restrictions on operation of the cranes, operator training, and procedural controls have been provided in the previous sections. Given these considerations, it is highly unlikely that a load will be dropped from these cranes. However, as required by NUREG-0612, load drops from each of these cranes have been postulated and the potential consequences of these postulated drops evaluated.

Rigging of the heavy loads described in Section 5 will be performed within the load paths defined on Figure 5-2. In the event of a non-mechanistic failure of a crane or rigging equipment resulting in a load handling accident, the load is assumed to impact within the evaluated load path.

As detailed in Section 5, the heaviest loads being handled are the Steam Generators. Other significant loads include Shield Building concrete sections, Containment Vessel steel sections, Steam Generator Compartment roof concrete sections, and OLS components during assembly/disassembly.

7.1 Steam Generator Load Drops

<u>SG Drop Above Containment</u>

Two SG load drop situations above the Containment have been considered; those within a radial distance from the center of Containment of about 60 ft. (remote from Containment Building ~131 ft. diameter cylindrical shell wall) and those between this region and the parapet (near the cylindrical shell wall). Since the Unit 1 Reactor will be defueled while the SGs are being moved, the primary concern with a SG drop is the SG trajectory following impact with the Shield Building dome and its subsequent impact location.

SG Drop Above Containment – Away From Shield Building Wall

The SG drop trajectory following vertical impact from an arbitrary height onto the dome is difficult to predict. Since the lift height of the SG is only limited by the capability of the OLS, a substantial clearance between the SG and the Shield Building dome will be maintained by lifting the SGs vertically through the Containment openings until a 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. Applying an energy balance methodology to a rigid-plastic shell model, it was analytically determined (Reference 26) that a SG drop from a height of 12.75 ft. or greater will perforate the concrete Containment shield wall and 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. 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. Some substantial conservatisms support the conclusion that perforation and entry will occur. These conservatisms are: 1) neglect of energies associated with local deformations, 2) consideration of the "laminar" concrete dome as a contiguous or single layer, 3) neglect of the weakening effect of the openings, and 4) use of a lift height (20 ft.) that is 50% higher than that calculated for perforating the dome.

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SG Drop Above Containment – Near Shield Building Wall

As the SGs near the edge of the Containment, it no longer becomes possible to analytically show that the SG penetrates the Containment dome. At this point, a dropped SG is assumed to tumble over the edge of the Containment and impact the ground somewhere near the Shield Building wall along the load path. It may also impact the side of the Shield Building as it falls. Since it is difficult to predict exactly where the SG will impact, SSCs within and near the load path were assumed to be affected. The potentially affected SSCs in the vicinity of this postulated drop location are the Unit 1 Shield Building, ERCW tunnel and pipes, RWST, PWST, MS piping, FW piping, and Fire Protection System piping.

SG Drop Along Shield Building Wall

The SGs will be lowered/raised by the OLS near the Shield Building wall above the load path shown on Figure 5-2. A SG drop in this area is assumed to impact directly below where the SG is being lowered/raised. Since the impact area is bounded by the area assumed for the postulated SG drop above the Containment near the Shield Building wall, the consequences of the drop along the Shield Building wall are also bounded.

SG Drop Between Lowering/Raising Area and Downending/Upending Area

A SG drop along the load path between the lowering/raising area and the downending/upending area is assumed to impact SSCs within the flopover distance (approximately 70 ft. from the impact point on the load path) of the SG. In addition to those SSCs potentially affected by the SG drop above Containment near the Shield Building wall, this postulated drop could also affect the two ERCW ductbanks shown on Figure 5-2. These ductbanks contain ERCW cables associated with trains A and B of both units.

SG Drop Dose Consequences

Since it is more conservative from a dose standpoint to assume a failure of the OSG outside Containment, Reference 23 determined the radiological consequences of a Steam Generator drop outside Containment along the load path between the Containment and the OSGSF.

The acceptability of the offsite dose consequences associated with a postulated drop of an OSG has been evaluated and compared to the consequences of postulated design basis accidents for a gaseous release. For assessing offsite dose consequences, the drop of an OSG is considered to most closely resemble a rupture of a tank containing radioactive material. Failure of the Waste Gas Decay Tank (WGDT) (Reference UFSAR Section 15.3.5) is the limiting event currently evaluated in the UFSAR for accidental gaseous release from a tank. As indicated in UFSAR Section 15.5.2, the gamma, beta, and thyroid doses at the EAB from a WGDT failure are 2.5 Rem, 5.8 Rem, and 5.9×10^{-2} Rem, respectively. The gamma, beta, and thyroid doses at the LPZ are 0.29 Rem, 0.68 Rem, and 6.9×10^{-3} Rem, respectively.

Reference 23 conservatively assumed that 10% of the Steam Generator activity is released due to the impact of the drop and 1% of this release amount is in the form of particulates small enough to become airborne. Confirmatory NRC analyses of the early

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SGRs also used this percentage of activity release. Based on an isotopic survey of the CVCS, the prime contributors to the offsite dose due to a SG drop were determined to be Ni-63, Co-60, Cs-134 and Cs-137. Using these conservative assumptions, the maximum calculated Control Room dose is 3.76×10^{-2} Rem whole body. The offsite doses from a postulated drop at the limiting location along the haul route are 4.86×10^{-2} Rem whole body (correlates to the WGDT gamma dose) and 3.02×10^{-4} Rem to the skin (correlates to the WGDT beta dose) at the EAB and 4.63×10^{-3} Rem whole body (correlates to the WGDT gamma dose) and 1.3×10^{-3} Rem to the skin (correlates to the WGDT beta dose) at the LPZ. A thyroid dose was not calculated since the SG dose is primarily due to activated corrosion products and contains no iodine.

UFSAR Section 15.5.2 presents the radiological consequences of a WGDT rupture in the context of 10CFR100. However, in NRC Standard Review Plan (SRP) Section 11.3, the WGDT radiological consequences are limited per the guidance of Branch Technical Position (BTP) ETSB 11-5. BTP ETSB 11-5 establishes an offsite dose limit of 0.5 Rem whole body which at the time of issuance was consistent with 10CFR20 limits. The Technical Specifications acknowledge this regulatory criterion by placing an activity limit on the WGDTs (Reference Technical Specification 3/4.11.2.6 and the associated bases) to ensure the whole body exposure of 0.5 Rem to an individual in an unrestricted area is not exceeded. This limit on dose is greater than the calculated dose for an OSG drop. The evaluated consequences of an OSG drop are within the applicable regulatory criteria of BTP ETSB 11-5 and are much less than the limiting licensing design basis accidents currently evaluated in Chapter 15 of the UFSAR.

<u>Auxiliary Building Flooding</u>

As indicated above, a postulated OLS load drop could affect the ERCW tunnel and pipes, RWST, PWST, and Fire Protection System piping. The failure of any of these tanks and pipes could result in flooding of the Auxiliary Building via 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. 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. Compensatory measures to preclude flooding of safety-related equipment in the Auxiliary Building following a postulated heavy load drop are described in Section 8.

7.2 Shield Building Concrete Section Load Drops

As indicated in Section 5.5, the Shield Building concrete sections will be approximately 20 ft. by 45 ft. and will weigh less than 132.5 tons (265 kips). These sections will follow the load paths shown on Figure 5-2. Unlike the SGs, they will only be raised a maximum of three feet above the Containment dome. This lift height and the inherent shape of the concrete sections will eliminate the potential for them to rebound from the Containment in an unanticipated direction. Given that the size and mass of these concrete sections are bounded by the SGs, the consequences of a Shield Building concrete section load drop are bounded by the SG drops described in Section 7.1.

7.3 Containment Vessel Steel Section Load Drops

As indicated in Section 5.5, the Containment Vessel steel sections will be approximately 20 ft. by 45 ft. and weigh no more than 15 tons (30 kips). These sections will follow the

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load paths shown on Figure 5-2. Unlike the SGs, they will only be raised a maximum of three feet above the Containment dome. This lift height and the inherent shape of the SCV sections will eliminate the potential for them to rebound from the Containment in an unanticipated direction. Given that the size and mass of these steel sections are bounded by the SGs, the consequences of a Containment Vessel steel section load drop are bounded by the SG drops described in Section 7.1.

7.4 Steam Generator Compartment Roof Plug Load Drops

As indicated in Section 5.5, the Steam Generator Compartment roof concrete sections will be 18-20 ft. in diameter and will weigh less than 65 tons (130 kips). These sections will follow the load paths shown on Figure 5-2. Unlike the SGs, they will only be raised a maximum of three feet above the Containment dome. This lift height and the inherent shape of the concrete sections will eliminate the potential for them to rebound from the Containment in an unanticipated direction. Given that the size and mass of these concrete sections are bounded by the SGs, the consequences of a Steam Generator Compartment roof plug concrete section load drop are bounded by the SG drops described in Section 7.1.

7.5 Outside Lift System Component Load Drops

As indicated in Section 5.3, the OLS components vary in size and weight. These components will be handled in the OLS assembly/disassembly area shown on Figure 5-2. The crane components will be off-loaded from the tractor-trailers using lattice boom and/or truck cranes and forklifts. During the offload process, the components will be lifted slightly higher than the trailer bed and lowered to the ground. Offloading locations used will minimize the potential for impacting the RWST and ERCW piping. When it is not possible to eliminate a potential impact with the ERCW piping, timber mats (as shown on Figure 5-2) will be used to distribute the impact from a load drop. The consequences of OLS component load drops have been evaluated to be acceptable based on provision of this protection.

8.0 Heavy Load Drop Protection Plans/Compensatory Measures

Section 4 details the regulatory requirements/criteria that are relevant to the handling of heavy loads over safety-related equipment and summarizes conformance with these requirements/criteria. As discussed in Section 4.2, Section 5.1.5 of NUREG-0612 indicates that the effects of load drops should be analyzed (in accordance with the guidelines of Appendix A to NUREG-0612) and the results should indicate that damage to safe shutdown equipment is not sufficient to preclude safe shutdown.

Each of the potentially affected SSCs identified in Section 6 has been analyzed in accordance with the NUREG-0612 guidance to determine the effects of a load drop. Summarized below is the protection required to preclude an adverse effect and/or the actions or compensatory measures required to mitigate these effects should a load drop occur. Provision of the identified protection and taking the specified actions and compensatory measures assures that safe shutdown can be achieved following a load drop. In addition, it will be confirmed that the assumptions made within this Topical Report regarding the status of the station are valid prior to load handling activities.

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8.1 Containment

The heavy loads of concern that will be handled above the Containment will only be moved while the Unit 1 reactor is defueled. With fuel removed from the Containment, the only other safety issue is whether a load drop into the Unit 1 Containment will affect systems common to both units that pass through the Unit 1 Containment. To preclude a SG drop inside the Unit 1 Containment from affecting Unit 2, the ERCW system and Component Cooling System (CCS) shall either be isolated or be capable of being isolated with valves located outside of Containment. In addition, the Spent Fuel Pool (SFP) shall be isolated from the Unit 1 containment.

8.2 Auxiliary Building

Heavy loads will not be handled over the Auxiliary Building and, as discussed in Sections 7.1, 7.2, 7.3, and 7.4, will not roll off the Containment roof onto the Auxiliary Building. Therefore, no additional protection of the Auxiliary Building roof is required.

To preclude flooding of the Auxiliary Building due to a heavy load drop a wall (see Figure 8-1) will be installed in the ERCW tunnel near the Auxiliary Building interface. A door will be provided as part of the wall to allow access to the tunnel, if required. 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. 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.

8.3 Essential Raw Cooling Water System

<u>Unit 1 ERCW Supply Piping and Train A Discharge Piping</u>

As noted in Section 6.3, 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 Unit 1 ERCW supply piping between the ERCW pumps and the various heat exchangers, as well as the Train A discharge piping from the various heat exchangers returning to the Ultimate Heat Sink, located near the Unit 1 Containment might fail (i.e., crimp or rupture, or both) as a result of the postulated load drops detailed in Sections 7.1, 7.2, 7.3, and 7.4.

Following pipe damage from a postulated heavy load drop from the OLS, all of the ERCW loads that are direct Unit 2 safe shutdown components will continue to receive their design flow rates without the need for any actions. Certain components that receive flow from the 1A and 1B ERCW supply headers are indirect Unit 2 safe shutdown components, and may not receive their design flow rates. The following listing is the indirect Unit 2 safe shutdown components that might not receive their design flow rates following a postulated heavy load drop:

- Emergency Diesel Generators
- Train A auxiliary air compressor
- Main control room chillers
- Electrical board room chillers
- Component cooling system pump space coolers
- Train A 6.9 kV shutdown board room chiller

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- Cooling to auxiliary building gas treatment system

All of the components in the above listing can have their function restored.

- Diesel Generators - The Diesel Generator cooling function may be restored by opening the alternate supply valves. These valves supply cooling water to the Diesel Generators from the opposite train, i.e., the Train A Diesel Generators receive their alternate supply from the B train ERCW.

- All of the remaining components can have their function restored by closing the valve downstream of the pipe damage area, and opening the valves that intertie the A and B ERCW supply headers in the Auxiliary Building. The required action consists of closing one valve and opening one valve. These valves are motor operated valves that can be operated remotely or locally. Alternatively, one train of these components can be restored by connection of a spool piece to allow the non-safety related Raw Cooling Water system to supply one of the ERCW supply headers.

An analysis has been performed that demonstrates that all Unit 2 safe shutdown components receive their design flow assuming no isolation of the failed 1A and 1B ERCW supply headers from the upstream side, and with total flow blockage of the 'A' ERCW return header, including the use of the alternate supply lines detailed above.

Mitigation of this assumed ERCW piping failure following a postulated heavy load drop from the OLS requires that compensatory measures be implemented to ensure that certain prerequisite actions are performed in order to support the analysis assumptions. These actions, as well as the actions required to restore equipment functionality, will be proceduralized prior to use of the OLS for load handling. Personnel will be trained to implement the compensatory measures.

Due to the potential to adversely affect both trains of ERCW, an operability issue has been identified that requires an amendment to the Unit 2 Operating License. A one-time condition for conduct of heavy load lifts associated with the Unit 1 steam generator replacement has been requested based on performance of heavy load lifts in accordance with NRC Bulletin 96-02 and adherence to the compensatory measures contained in this topical report.

Unit 2 ERCW Supply Piping

The Unit 2 30" ERCW pipes running parallel to the west side of the Solar Building and east of the Unit 1 Containment (see Figure 5-2) do not directly lie on the load path and are located approximately 128 ft. from the load path. They were evaluated in Reference 24 for the effects of impact energy due to a postulated drop of the SG at a distance away and transmitted to it by wave propagation through the soil. The worse case postulated impact location was determined to be located at least 63 ft. away from the nearest Unit 2 ERCW pipe. The peak particle velocity (PPV) of the shock wave at the ERCW piping from a load drop was determined using the scaled-distance wave propagation equation proposed by Wiss in Reference 14. The computed PPV was then used to estimate the free field soil pressure on the buried piping, which was then used to evaluate the adequacy of the ERCW pipe as a flexible pipe. Reference 24 concluded that the Unit 2 ERCW piping will not fail and will remain functional under the impact effects of the postulated SG drop at a distance away from the piping.

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ERCW Ductbanks

As noted in Section 7.1, the load path for the SGs crosses over two buried ERCW ductbanks (one between manhole MH12 and handhole HH3 and the other between manhole MH12 and handhole HH29). The ductbanks contain cables associated with ERCW trains A and B for both units. Therefore, it is vital that a SG drop does not affect the functioning of these cables. These ductbanks have been evaluated in Reference 24 for impact loading from a direct vertical drop of the SG as well as from the subsequent flopover fall of the SG. In order to minimize the impact energy from a vertical drop of the SG, the bottom of the SGs will be carried at an elevation not to exceed 3 ft. above grade while traversing the load path at and near these ductbanks. The impact energy from the flopover fall was determined to be more critical than from a direct vertical drop of 3 ft.

In evaluating the ductbanks, the depth of penetration of the dropped SG into the soil and the resulting contact-pressure time history were estimated considering the bearing resistance of the soil stratum overlaying the ductbank using Meyerhoff's Bearing Capacity equations (Reference 25). Suitable attenuation of the surface pressures were considered based on Boussinesq's equation (Reference 25). The ductbanks were analyzed dynamically as beams on elastic foundation subjected to the attenuated pressure time-history. The ductbank loading and boundary conditions are appropriately specified. The total response of the ductbank was calculated using modal superposition in terms of deflection, shear and bending moment based on which the adequacy of the ductbank is assessed.

The evaluation in Reference 24 concluded that the ERCW ductbanks will remain adequate in the event of an SG drop if sufficient soil cover is available over the ductbanks. Therefore, as shown on Figure 5-2, additional earth fill and/or crane mat protection will be provided in the potentially affected areas above the ductbanks where the grade elevation is lower so as to bring the grade to a sufficient height to protect the ductbanks.

8.4 Refueling Water Storage Tank

As noted in Section 6.4, the RWST is a Seismic Category I structure, but is not tornado missile protected. Pipes from the RWST to the Auxiliary Building are housed in reinforced concrete tunnels. A storage basin is provided around the tank to retain a quantity of borated water in the event the tank is ruptured by a tornado missile or other initiating event.

As shown on Figure 5-2, no heavy loads will be carried over the RWST by the OLS. Since a potential load drop from the OLS could only occur when Unit 1 is defueled, loss of the RWST function has no safety impact. However, a failure of the RWST piping in the pipe tunnel between the RWST and the Auxiliary Building could result in flooding in the Auxiliary Building. The passive sump in the Auxiliary Building has been sized to account for flooding from the RWST, but not concurrent with an ERCW piping failure in the pipe tunnel. To minimize the potential for flooding of the Auxiliary Building due to a failure of the RWST, PWST, and/or ERCW piping inside the pipe tunnel, a wall will be installed near the pipe tunnel opening into the Auxiliary Building. This wall will be installed prior to movement of loads with the OLS and will be capable of withstanding the hydrostatic and velocity head of water from the postulated piping failures and loads created by the nearby drop of a steam generator. It will also meet Sequoyah Seismic I

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(L) requirements, so that an earthquake would not cause a failure of nearby safety-related SSCs as a result of a seismically-induced failure of the wall.

The mobile cranes used for assembly/disassembly of the OLS will be positioned such that a load drop will not impact the RWST. Since the mobile cranes will be used while Unit 1 is in Modes 1-6, the positioning of these cranes away from the RWST assures that the RWST function will be available, if required.

8.5 Primary Water Storage Tank

As noted in Section 6.5 and shown on Figure 5-2, heavy loads will be carried over the PWST by the OLS. Since a potential load drop from the OLS could only occur when Unit 1 is defueled, loss of the PWST function has no safety impact. However, a failure of the PWST piping in the pipe tunnel between the PWST and the Auxiliary Building could result in flooding in the Auxiliary Building. To minimize the potential for flooding of the Auxiliary Building due to a failure of the RWST, PWST, and/or ERCW piping inside the pipe tunnel, a wall will be installed near the pipe tunnel opening into the Auxiliary Building. This wall will be installed prior to movement of loads with the OLS and will be capable of withstanding the hydrostatic and velocity head of water from the postulated piping failures and loads created by the nearby drop of a steam generator. It will also meet Sequoyah Seismic I (L) requirements, so that an earthquake would not cause a failure of nearby safety-related SSCs as a result of a seismically-induced failure of the wall.

8.6 Main Steam Piping

As noted in Section 6.6, the MS piping outside the Shield Building is a potentially affected SSC for the postulated load drops described in Section 7. Since a heavy load drop induced failure of the MS piping will be isolated by closure of the MSIVs, no protective measures are required.

8.7 Feedwater Piping

As noted in Section 6.7, the FW piping outside the Shield Building is a potentially affected SSC for the postulated load drops described in Section 7. Since a heavy load drop induced failure of the FW piping will be isolated by closure of the FW isolation valves, no protective measures are required.

8.8 Fire Protection System Piping

As noted in Section 6.8, the high-pressure fire pump and flood mode pump piping in the pipe tunnel is a potentially affected SSC for the postulated load drops described in Section 7. To minimize the impact of a rupture of this piping on flooding of the pipe tunnel, valves 1-26-575 and 1-26-653 will be closed prior to movement of heavy loads with the OLS. Closure of these valves minimizes the actions that need to be taken to isolate a break. Closing these valves will not affect plant operation.

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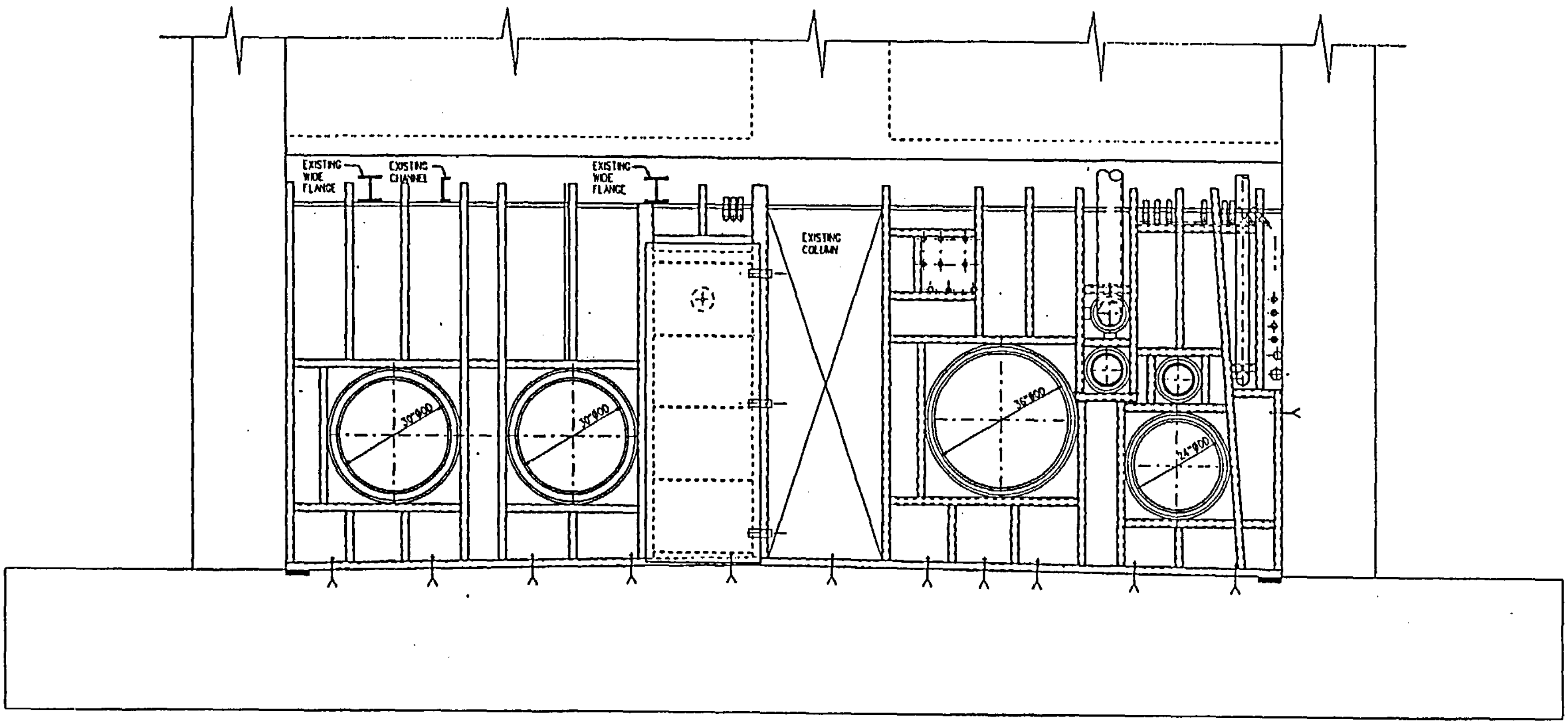

Figure 8-1 – ERCW Tunnel Wall

9.0 Summary and Conclusions

The Steam Generator Replacement at Sequoyah Unit 1 will involve the handling of heavy loads that are larger and must travel along load paths different from those evaluated during the original licensing of the plant. Paralleling the guidelines of NUREG-0612, a safe load path has been selected which generally moves the loads away from the plant and away from sensitive SSCs supporting the continued safe operation of the station. In a few cases, handling over equipment supporting safe shutdown could not be avoided. Therefore, the continued safety of the plant will be assured by:

- Equipment selection,
- Equipment evaluation for certain external events,
- Operator training, and
- Procedural controls, including lift heights, load paths, and limitations related to weather conditions.

Due to the potential to adversely affect both trains of ERCW, an operability issue has been identified that requires an amendment to the Unit 2 Operating License. A one-time condition for conduct of heavy load lifts associated with the Unit 1 steam generator replacement has been requested.

Based upon these considerations and the relatively short periods of time that loads will be suspended over safe shutdown equipment, the risk associated with the drop of a heavy load as discussed in this Topical Report is considered to be small. However, as further protection from the postulated load drop: 1) protection will be provided from secondary flooding effects that could occur as a result of the postulated load drop, and 2) compensatory measures that will be implemented in the event of a load drop have been developed and will be proceduralized for use during the SGRO. These measures provide assurance that the operating unit can be safely shut down in the event of a heavy load drop. Further, as concluded in Appendix B, these compensatory measures and proposed Operating License change do not involve a significant hazards consideration.

10.0 References

1. Sequoyah Updated Final Safety Analysis Report, Amendment 16.
2. Sequoyah Nuclear Plant Unit 1 Technical Specifications
3. Sequoyah Nuclear Plant Unit 2 Technical Specifications
4. Design Criteria Document No. SQN-DC-V-7.4, "Essential Raw Cooling Water System", Rev. 19.
5. System Operating Instruction No. 0-SO-67-1, "Essential Raw Cooling Water", Rev. 38.
6. Abnormal Operating Procedure AOP-M.01, "Loss of Essential Raw Cooling Water", Rev. 4.
7. Bechtel Supplier Document 24370-SC-004-PTCManual-001, "Users Manual – Platform Twin-Ring Containerized Crane", Rev. 0.
8. Maintenance Instruction 0-MI-MXX-000-026.0, "Control of Heavy Loads in Critical Lifting Zones, NUREG-0612, C.1", Rev. 8.
9. Procedure MMDP-2, "Safe Practices for Operation of Overhead Handling Equipment", Rev. 1.
10. ASME NQA-1, Subpart 2.15, "Quality Assurance Requirements for Hoisting, Rigging, and Transporting of Items for Nuclear Power Plants", 1997 Edition.

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11. ASME B30.5, "Mobile and Locomotive Cranes", 1994 and 2000 Editions.
12. NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants", July 1980.
13. NRC Bulletin 96-02, "Movement of Heavy Loads Over Spent Fuel, Over Fuel in the Reactor Core, or Over Safety-Related Equipment", April 11, 1996.
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15. 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.
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17. Moser, A.P., Buried Pipe Design, McGraw Hill Inc., 1990.
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19. NRC Generic Letter 81-07, Control of Heavy Loads, February 3, 1981.
20. NRC Regulatory Issue Summary 2001-03, Changes, Tests, and Experiments, January 23, 2001.
21. Calculation 24370-C-026, "Evaluation of PTC Crane for Seismic and Wind/Tornado Loads", Revision 0.
22. Calculation 24370-C-039, "Foundation for Upending/Downending Device", Revision 0.
23. Calculation 24370-M-002, "Old Steam Generator Drop Dose Analysis", Revision 0.
24. Calculation 24370-C-025, "Evaluation of Safety-Related Buried Commodities in the Vicinity of Heavy Lift Load Path for Postulated Load Drop from OLS", Revision 1.
25. Bowles, J.E., Foundation Analysis and Design, Fourth Edition, McGraw Hill, Inc., 1988.
26. Calculation 24370-C-022, "Evaluation of SG Drop on Containment Shell", Revision 0.
27. Calculation CSG-87-018, "5% Damped Free Field Top of Soil Response Spectra, SQN Units 1 & 2", Revision 0.

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Appendix A
NRC Commitments

There are a number of actions required to support the conclusions of Topical Report 24370-TR-C-002. The below listed actions ensure prerequisite actions to heavy load movement, active monitoring during heavy load movement, and protective actions in response to the unlikely event of a heavy load drop are in place. These actions are NRC commitments as listed below:

Prerequisite Actions to Heavy Load Movement

1. Install temporary pressure and flow gauges in selected locations of the Unit 1 ERCW piping.
2. Install a wall in the Unit 1 pipe tunnel to seal the tunnel from the Auxiliary Building. Develop criteria to quantify the amount of water behind the temporary pipe tunnel wall.
3. Realign the ERCW system to minimize operator actions in the event of a heavy load drop.
4. Isolate the high-pressure fire pump and the flood mode pump piping in the pipe tunnel to the Auxiliary Building.
5. Isolate systems shared 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.
6. Ensure that measures are in place to suitably handle any leakage through the temporary Unit 1 pipe tunnel wall.

Active Monitoring Actions During Heavy Load Movement

1. Monitor weather conditions, for the expected duration of the lift, to ensure conditions are acceptable for outside lift system operation.
2. Monitor outside lift system operation to ensure a minimum clearance of 20 feet exists between the Shield Building dome and the bottom of the steam generator when a steam generator is being moved over the Shield Building.

Actions in Response to the Unlikely Event of a Heavy Load Drop

1. Develop and issue plant procedure(s) to delineate specific actions required in case of a heavy load drop.

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Appendix B
No Significant Hazards Consideration Determination

I. DESCRIPTION OF THE PROPOSED CHANGE

The four steam generators of the Sequoyah Nuclear Plant Unit 1 will be replaced during the spring of 2003. To support the replacement of the old steam generators (OSGs) with the replacement steam generators (RSGs), several heavy loads will be moved over safety-related plant structures, systems, and components (SSCs). While many of these SSCs could be called upon to perform a safety function during the subject lifts, only the ERCW system is a safety-related system that is common to both units. During the Unit 1 Steam Generator Replacement Outage (SGRO), the ERCW system will be supporting continuous operation and safe-shutdown capability for Unit 2.

Mitigation of the assumed ERCW piping failures following a postulated heavy load drop requires that compensatory measures be implemented to isolate the affected ERCW piping and restore ERCW flow to required equipment, as necessary. Due to the potential to adversely affect both trains of ERCW as a result of a postulated load drop, an operability issue has been identified that requires an amendment to the Unit 2 Operating License. A one-time condition for conduct of heavy load lifts associated with the Unit 1 steam generator replacement has been requested based on performance of heavy load lifts in accordance with NRC Bulletin 96-02 and adherence to the compensatory measures contained in Topical Report 24370-TR-C-002.

II. REASON FOR THE PROPOSED CHANGE

As defined in NRC Bulletin 96-02, licensees planning to perform activities involving the handling of heavy loads over safety-related equipment while the reactor is at power and involving a potential load drop accident that has not previously been addressed in the FSAR should submit a license amendment request to the NRC. Following recent revisions to 10CFR50.59, the Bulletin's guidance was supplemented by NRC Regulatory Issue Summary 2001-03, which states, "The fact that the load is larger or is moving in a different load path than previously evaluated would enter into the risk assessment required by 10CFR50.65(a)(4) and determine under what plant conditions the load lift should occur." The Sequoyah issues of interest are, perhaps, unique with respect to that guidance, in that during the Unit 1 SGRO, Unit 1-related maintenance/heavy load activities must be considered in light of their potential to influence the operation of Unit 2.

TVA Topical Report 24370-TR-C-002 documents the provisions made to ensure that heavy load handling activities associated with the Unit 1 SGRO can be accomplished without impacting the safe operation of Unit 2. These provisions support the risk assessment required by 10CFR50.65(a)(4) and an application for a one-time Unit 2 license amendment associated with the operability of the ERCW System.

III. SAFETY ANALYSIS

The Outside Lift System (OLS) that will be used to move the OSGs and RSGs during the Sequoyah Unit 1 SGRO (i.e., Mammoet PTC Heavy Lift Crane) is a commercial design. The OLS was not specifically designed to withstand the external events addressed by 10CFR50, Appendix A, General Design Criterion (GDC) 2 that are a part of the Sequoyah design and licensing basis. However, due to the size of the OLS and

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because of the OLS location and proximity to the Containment, Auxiliary Building, Essential Raw Cooling Water (ERCW) piping, Refueling Water Storage Tank (RWST), Main Steam (MS) piping, and Feedwater (FW) piping, the OLS was evaluated for those external events that might cause it to collapse when these SSCs are required to be operable.

The OLS meets or exceeds ASME NQA-1 Subpart 2.15 design requirements, and its load charts and operating restrictions consider applicable dead, live, wind, impact, and out-of-plumb lift loads. The OLS, supplied with standard load charts for its various boom configurations, has a rated load capacity certified by the manufacturer and has been load tested during its production; this meets the load test requirements of ASME NQA-1, 1997 Edition, Subpart 2.15, Section 601.2. In addition, after the OLS has been erected, it will be load tested by lifting a 275 ton (550 kip) test load assembly with the OLS boomed out to a radius where the test load represents 110% of the OLS rated capacity at this radius.

The OLS attachments and rigging meet the requirements of ASME NQA-1-1997, Subpart 2.15 and the applicable ASME B30 series standards. The attachments and rigging used to attach the OLS to the SGs have been previously load tested in accordance ASME NQA-1, Subpart 2.15 or have a previous load history that exceeds the loads to be lifted. Rigging will be inspected prior to use in accordance with approved procedures and rigging operations will be controlled and conducted by highly trained and qualified personnel in accordance with approved procedures.

Personnel involved in operating the OLS will receive the following instruction:

- Operators will receive the applicable Sequoyah site-specific training specified in Appendix C of MMDP-2, "Safe Practices for Overhead Handling Equipment" (Reference 9).
- Personnel will undergo hands on training with the equipment before a load is attached to the equipment.
- Prior to a lift, detailed pre-lift meetings will be conducted.
- Direction to the operators during each OLS lift will be given by technical representatives of the equipment owner and the SGR contractor's rigging specialist.

During the lifting operation, the exact location of boom tip and load block will be monitored by two independent methods. Instrumentation internal to the crane provides continuous readout of crane and boom orientation and the location of the boom tip and load block. In addition, the boom tip will be continuously monitored from a remote survey station independent from the crane instrumentation. This survey station will have the necessary data input to monitor and calculate the boom tip location relative to the interfacing structures and components. The individual directing the rigging operations will be in constant communication with both the crane operator and the surveyor manning the remote survey station. These controls will be utilized to ensure that the exact location of the load is known and compliance with design requirements are maintained.

Assembly and disassembly of the OLS will be performed in accordance with the crane manufacturer's procedures and drawings and may be performed with Unit 1 and Unit 2 in Modes 1-6 or defueled. The assembly/disassembly process will require the use of mobile cranes and other equipment. During assembly and disassembly of the OLS, the

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main boom will lay in an area to the north of the Unit 1 Containment. The orientation of the main boom during assembly/disassembly, along with restrictions on mobile crane usage and SSC protection provisions, ensure that Unit 1 and Unit 2 can be safely shut down and/or maintained in a safe condition in the unlikely event of a load drop during assembly/disassembly of the OLS.

The OLS has been evaluated for seismic loads while unloaded and loaded with a steam generator (SG). A SG is the heaviest load that will be handled by the OLS. This seismic evaluation determined that the OLS would not collapse or result in a drop of the load during a seismic design basis Safe Shutdown Earthquake (SSE) event for the lift configurations to be used during the Sequoyah Unit 1 SGR. Therefore, use of the crane for the Sequoyah Unit 1 SGR will not result in Seismic II/I interaction issues on the SSCs located in the vicinity of the OLS.

To further demonstrate the capability of the OLS, it was determined that a whip-lash effect resulting from a postulated drop of a load from the OLS will not cause instability of the boom masts in the reverse direction, i.e. the masts will not flip over backwards and impact SSCs (e.g., Auxiliary Building, Control Building, etc.) behind the OLS.

Rigging operations will not be performed when wind speeds exceed the maximum operating wind speed for the OLS. If wind speeds increase during a rigging operation such that the wind speed may exceed the maximum operating speed, rigging operations will be suspended and the unloaded OLS will be secured by implementing administrative controls specified by the manufacturer. These administrative controls define the allowable mainmast and jib angles, and the slew drive and load block configurations, and are dependent on the wind speed.

To eliminate the effects of wind conditions beyond the maximum operating wind speed, a lift will not commence if analysis of weather data for the expected duration of the lift indicates the potential for wind conditions in excess of the maximum operating wind speed. Further, should there be an unexpected detrimental change in weather while the OLS is loaded, the lift will be completed and the OLS will be placed in its optimum safe configuration or the load will be grounded and the crane will be placed in a safe configuration.

The acceptability of the offsite dose consequences associated with a postulated drop of an OSG has been evaluated and compared to the consequences of postulated design basis accidents for a gaseous release. The evaluated consequences of an OSG drop are within the applicable regulatory requirements and are much less than the limiting licensing design basis accidents currently evaluated in Chapter 15 of the UFSAR.

Section 5.1.5 of NUREG-0612 indicates that the effects of load drops should be analyzed (in accordance with the guidelines of Appendix A to NUREG-0612) and the results should indicate that damage to safe shutdown equipment is not sufficient to preclude safe shutdown. Each of the potentially affected SSCs has been analyzed in accordance with the NUREG-0612 guidance to determine the effects of a load drop. Summarized below is the protection required to preclude an adverse effect and/or the actions or compensatory measures required to mitigate these effects should a load drop occur. Provision of the identified protection and implementation of the specified actions and compensatory measures assures that safe shutdown can be achieved following a load drop.

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- Containment

The heavy loads of concern that will be handled above the Containment will only be moved while the Unit 1 reactor is defueled. With fuel removed from the Containment, the only other safety issue is whether a load drop into the Unit 1 Containment will affect systems common to both units that pass through the Unit 1 Containment. To preclude a SG drop inside the Unit 1 Containment from affecting Unit 2, the ERCW system and Component Cooling System (CCS) shall either be isolated or be capable of being isolated with valves located outside of Containment. In addition, the Spent Fuel Pool (SFP) shall be isolated from the Unit 1 containment.

- Auxiliary Building

Heavy loads will not be handled over the Auxiliary Building and will not roll off the Containment roof onto the Auxiliary Building. Therefore, no additional load drop protection of the Auxiliary Building roof is required.

To preclude flooding of the Auxiliary Building due to a heavy load drop that causes a failure of piping (i.e., ERCW, RWST, PWST, and fire protection piping) in the ERCW pipe tunnel, a wall will be installed in the tunnel near the Auxiliary Building interface. The wall has been designed for 1) the hydrostatic head generated if the tunnel was completely filled with water and 2) an impact load associated with the rushing water just after a pipe break. It will also meet Sequoyah Seismic I (L) requirements, so that an earthquake would not cause a failure of nearby safety-related SSCs as a result of a seismically-induced failure of the wall. 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.

- Essential Raw Cooling Water System

Unit 1 ERCW Supply Piping and Train A Discharge Piping

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 Unit 1 ERCW system piping near the Unit 1 Containment would likely fail (i.e., crimp or rupture) as a result of the postulated load drops.

The postulated heavy load drop from the OLS might result in the failure of the Unit 1 ERCW trains A and B supply and/or ERCW train A discharge piping for both units. Mitigation of this assumed ERCW piping failure following a postulated heavy-load drop requires that compensatory measures be implemented to isolate the affected Unit 1 ERCW piping and restore ERCW flow to required equipment, as necessary. These compensatory measures will be proceduralized prior to use of the OLS for load handling. Personnel will be trained to implement the compensatory measures.

Unit 2 ERCW Supply Piping

The Unit 2 ERCW pipes running parallel to the west side of the Solar Building and east of the Unit 1 Containment do not directly lie on the load path and are located approximately 128 ft. from the load path. They were evaluated for the effects of impact energy due to a postulated drop of the SG at a distance away. This evaluation

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concluded that the Unit 2 ERCW piping will not fail and will remain functional under the impact effects of the postulated SG drop at a distance away from the piping.

ERCW Ductbanks

The load path for the SGs crosses over two buried ERCW ductbanks. The ductbanks contain cables associated with ERCW trains A and B for both units. Therefore, it is vital that a SG drop does not adversely affect the functioning of these cables. These ductbanks have been evaluated for impact loading from a direct vertical drop of the SG, as well as from the subsequent flopover fall of the SG. To minimize the impact energy from a vertical drop of the SG, the bottom of the SGs will be carried at an elevation not to exceed 3 ft. above grade while traversing the load path at and near these ductbanks.

The impact energy from a flopover fall was determined to be more critical than from a direct vertical drop of 3 ft. The evaluation of a flopover fall of a SG concluded that the ERCW ductbanks will remain functional in the event of a SG drop if sufficient soil cover is available over the ductbanks. Therefore, additional soil fill protection will be provided in the potentially affected areas above the ductbanks where the grade elevation is lower so as to bring the grade to a sufficient height to protect the ductbanks.

- Refueling Water Storage Tank

No heavy loads will be carried over the RWST by the OLS. Since a potential load drop from the OLS could only occur when Unit 1 is defueled, loss of the RWST function has no safety impact.

- Primary Water Storage Tank

Heavy loads may be carried over the PWST by the OLS. Since a potential load drop from the OLS could only occur when Unit 1 is defueled, loss of the PWST function has no safety impact.

- Main Steam Piping

The MS piping outside the Shield Building is a potentially affected SSC for the postulated load drops. Since a heavy load drop-induced failure of the MS piping will be isolated by closure of the MSIVs, no protective measures are required.

- Feedwater Piping

The FW piping outside the Shield Building is a potentially affected SSC for the postulated load drops. Since a heavy load drop-induced failure of the FW piping will be isolated by closure of the FW isolation valves, no protective measures are required.

- Fire Protection System Piping

The high-pressure fire pump and flood mode pump piping in the pipe tunnel is a potentially affected SSC for the postulated load drops. To minimize the impact of a rupture of this piping on flooding of the pipe tunnel, valves will be closed prior to movement of heavy loads with the OLS. Closure of these valves minimizes the actions that need to be taken to isolate a break. Closing these valves will not affect plant operation.

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IV. NO SIGNIFICANT HAZARDS CONSIDERATION DETERMINATION

TVA has concluded that operation of SQN Unit 2, in accordance with the proposed one-time change to the Operating License and implementation of compensatory measures following a load drop from the OLS during the Unit 1 steam generator replacement, does not involve a significant hazards consideration. TVA's conclusion is based on its evaluation, in accordance with 10 CFR 50.91(a)(1), of the three standards set forth in 10 CFR 50.92(c).

A. <u>The proposed amendment does not involve a significant increase in the probability or consequences of an accident previously evaluated.</u>

No changes in event classification as discussed in UFSAR Chapter 15 will occur due to the one-time change to the Unit 2 Operating License and implementation of compensatory measures following a load drop from the OLS during the Unit 1 steam generator replacement.

Accidents previously evaluated that are relevant to this determination are related to plant external events and load handling. The probability of an occurrence of a seismic event is determined by regional geologic conditions. Weather related events are determined by regional meteorological conditions.

The consequences of an earthquake have not changed. A seismic evaluation has determined that the OLS would not collapse or result in a drop of the load during a seismic design basis SSE event for the lift configurations to be used during the Sequoyah Unit 1 SGR. Therefore, use of the OLS for the Sequoyah Unit 1 SGR will not result in Seismic II/I interaction issues on the SSCs located in the vicinity of the OLS.

The consequences of a tornado have not changed. A lift will not commence if analysis of weather data for the expected duration of the lift indicates the potential for wind conditions in excess of the maximum operating wind speed. Rigging operations will not be performed when wind speeds exceed the maximum operating wind speed for the OLS. If wind speeds increase during a rigging operation such that the wind speed may exceed the maximum operating speed, rigging operations will be suspended and the unloaded OLS will be secured by implementing administrative controls specified by the manufacturer. Further, should there be an unexpected detrimental change in weather while the OLS is loaded, the lift will be completed and the OLS will be placed in its optimum safe configuration or the load will be grounded and the crane will be placed in a safe configuration.

An OSG drop has been postulated to occur to address the radiological consequences associated with the drop. The event is bounded by an OSG drop outside the containment (versus inside containment), since a steam generator failure outside containment results in more conservative doses. The dose analysis demonstrated that the OSG drop accident consequences remain below applicable regulatory limits and are bounded by similar, previously evaluated accidents at Sequoyah.

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Therefore, the proposed one-time change to the Unit 2 Operating License and implementation of compensatory measures following a load drop from the OLS during the Unit 1 steam generator replacement will not significantly increase the probability or consequences of an accident previously evaluated.

B. <u>The proposed amendment does not create the possibility of a new or different kind of accident from any accident previously evaluated.</u>

The possibility of a new or different accident situation occurring as a result of this condition is not created.

Three postulated scenarios related to heavy load handling during the SGRO were examined for their potential to represent a new or different kind of accident from those previously evaluated: 1) a breach of an OSG, resulting in the release of contained radioactive material, 2) flooding in the Auxiliary Building caused by the failure of piping in the ERCW tunnel, and 3) loss of ERCW to support safe shutdown in the operating Unit.

Failure of an OSG that results in a breach of the primary side of the steam generator could potentially result in a release of a contained source outside containment. The consequences of this event, both offsite and in the control room, were examined and were found to be within the consequences of the failure of other contained sources outside containment at the Sequoyah site.

To preclude flooding of the Auxiliary Building due to a heavy load drop, a wall will be installed in the ERCW tunnel near the Auxiliary Building interface. Thus, the postulated flooding of the ERCW tunnel will not result in flooding of the Auxiliary Building beyond those events previously evaluated.

The potential for a heavy load drop to cause loss of ERCW supply to Unit 2 is considered an unlikely accident for the following reasons:

- The lifting equipment was specifically chosen for the subject heavy lifts,
- Operators will be specially trained in the operation of the equipment and in the Sequoyah site conditions,
- Qualifying analyses and administrative controls will be used to protect the lifts from the effects of external events,
- The areas over which a load drop could cause loss of ERCW are a small part of the total travel path of the loads.

However, as additional protection against the potential for loss of ERCW, compensatory measures will be in place during heavy lifts that could cause such a loss to isolate the breaks and redirect flow to essential equipment.

Therefore, the potential for creating a new or unanalyzed condition is not created.

C. <u>The proposed amendment does not involve a significant reduction in a margin of safety.</u>

The OLS load handling activities support the replacement of the Unit 1 steam generators. The proposed one-time change to the Unit 2 Operating License and compensatory measures support Unit 2 operation and safe shutdown following a

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load drop. They do not result in changes in the design basis for plant SSCs. They do not, therefore, affect the margin of safety for plant SSCs.

Therefore, a significant reduction in the margin to safety is not created by this modification.

V. ENVIRONMENTAL IMPACT CONSIDERATION

The proposed change does not involve a significant hazards consideration, a significant change in the types of or significant increase in the amounts of effluents that may be released offsite, or a significant increase in individual or cumulative occupational radiation exposure. Therefore, the proposed change meets the eligibility criteria for categorical exclusion set forth in 10 CFR 51.22(c)(9). Therefore, pursuant to 10 CFR 51.22(b), an environmental assessment of the proposed change is not required.

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TVA Letter to NRC dated February 4, 2003,
Steam Generator Replacement Project –
Topical Report No. 24370-TR-C-002,
"Rigging and Load Handling Topical Report,"
Additional Information

Tennessee Valley Authority, Post Office Box 2000, Soddy-Daisy, Tennessee 37384-2000

February 4, 2003

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555

Gentlemen:

In the Matter of	)	Docket No. 50-327
Tennessee Valley Authority	)	

SEQUOYAH NUCLEAR PLANT (SQN) - STEAM GENERATOR REPLACEMENT PROJECT - TOPICAL REPORT NO. 24370-TR-C-002, "RIGGING AND HEAVY LOAD HANDLING TOPICAL REPORT," ADDITIONAL INFORMATION

Reference: TVA letter to NRC dated January 15, 2002, "Sequoyah Nuclear Plant (SQN) - Steam Generator Replacement Project - Topical Report No. 24370-TR-C-002, "Rigging and Heavy Load Handling Topical Report," Response to NRC Request for Additional Information

The purpose of this submittal is to provide additional information in response to NRC question 14 as contained in the reference letter. The additional information provides clarification of TVA's response to question 14 as discussed during a teleconference between TVA, NRC staff, and Bechtel on January 28, 2003.

In addition, this submittal provides information that shows equivalency between the crane described in the subject topical report and the alternate crane that TVA plans to utilize for the Sequoyah Unit 1 steam generator replacement project.

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TVA understands that the additional information will allow the staff to complete their review of the subject topical report. The approval of the topical report supports SQN's Unit 1 steam generator replacement outage that is scheduled to begin on March 16, 2003.

Enclosure 1 provides the additional information that supports TVA's response to NRC Question 14. Enclosure 2 provides the equivalency information associated with cranes.

This letter is being sent in accordance with NRC Regulatory Issue Summary 2001-05. There are no commitments contained in this submittal.

If you have any questions about this change, please telephone me at (423) 843-7170 or J. D. Smith at (423) 843-6672.

Sincerely,

Pedro Salas
Licensing and Industry Affairs Manager

I declare under penalty of perjury that the foregoing is true and correct. Executed on this 4 day of February, 2003

Enclosures
cc (Enclosures):
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ENCLOSURE 1
TENNESSEE VALLEY AUTHORITY
SEQUOYAH NUCLEAR PLANT (SQN)
UNIT 1
DOCKET NO. 327
ADDITIONAL INFORMATION FOR NRC QUESTION 14
TOPICAL REPORT NO. 24370-TR-C-002,
RIGGING AND HEAVY LOAD HANDLING"

Note: TVA is providing the following response to NRC Question 14. This response supersedes TVA's response previously provided in TVA letter to NRC dated January 15, 2003.

NRC Question No. 14

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.

TVA Response

The OLS will be supported on top of an 8 ft wide, 78.5 ft outer diameter concrete ring foundation that is supported by approximately 80 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 seated 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 and seismic II/I qualified in Reference 21 of Topical Report 24370-TR-C002 for strength and stability 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 strength and stability calculations. Therefore, lateral loading of the boom tip due to "swinging" was neglected in the stability and stress calculations.

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A seismic analysis has been performed for the OLS, which demonstrates that the OLS is capable of sustaining SSE loads without failure of the OLS foundation, the crane structural components or the rigging devices. The seismic evaluation of the OLS was based on dynamic modal analysis by the response spectrum method using a GT-Strudl finite element model (FEM).

Description of the Finite Element Model:

Schematic sketches of the finite element model showing the members, joints and boundary conditions (at the base) are provided as Figures 1 and 2.

The PTC Crane base ring (~21.5 m dia) is supported by 24 jack or ring cylinders with out-rigger plates at its base where it sits on the foundation. The jack cylinders are enclosed in support rings for protection. The outrigger plates ensure proper spreading of the load from the jack cylinders. These outrigger plates are permanently connected to the ring cylinders by means of two 40 mm dia shafts. The ring segments spread the load of the wheels to the jack cylinders by means of shear shafts (locked in place using a bush) and links that couple male and female segments alternately. The outrigger plates, support ring and jack cylinders are connected to the base ring segments.

The base ring and jack stands were not explicitly modeled. The wheel-system on the base ring of the crane is represented by vertical members with pinned ends at the 8 joints (Joints 139 to 146 - see Figures 1 and 2) in the model where the wheel-system is in contact with the base ring. Note that the wheel system is provided with up-stop devices to the under side of the base ring flange plates on which the wheels ride, thereby providing restraint against uplift from the ringer base. The base-ring and outrigger ringer/plate system of the crane is a rigid system. Therefore, the base-ring and outrigger ringer/plate system is represented in the model by rigid horizontal links from the 8 wheel joints to the center of the crane at the level of the base ring and thereon by a rigid vertical link to the foundation (pile cap) (Joint 150 - see Figures 1 and 2).

Soil-structure interaction effects at the foundation have been incorporated by including 6 soil springs at the foundation joint (Joint 150). The spring constants represent the horizontal (kx & kz), vertical (ky), rocking (kmx & kmz) and torsional (kmy) stiffnesses of the soil-pile foundation system. These spring constants were computed using published formulas in soil dynamics literature for circular bases (Reference: p169, Table 7.1 of Wu, T.H., *Soil Dynamics*, Allyn

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and Bacon Inc, Boston, 1971.). Since the piles are almost vertical (10° batter), the contribution to these spring stiffnesses from the piles were included only for the vertical and rocking springs. It is also noted that the PTC crane for the Sequoyah SGR project will operate with a total ballast (counterweight) of 1300 tonnes located at a radius of 11.83 m from the center of the crane (which is the center of the base ring). This ballast weight was accounted for in the model.

The seismic SSE response spectrum input into the model was derived as explained in the response to Question 34. The model assumes that the crane will remain firmly seated to the foundation in a seismic event without the jacks/outrigger plates sliding or lifting off from the foundation. This assumption was later verified by examining and evaluating the base reactions obtained from the finite element analysis at Joint 150 for sliding and overturning considerations.

Critical Crane Configurations Analyzed:

Three critical OLS load handling configurations (based on lift radius and load) that envelop all the configurations of the OLS for the Sequoyah Unit 1 SGRP were analyzed separately. Each of these three configurations were analyzed both with and without the lift load and the results evaluated for strength and stability. These critical crane configurations are:

Configuration 1

Lift Radius = 23.2 m. This configuration was analyzed for two lift load cases: (a) no lifted load, and (b) Lifted load (L) = maximum weight of a Steam Generator (SG) (Note that this is a hypothetical load case since minimum lift radius for an SG lift is 34 m).

This is the crane configuration where the lift radius is the smallest possible. This is the most vertical orientation the crane can physically be in.

Configuration 2

Lift radius = 55 m. This configuration was analyzed for two lift load cases: (a) no lifted load, and (b) Lifted load (L)= maximum weight of a Steam Generator.

This is the crane configuration where the lift radius is the maximum used when the lifted load is the full weight of a Steam Generator.

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Configuration 3

Lift radius = 83.5 m. This configuration was analyzed for two lift load cases: (a) no lifted load and (b) Lifted load (L) = 250 tonne (550 kip) test load that will be used for the load test of the OLS after erection at the Sequoyah site.

This is the crane configuration where the lift radius is the maximum lift radius used for any SGR Project lift with a relatively significant lift load (e.g. partial load of SG during upending/down-ending from/to the transporter, shield building concrete sections, SG compartment roof concrete sections, OLS test load concrete blocks, etc.) other than the full weight of a Steam Generator. The governing load for this condition was the 250 tonne (550 kip test load).

Responses were obtained for dead (D) + lifted loads (L)± E load combinations, where ±E is the seismic SSE load, which could act in either positive or negative sign. For the case where there is no lifted load, L = 0. Based on the structural responses from the finite element analyses for the D+L±E load combination, the OLS was evaluated for strength (stress) and stability under a seismic (SSE) event.

Summary of Results:

(a) Strength: Check for Stresses

The maximum enveloped (for the three critical configurations) stresses in the structural members/connections of the OLS under the D+L±E load combination (where E is the safe shutdown earthquake) were as follows.

- The maximum stress (axial) in the chord of the superstructure lattice frame mast components (main mast, back mast, jib, stay beams) is 54.4 ksi against the yield strength of 101.4 ksi. This stress occurred in the Back Mast chord made of DIN StE 690 material with a yield strength of 700 N/mm^2 (101.4 ksi).

- The maximum stress (axial) in the diagonal bracing of the superstructure lattice frame mast components (main mast, back mast, jib, stay beams) is 26.6 ksi against the yield strength of 66.6 ksi. This stress occurred in the bracing of the Jib made of DIN StE 460 material with a yield strength of 460 N/mm^2 (66.6 ksi).

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- The maximum stress (combined axial and bending) in a base component (longitudinal beams, ring segments, cross beams, winch beams) is 79.1 ksi against the yield strength of 101.4 ksi. This stress occurred in the longitudinal beams made of DIN StE 690 material with a yield strength of 700 N/mm^2 (101.4 ksi).

- The maximum stress (bearing) in a connection is 78.1 ksi against the yield strength of 101.4 ksi. This occurred at the eye of the connection between two insert sections of the back mast made of DIN StE 690 material with a yield strength of 700 N/mm^2 (101.4 ksi).

The above results demonstrate that the stress in the structural members/connections of the OLS under the D+L±E combination (where E is the safe shutdown earthquake) is less than the yield stress of the material.

(b) Stability: Check for Overturning and Sliding

Check for Overturning:

The overturning moment (Mo) is the maximum base moment reaction obtained from the analysis for the D+L±E load combinations. Overturning can occur about the edge of the jack cylinders located at a radius R = 21.5 m/2 = 11.75 m from the center of the crane. The resistance moment (Mr) was computed as the minimum vertical reaction obtained at the crane base for the D+L±E load combinations times the radius R.

The worse case overturning and resistance moments and the corresponding factor of safety (FOS = Mr/Mo) obtained for the D+L±E load combination for the three critical configurations analyzed is as below:

Configuration 1

No Load: Mo = 126180 k-ft, Mr = 142586 k-ft, FOS = 1.13.

Configuration 2

No Load: Mo = 92273 k-ft, Mr = 143820 k-ft, FOS = 1.56.
With Load: Mo = 105391 k-ft, Mr = 170046 k-ft, FOS = 1.61.

Configuration 3

With Load: Mo = 138840 k-ft, Mr = 160952 k-ft, FOS = 1.16.

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The above results demonstrate that the resistance moment is always greater than the overturning moment for all three configurations. This verifies the assumption made in the model with regard to overturning. It is, thus, concluded that the OLS crane will not overturn during a SSE.

Check for Sliding:

The sliding force (Fs) is the maximum lateral reaction at the base obtained from the analysis for the D+L±E load combinations. Sliding can occur at the interface of the jack stand steel outrigger plates and the top surface of the concrete pile cap. The force resisting sliding (Fr) is provided by the frictional resistance between the steel outrigger plates and the concrete pile cap. The coefficient of friction, μ, between steel and concrete considered in the evaluation is 0.57 (Reference: Rabbat, B.G, and Russel, H.G., Friction Coefficient of Steel on Concrete or Grout, ASCE Journal of Structural Engineering, Volume 111, No. 3, March 1985, pp 505-515). Fr is computed as the minimum vertical reaction obtained at the crane base for the D+L±E load combinations times μ.

The worse case sliding force (Fs) and sliding resistance (Fr) and the corresponding factor of safety (FOS = Fr/Fs) obtained for the D+L±E load combination for the three critical configurations analyzed is as below:

Configuration 1

No Load: Fs = 1483 k, Fr = 2306 k-ft, FOS = 1.55.

Configuration 2

No Load: Fs = 1395 k, Fr = 2326 k, FOS = 1.67.
With Load: Fs = 1371 k, Fr = 2750 k, FOS = 2.01.

Configuration 3

No Load: Fs = 1408 k, Fr = 2332 k-ft, FOS = 1.66.

The above results demonstrate that the resistance force is always greater than the sliding force for all three configurations. This verifies the assumption made in the model with regard to sliding. It is thus, concluded that the OLS crane will not slide during a SSE.

The results of the evaluation show that the critical failure mode of the OLS in a seismic event is overturning (tipping).

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The maximum lifted load of the generators during the SGRP is approximately 386 mt (metric tonnes). The maximum lift radius with the full SG load is 54.84 m. The rated chart capacity (including effect of allowable operating wind speed) of the OLS based on a 55 m lift radius is 408 mt. The worse case lifted load is therefore 94.3% of chart capacity. It is noted that this 94.3% chart capacity happens only for one of the Replacement Steam Generators (RSGs). For the other RSGs and the OSGs, the percentage of chart capacity at their maximum lift radii are around 91% or less. Further, the OLS is seated on a firm engineered pile foundation that is adequately designed for the design loads including seismic SSE loads obtained from the finite element analysis, ensuring that there will not be a collapse of the OLS due to a foundation failure in a seismic event.

The evaluation thus demonstrated that the OLS will remain structurally adequate and stable and will not collapse or result in a drop of the load during a design basis SSE event for the lift configurations to be used for the Sequoyah Unit 1 SGRP. Therefore, use of the OLS for the Sequoyah Unit 1 SGRP will not result in any seismic II/I interaction issues on the Category 1 SSCs located in the vicinity of the OLS.

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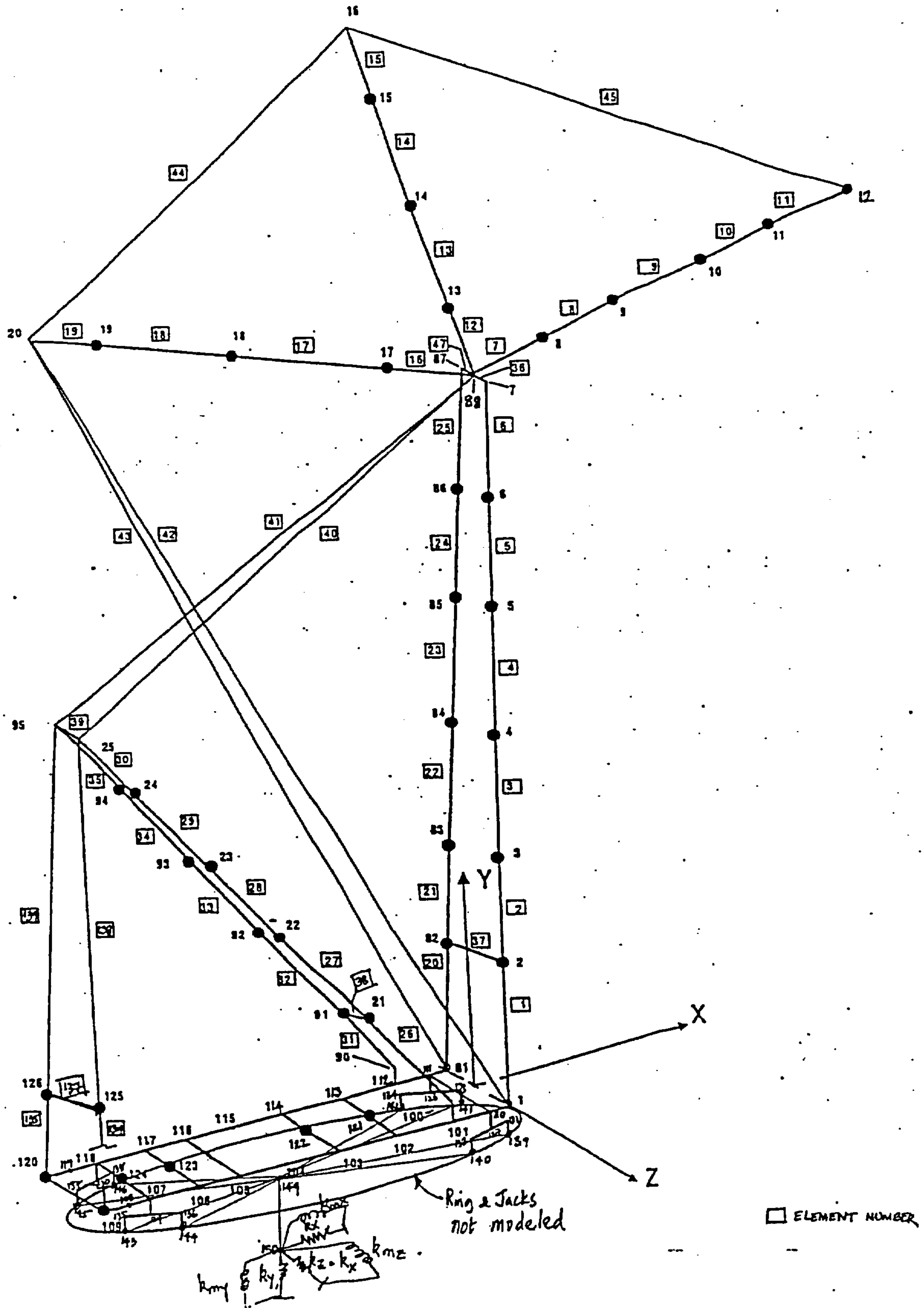

Figure 1 - GT Strudl Model of PTC Crane

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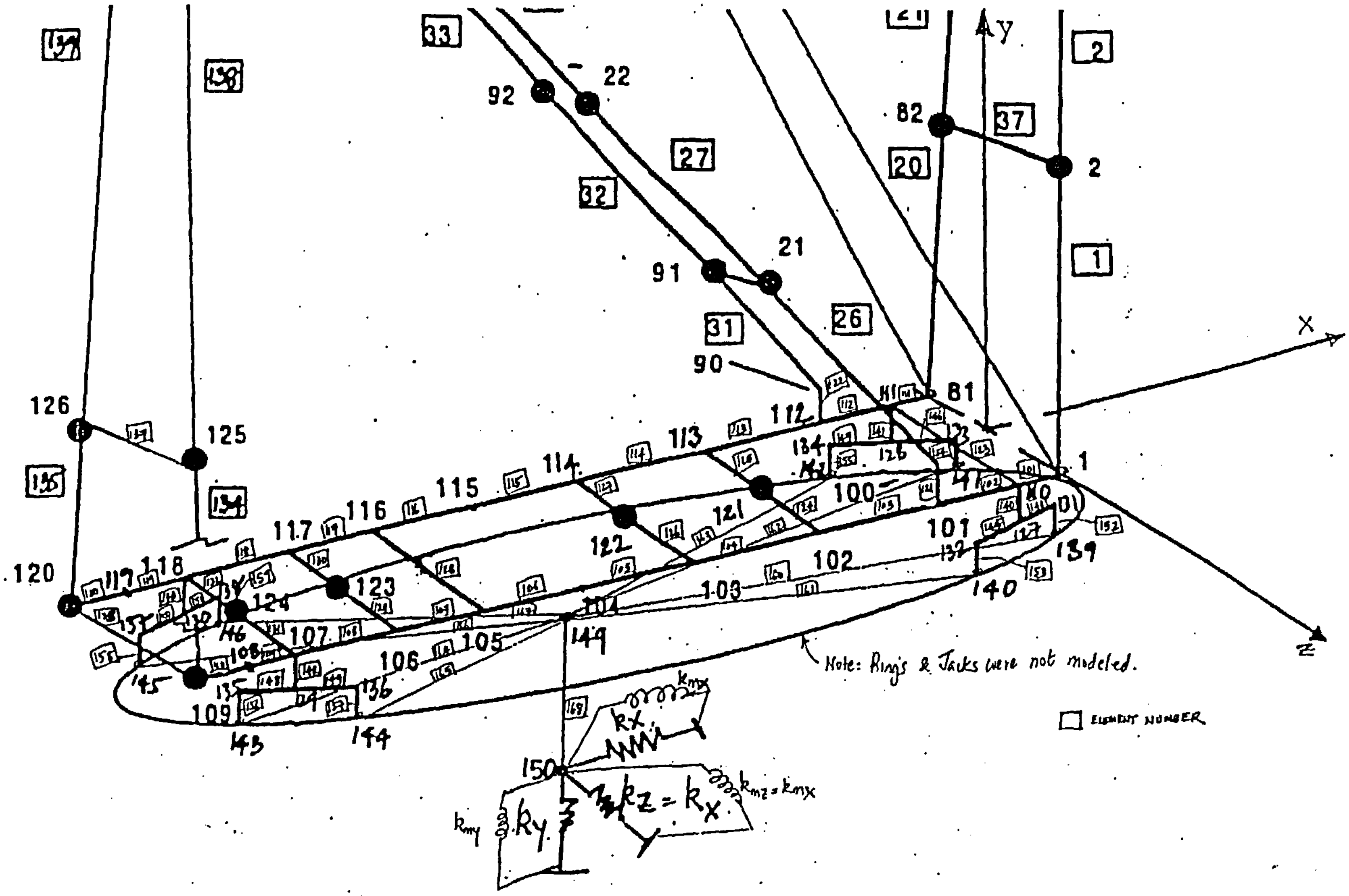

Figure 2 - GT Strudl Model of PTC Crane - Enlargement of Crane Base

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ENCLOSURE 2
TENNESSEE VALLEY AUTHORITY
SEQUOYAH NUCLEAR PLANT (SQN)
UNIT 1
DOCKET NO. 327
COMPARISON OF CRANES

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PTC Crane versus PRHD Crane Comparison

General Discussion

Topical Report 24370-TR-C-002, Section 5.1, describes the Outside Lift System (OLS) crane to be used for handling heavy loads during the steam generator replacement at Sequoyah Unit 1. Responses to NRC requests for additional information (RAIs) provided further details of the OLS. The crane to be used as the OLS will be one of two that are owned by the crane supplier. They are the Platform Twin-Ring Containerized (PTC) Heavy Lift Crane (see Figure 1) upon which the Topical Report was based, and the Platform Ring Heavy Duty (PRHD) Crane (see Figure 2). Recent developments in the usage schedule for the PTC may require the use of the PRHD crane instead of the PTC crane. The PRHD crane is a similarly configured crane and is the predecessor to the PTC crane. Therefore, its features are generally the same.

The standout feature of the PTC crane is that it is containerized. This means that it is easily transportable as standard containers of 20 or 40 ft length weighing no more than 35 tons. If used, the PTC crane would arrive in approximately 90 standard container-sized pieces. The PRHD crane is not as easy to transport as the PTC. The PRHD crane would arrive in approximately 140 pieces, not all of which are standard container-sized. The containerized feature of the PTC crane facilitates easy transport without any special requirements and is the most significant difference from the PRHD crane. However, this difference has no impact on the crane's ability to perform heavy load handling operations for the Sequoyah Unit 1 Steam Generator Replacement Project.

To evaluate and document the acceptability of the PRHD crane, a comparison of PTC crane attributes (detailed in Topical Report 24370-TR-C-002 and RAI responses) to the corresponding PRHD crane attributes is provided in the following table. Elevation views of the PTC and PHRD cranes are also provided as Figures 1 and 2, respectively. Two comparison bases are used to demonstrate similarity of the cranes: (1) direct comparison of the physical attributes; and (2) evaluation of the PRHD dynamic response characteristics under SSE for one of the critical configurations. Based on the comparison of the two cranes, it is shown that the conclusions of Topical Report 24370-TR-C-002 remain unchanged in that the attributes of the PRHD crane meet or exceed those of the PTC crane.

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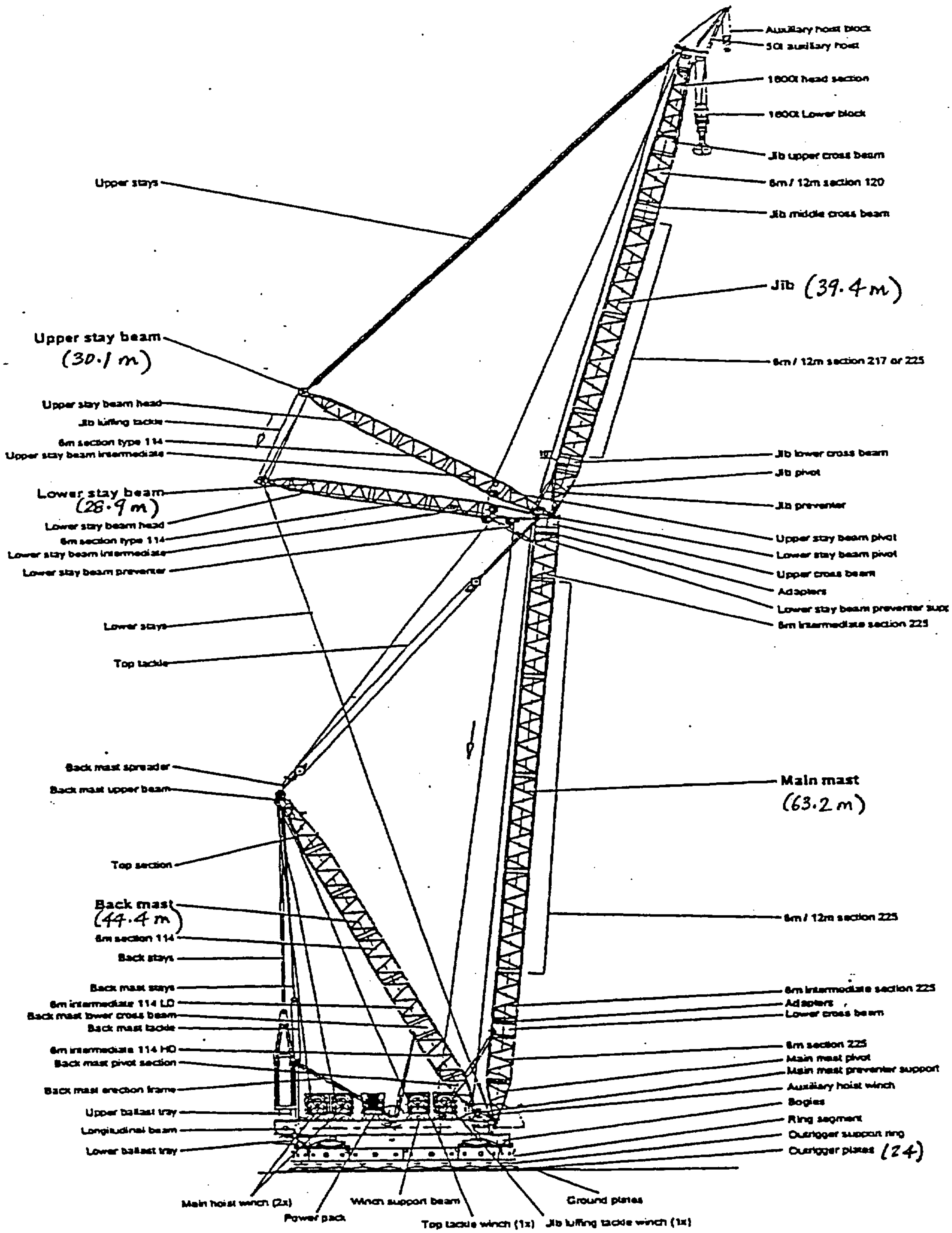

Figure 1 – PTC Crane

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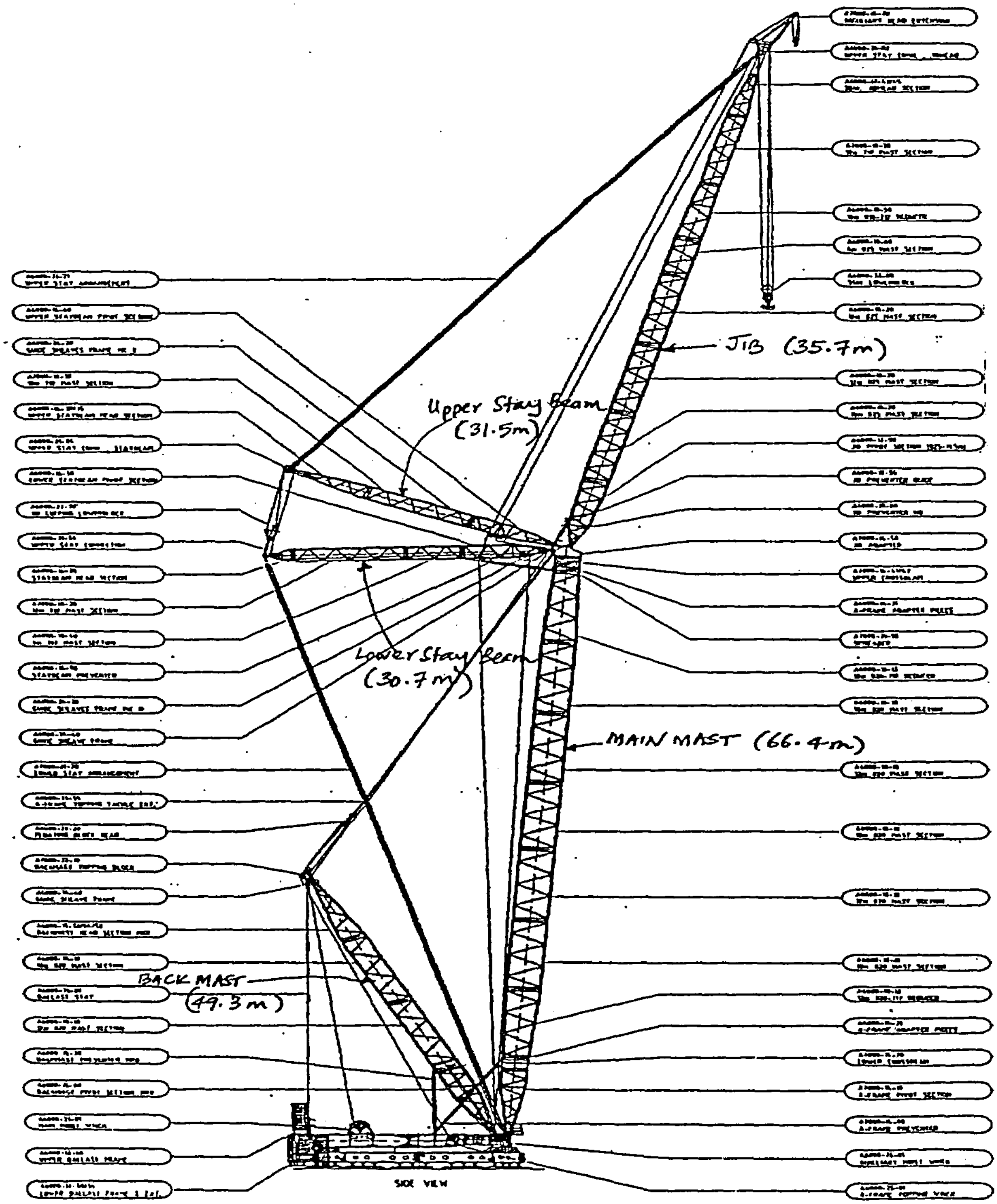

Figure 2 – PRHD Crane

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Comparison Table
PTC versus PRHD Crane

Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
General	• Name	Platform Twin-Ring Containerized Crane (PTC)	Platform Ring Heavy Duty Crane (PRHD) (also known as the Platform Twin-Ring HD crane)	The PTC Crane evolved from the PRHD crane with the intent to accomplish ease of transport.
	• Designed and manufactured by	Huisman-Itrec b.v. Rotterdam, The Netherlands and Van Seumeren, de Meern, The Netherlands	Huisman-Itrec b.v. Rotterdam, The Netherlands and Van Seumeren, de Meern, The Netherlands	Same Manufacturer, Designer and Supplier.
	• Owner/supplier	Mammoet b.v. (Van Seumeren Group), The Netherlands	Mammoet b.v (Van Seumeren Group), The Netherlands	
	• General construction	Ringer base mounted on jacks; longitudinal beams, ballast, main mast, luffing jib, back mast, and 2 stay beams	Ringer base mounted on jacks; longitudinal beams, ballast, main mast, luffing jib, back mast, and 2 stay beams	Very similar construction
Physical Construction Of Important Structural Components	***Main mast*** (RAI 15)			
	• Construction	A-Frame Lattice framework with the two legs of the A-Frame connected by a horizontal cross beam/frame	A-Frame Lattice framework with the two legs of the A-Frame connected by a horizontal cross beam/frame	It is seen from the physical construction of components described in this section that the stiffness properties of the major structural components (main mast, jib, back mast, stay beams, longitudinal beams etc.) are in general
	• Length	63.2 m	66.4 m	
	• Height to mast pivot	5070 mm	4620 mm	
	• Width between pivots	10080 mm	9975 mm	
	• Distance from center line of crane base to main	9210 mm	9500 mm	

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	mast pivot			equivalent or better for the PRHD crane. Considering the large geometry and mass of the crane system, any differences are not sensitive enough to cause any significant change in the structural/ dynamic response characteristics of the crane. The similarity in dynamic response characteristics for the SSE has been demonstrated in Calculation 24370-C-026, Rev. 1.
	• Chord centerline dimensions of lattice frame sections (width x depth)	1880 mm x 2360 mm (each A-Frame leg)	2750 mm x 3650 mm (each A-Frame leg)	
	• Chords (dia x thk)	193.7 mm x 25 mm	267 mm x 20 mm	
	• Bracings (dia x thk)	121 mm x 7.1 mm	152.4 mm x 5 mm & 152.4 mm x 8 mm	
	• Equivalent structural section properties of each leg of A-Frame	A = 0.57 ft^2, Iy = 5.45 ft^4, Iz = 8.57 ft^4	A = 0.67 ft^2, Iy = 13.65 ft^4, Iz = 24 ft^4	
	Jib (RAI 15)			
	• Construction	Double-framed lattice framework with the two legs of the double frame connected by horizontal cross beam at three locations	Single-frame lattice framework	
	• Length	39.4 m	35.7 m	
	• Width between pivots	3950 mm	2960 mm	
	• Chord centerline dimensions of lattice frame sections (width x depth)	1880 mm x 2360 mm (each leg)	3650 mm x 2750 mm	
	• Chords (dia x thk)	193.7 mm x 25 mm	267 mm x 25 mm	
	• Bracings (dia x thk)	121 mm x 7.1 mm	168.6 mm x 6.3 mm	
	• Equivalent structural section properties	A = 1.14 ft^2, Iy = 58.7 ft^4, Iz = 17.4 ft^4	A = 0.82 ft^2, Iy = 29.4 ft^4, Iz = 16.7 ft^4	

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	Backmast (RAI 15)			
	• Construction	A-framed lattice framework with the two legs of the A- frame connected by a horizontal cross beam	Single-frame lattice framework forked at the pivot section	
	• Length	44.4 m	49.3 m	
	• Backmast Angle (fixed)	~66°	~64°	
	• Width between pivots	10080 mm	8000 mm	
	• Chord centerline dimensions of lattice frame sections (width x depth)	1840 mm x 2360 mm (each leg)	3650 mm x 2750 mm	
	• Chords (dia x thk)	168.3 mm x 20 mm	267 mm x 20 mm	
	• Bracings (dia x thk)	114.3 mm x 4 mm	152.4 mm x 5 mm	
	• Equivalent structural section properties	A = 0.6 ft^2, Iy = 99 ft^4, Iz = 8.9 ft^4	A = 0.67 ft^2, Iy = 24 ft^4, Iz = 13.7 ft^4	
	Stay Beams (Upper and Lower) (TR Sect. 5.1)			
	• Length	29.3 m (upper) and 28.01 m (lower)	31.5 m (upper) and 30.66 m (lower)	
	• Chord centerline dimensions of lattice frame sections (width x depth)	2360 mm x 1840 mm	2750 mm x 2100 mm	
	• Chords (dia x thk)	168.3 mm x 20 mm	93.7 mm x 17.5 thick mm	
	• Bracings (dia x thk)	114.3 mm x 4 mm	121 mm x 4 mm	
		A = 0.3 ft^2, Iy = 4.45 ft^4, Iz = 2.71 ft^4	A = 0.43 ft^2, Iy = 8.67 ft^4, Iz = 5.07 ft^4	

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	• Equivalent structural section properties			
	Longitudinal Beams (2)	1474 mm x 1800 mm box section built up from 20 mm flange plates and 12 mm side plates; 21 m long	1250 mm x 2000 mm box section built up from 20 mm flange plates and 15 mm side plates; 21 m long	
	• Construction	A = 1.05 ft^2, Iy = 4.52 ft^4, Iz = 7.84 ft^4	A = 1.17 ft^2, Iy = 3.23 ft^4, Iz = 7.86 ft^4	
	• Structural section properties			
	Base	21.5 m diameter ringer base mounted on 24 jacks, self leveling	21.5 m diameter ringer base mounted on 48 jacks, self leveling	Increased number of jacks/out-rigger plates enables better distribution and minimizes foundation bearing pressures.
	• Ringer and jacks (RAI 14, 15, TR Sect. 5.1)	24 Rectangular – 5 ft x 7.5 ft	48 Trapezoidal – 3.8 ft to 5 ft x 8.5 ft	
	• Outrigger plates under jacks (RAI 14)	920 mm x 1560 mm box section built up from 40 mm flange plates and 20 mm side plates	680 mm x 1560 mm box section built up from 50 mm flange plates and 25 mm side plates	
	• Construction of Base ring segments	A = 1.43 ft^2, Iy = 1.47ft^4, Iz = 6.18 ft^4	A = 1. 5 ft^2, Iy = 0.8 ft^4, Iz = 6 ft^4	
	• Structural section properties of base ring	2685 mm	2750 mm	
		32 in front and 32 at the rear	64 in front and 32 at the rear	
	• Height to top of base ring	1600 tonnes (metric)	2000 tonnes (metric)	PRHD has better mast head capacity
	• Bogie wheels			Comparable
	Mast Head Capacity	1300 tonnes (metric)	1250 tonnes (metric)	
	Ballast (Counterweight) for Sequoyah SGR Project			

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
Crane Weight and center of gravity	• Total weight including ballast • Vertical height of crane center of gravity • Percentage of crane mass located at its base	2200 tonnes (approx.) 13.94 m Approximately 80% - 85%	2200 tonnes (approx.) 11.14 m Approximately 80% - 85%	Total crane weight is approximately the same. The cg of the PRHD crane is lower by about 9 ft. This is favorable for stability (overturning) of the crane under seismic loads. Since the total mass of the two cranes is approximately the same, the mass of the superstructure components will also be very comparable. Since the stiffness characteristics are also similar, the dynamic response of the structure under seismic loads will also be similar.
Material of Structural Components	• Main chords of main mast, jib, backmast and stay beams • Bracings of main mast, jib, backmast and stay beams • Base Components: Longitudinal beams, cross	StE 690 (Fy = 101.4 ksi) StE 460 (Fy = 66.6 ksi) StE 690 (Fy = 101.4 ksi)	StE 690 (Fy = 101.4 ksi) StE 460 (Fy = 66.6 ksi) StE 690 (Fy = 101.4 ksi)	Material of all structural components is the same.

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	beam, winch beams, ballast trays, base ringer			
Operating Configurations	For the crane configuration being supplied for the SGR Project	SFSL: • Jib offset fixed at 10° or 40° • Main mast angle 10° or 40° to 86° SWSL: • Main mast angle fixed at 86° or 80° • Jib offset 5° to 86° or 80°	SFSL: • Jib offset fixed at 10° or 40° • Main mast angle 10° or 40° to 86° SWSL: • Main mast angle fixed at 86° or 80° • Jib offset 10° to 86° or 80°	Operating configurations are the same.
Design	• Lift Capacity Design Codes (RAI 15, 19)	DIN 15018 Parts 1 & 3, 15019 Part 2, 15120 Part 1, 1055 Part 4, CE, ASME B30.5-1994, SAE J987 & SAE J765	DIN 15018 Parts 1 & 3, 15019 Part 2, 15120 Part 1, 1055 Part 4, CE & ASME B30.5-1994	Primary codes are the same. The additional SAE codes J987 & J765 mentioned for the PTC crane are related to testing for ASME B30.5 certification . The PHRD crane has been certified by All Test & Inspection Inc., Blaine Minnesota, to be in compliance with the intent of B30.5. Therefore, no impact.
	• Structural Design Certified and Approved by.	Lloyd's Register, Croydon, Great Britain	Lloyd's Register, Croydon, Great Britain	Same
	• Meets or exceeds ASME NQA-1 Subpart 2.15 requirements (TR Sect.	Yes.	Yes.	Same

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	5.1) • Crane design conforms to the guidelines of ANSI B30.5, which meets the intent of ANSI B30.2 and CMAA-70 (TR Sect. 4.2(7))	Yes.	Yes	Same
Wind Speed Limitations	• Wind speed monitoring instruments (RAI 10)	Two anemometers – one at the jib tip and the other at the top of the backstay. The anemometers are verified to be operational prior to jib/backstay being erected.	Two anemometers – one at the jib tip and the other at the top of the backstay. The anemometers are verified to be operational prior to jib/backstay being erected.	Same
	• Maximum permissible wind load contributed from the lifted load (RAI 15)	0.75% of the Safe Working Load specified in the Load Capacity Chart	0.75% of the Safe Working Load specified in the Load Capacity Chart	Same
	• Maximum operating wind speed (RAI 15, 35)	Limited to 10 m/s (22 mph) when the lifted load is outside containment and more than 3 ft from grade. Limited to 15 m/s (33 mph) when the lifted load is 3 ft or less from grade or when the lifted load is inside containment.	Limited to 10 m/s (22 mph) when the lifted load is outside containment and more than 3 ft from grade. Limited to 15 m/s (33 mph) when the lifted load is 3 ft or less from grade or when the lifted load is inside containment.	Same
	• Actions/configurations for crane placement, when wind speed exceeds or expected to exceed maximum operating wind speed and time taken to accomplish these actions (RAI 15)	See response to RAI 15.	Per Section 5 of the Crane Manual. The requirements are the same as for the PTC crane (see Response to RAI 15) except for an additional requirement for the case when wind speeds are expected to exceed 46 m/s (103 mph). As for the PTC crane, the main mast and jib must be lowered to the ground when wind speeds are expected to exceed 46 m/s. However, for the PRHD crane the lowering must be	For the Sequoyah U1 SGR Project, the OLS Crane will be lowered in accordance with the procedures in the Operating Manual when wind speeds are expected to exceed 30 m/s (67 mph) based on

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
			performed at wind speeds below 30 m/s (67 mph). Also, the slewing motors brake shall be on. In the event wind speeds exceed 30 m/s and the crane is not lowered prior to 30 m/s, the crane will be placed in the configuration recommended by the manufacturer for winds in the 30-46 m/s range (same as for the PTC). The time taken to accomplish these actions are approximately the same as for the PTC crane as stated in response to RAI 15.	weather forecasts. Therefore, no impact.
Rated Chart Capacity	• Maximum rated chart capacity of the crane configuration being supplied for the SGR Project	1135 tonnes (1250 tons).	1500 tonnes (1653 tons).	PRHD has a higher maximum rated chart capacity.
	• Minimum tipping factor of safety (FOS) associated with the safe working load (SWL) specified in the crane rated load capacity charts (includes effect of permissible operating wind speed) – RAI 14, 35	1.25	1.3	PRHD has a better FOS against tipping.
	• Load charts and operating restrictions consider applicable dead, live, wind, impact, and out-of-plumb lift loads (TR Sect. 5.1)	Yes	Yes	Same
	• Rated load capacity for the range of heavy lifts for	440.8 tons (400 mt) to 517.9 tons (470 mt)	465.2 tons (422 mt) to 554.6 tons (493 mt)	PRHD has a higher rated load capacity than the PTC for the

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	the Sequoyah SGR Project as stated in TR Sect. 5.1 (also see RAI 12)			range of SGR lifts.
Lifted load Vs chart capacity (% of chart capacity) for the Critical SG lifts (RSGs)	• RSG 1 at 52 m lift radius • RSG 2 at 52 m lift radius • RSG 3 at 55m (56 m using PRHD crane) lift radius • RSG 4 at 52 m lift radius	385 mt Vs 420 mt (91.7%) 386 mt Vs 420 mt (91.9%) 385 mt Vs 408 mt (94.3%) 385 mt Vs 420 mt (91.6%)	385 mt Vs 439 mt (87.7%) 386 mt Vs 439 mt (87.9%) 385 mt Vs 422 mt (91.2%) 385 mt Vs 439 mt (87.7%)	PRHD crane has better % chart capacity for the critical lifts. Note that the OSGs weigh less than the RSGs.
Load Testing	***By Manufacturer at Production***			
	Load Test soon after production (TR Sect.5.1, RAI 19, 12(a))	Load tested to 125% of rated load capacity. This meets load test requirements of ASME NQA-1, 1997, Subpart 2.15 Sect. 601.2.	Load tested to 130% of rated load capacity. This meets load test requirements of ASME NQA-1, 1997, Subpart 2.15 Sect. 601.2.	Similar. The PRHD crane was load tested to a higher percentage of the rated load capacity than for the PTC.
	Side load at the load to which tested (RA 15)	2% of Safe Working Load (SWL).	Accepted by ANSI as being equivalent to being tested for side loads equal 2% of Safe Working Load (SWL): The equivalency was established by comparing calculations performed on the PRHD to the test results of the PTC crane for 2% SWL side loads.	Same
	Construction, fabrication of steel structure and testing certified and approved by	Lloyd's Register, Rotterdam, The Netherlands. Testing of the crane was further witnessed by Keboma, The Netherlands. ANSI and DIN Inspectors	Lloyd's Register, Rotterdam, The Netherlands. Testing of the crane was further witnessed by Keboma, The Netherlands. DIN Inspectors also	Similar

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
		also witnessed the test and have certified the crane (RAI 19).	witnessed the test and have certified the crane.	
	Load Test to be performed at the Sequoyah Site during and after erection and prior to use			
	Load test after erection (RAI 10, 12(a), TR Sect. 5.1)	Crane will be load tested by lifting a 250 mt (550 kip) test load assembly with the crane boomed out to a radius where the test load represents 110% of the crane rated capacity at this radius	Crane will be load tested by lifting a 250 mt (550 kip) test load assembly with the crane boomed out to a radius where the test load represents 110% of the crane rated capacity at this radius.	Same
Crane Safety Systems and Verification Actions during and following erection	• Whether redundancy available for crane systems (RAI 19)	Yes. The crane has dual engines, dual hydraulic systems, and dual computers.	Yes. The crane has dual engines, dual hydraulic systems, but a single computer. However, all sensors leading to the computer are dual.	Same except that the PRHD has only a single computer system. The supplier will provide a spare computer or spare parts for the computer at site. Also, the crane can be manually controlled in the event of a computer failure.
	• Whether the load can be safely lowered using a 12-volt car battery and manual controls in the event that all power and hydraulic systems fail (RAI 19)	Yes.	Yes.	Same
	• Actions to be performed	See response to RAI 12(b).	Same as for PTC as described in	Same

Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	for verification, during and following erection, of the proper assembly of electrical and structural components – Inspection, Functional test etc. (RAI 12(b))		response to RAI 12(b) in accordance with the corresponding sections of the Crane User Manual.	
	• Actions to be performed for verification of the integrity of all control, operating, and safety systems following erection (RAI 12(c))	Functional tests and load tests as described in response to RAI 12(c).	Same as for PTC as described in response to RAI 12(c).	Same
	• Ability to protect against an overload situation (RAI 12(d))	Crane equipped with load measuring devices that provide indication to operator and a redundant load-moment safety system that progressively warns and then disables crane operations. See response to RAI 12(d). Also equipped with instrumentation that provide continuous read out of crane/boom orientation and location of boom tip and load bock (TR Sect. 5.1). The crane computer system is equipped with safe load indicator (SLI) software that prevents load movement outside of specified limits (RAI 38).	Same as for PTC as described in response to RAI 12(d). Also equipped with instrumentation that provide continuous read out of crane/boom orientation and location of boom tip and load block (TR Sect. 5.1). The crane computer system is equipped with safe load indicator (SLI) software as for the PTC crane.	Same
	• Crane Safe Load Indicator SLI) software (RAI 38)	Software by Pietz Automatiserings Techniek (PAT) supplied by Krüger Systemtechnik (a leading specialist/supplier in electronic control and measurement services based in Germany)	Software by Pietz Automatiserings Techniek (PAT) supplied by Krüger Systemtechnik (a leading specialist/supplier in electronic control and measurement services based in Germany)	Software from the same designer/ supplier
Calibration of Crane	• Date of last performed instrument calibration	September, 2002	November, 2002	No impact.

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
Crane Instruments (RAI 11)	instrument calibration • Actions to ensure crane is equipped with correctly calibrated instruments (load cell, boom radius indication readouts, incline meter readings, safe load indicator, anti-two block switches, airplane warning lights and boom stops)	See response to RAI 11.	Same as for PTC as described in response to RAI 11.	Same
Inspections and Maintenance After Erection at Sequoyah Site	• Daily Inspections (RAI 36) • Maintenance	See Response to RAI 36. Maintenance will be performed as required, based on daily inspections, in accordance with the Crane Operating Manual.	Same as for PTC. See Response to RAI 36. Maintenance will be performed as required, based on daily inspections, in accordance with the Crane Operating Manual.	Same Same
Crane Assembly (Erection)/ Disassembly	• Heaviest individual crane component of the unassembled crane (TR Sect. 5.3) • Heaviest assembled component lifted during the erection process (TR Sect. 5.3) • Largest ballast blocks (TR Sect. 5.3) • Assembly and	Lower counterweight tray – 25.3 mt (55.6 kips) Main mast at 122.7 mt (270 kips) 10.9 tons (21.8 kips) Yes.	Main Boom Head (2000t capacity) - 36.5 mt (80.4 kips) Back Mast – 95 mt (209 kips) 12.5 mt (27.5 kips) Yes.	Enveloped by heaviest assembled lifted component (see below). Less than PTC. Therefore, better. Enveloped by heaviest lifted component. Same

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	disassembly will be performed in accordance with the crane manufacturer's procedures and drawings. (RAI 12.(b), TR Sects. 4.2, 5.1) • Main boom assembly (TR Sect. 5.1)	Assembly of the main boom will be performed in an area to the north of the Unit 1 Containment as shown on Figure 5-2 of Topical Report 24370-TR-C-002.	Assembly of main boom will start to the west of the position shown on Figure 5-2 of Topical Report 24370-TR-C-002 (as shown on Figure 3), which places the boom in a northerly direction from the crane base. This will allow assembly of the main boom on the ground without interfering with the RWST structure. The main boom will be attached to the base of the crane and moved to the position shown on Figure 5-2 of Topical Report 24370-TR-C-002 for erection of the jib and stay beams.	Assembly process is the same. If initial assembly occurred as shown in the topical report, it would have to occur off the ground to clear an interference with the RWST retaining wall. Controls for erection cranes and protection of underground equipment are the same. Therefore, no impact.
Seismic II/I Qualification (TR Sect. 4.1, 5.1, RAI 14)	***Methodology/Criteria used***	Dynamic Modal Analysis by the Response Spectrum method using a GT-STRUDL Finite Element model. The response spectra used is described in response to RAI 34. Three enveloping critical configurations (as described in RAI 14) in both loaded and not-loaded conditions were evaluated. The criteria used were: (a) *Strength*: limit the stresses in the crane components under seismic loads less than yield; and (b) *Stability:* Resistance moment (Mr) is always greater than the Overturning moment (Mo), and Sliding resistance (friction) force (Fr) is always	The design, construction, lift capacity, operation and mass of the PRHD crane are very comparable to the PTC crane. The vertical height of the cg of the PRHD crane is lower than that of the PTC by ~9 ft. Based on review of the PTC evaluation, the governing failure mode of the crane in a seismic SSE event is not over-stress, but stability by overturning. The most critical configuration for overturning for the PTC was with the boom in it's most vertical position (LR = 23.2 m) without load (FOS = 1.13). Therefore, the dynamic characteristics of the PRHD	The evaluation established similarity in dynamic response characteristics between the two cranes and improvement in FOS against overturning for the PRHD crane.

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
		resistance (friction) force (Fr) is always greater than the sliding force (Fs).	crane were established and compared to that of the PTC crane for this critical configuration. This was accomplished by modifying the GT-STRUDL model to incorporate the changes in the PRHD geometry, mass and stiffness from that of the PTC crane and re-running the response spectrum analysis for the above critical configuration for stability. The results obtained for stability, in comparison to the PTC, is as below.	
	Results Summary for Dead (D) + Lifted (L) ± SSE (E) :			
	• Most Critical Case for Overturning (LR = 23.2 m without a load: Config. 1 in RAI 14)	Mo = 126180 k-ft, Mr = 142586 k-ft, Mr > Mo. (FOS = Mr/Mo = 1.13)	Mo = 120574 k-ft, Mr = 150165 k-ft, Mr > Mo. (FOS = Mr/Mo = 1.25)	The difference in overturning and resistance moments between the two cranes is of the order of 5%, which is insignificant. The FOS against overturning improved for the PRHD crane.
	• Most Critical Case for Sliding (LR = 23.2 m without a load: Config. 1 in RAI 14)	Fs = 1483 k, Fr = 2306 k-ft, Fr > Fs. (FOS = Fr/Fs = 1.55)	Fs = 1562 k, Fr = 2428 k-ft, Fr > Fs. (FOS = Fr/Fs = 1.55)	The difference in sliding and resistance forces between the two cranes is of the order of 5%, which is insignificant.
	• Maximum stress in a Superstructure Mast component	54.4 ksi (axial stress in back mast chord) against yield strength of 101.4 ksi.	The analysis showed that the PHRD crane mode shapes and frequencies of the modes with significant dynamic participation under a SSE compared fairly well to that for the PTC crane,	The stress levels in the PRHD crane structural components will also remain below yield stress.

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Crane Attribute	Description	PTC Crane	PRHD Crane	Remarks/Basis for Acceptability of PRHD Crane
	• Maximum Stress in base components	79.1 ksi (combined axial and bending in the longitudinal beams) against yield strength of 101.4 ksi.	thereby establishing similarity in dynamic response of the two cranes. The FOS against overturning improved to 1.25 for the PRHD crane. Since the stiffness properties of the PRHD crane are in general comparable or better than that of the PTC crane in addition to margins available in the PTC results, the stress levels in the PRHD crane components will also remain below yield strength. Therefore, the results and conclusions of the PTC crane remain valid for the PRHD crane.	
	Conclusion	The evaluation demonstrated that the crane will not collapse nor drop a load in a SSE event and, therefore, will not cause a seismic II/I interaction with safety-related SSCs.	Same as for PTC crane. The PRHD crane may be used as an alternated crane	Conclusions for the PTC crane remain valid for the PRHD crane.
References	Crane manual reference (TR Sect. 10.0)	Bechtel Supplier Document 24370-SC-004-PTCManual-001, User Manual – Platform Twin-Ring Containerized Crane, Rev. 0	Bechtel Supplier Document 24370-SC-004-003-001, Rev. 2, User Manual – Platform Ring Crane, Huisman B.V.-Itrec B.V., Rotterdam, December 17, 1997	
	Seismic Evaluation Calculation (TR Sect. 10.0)	Calculation 24370-C-026, "Evaluation of PTC Crane for Seismic and Wind/Tornado Loads", Revision 0	Calculation 24370-C-026, "Evaluation of OLS Crane for Seismic and Wind/Tornado Loads", Revision 1	

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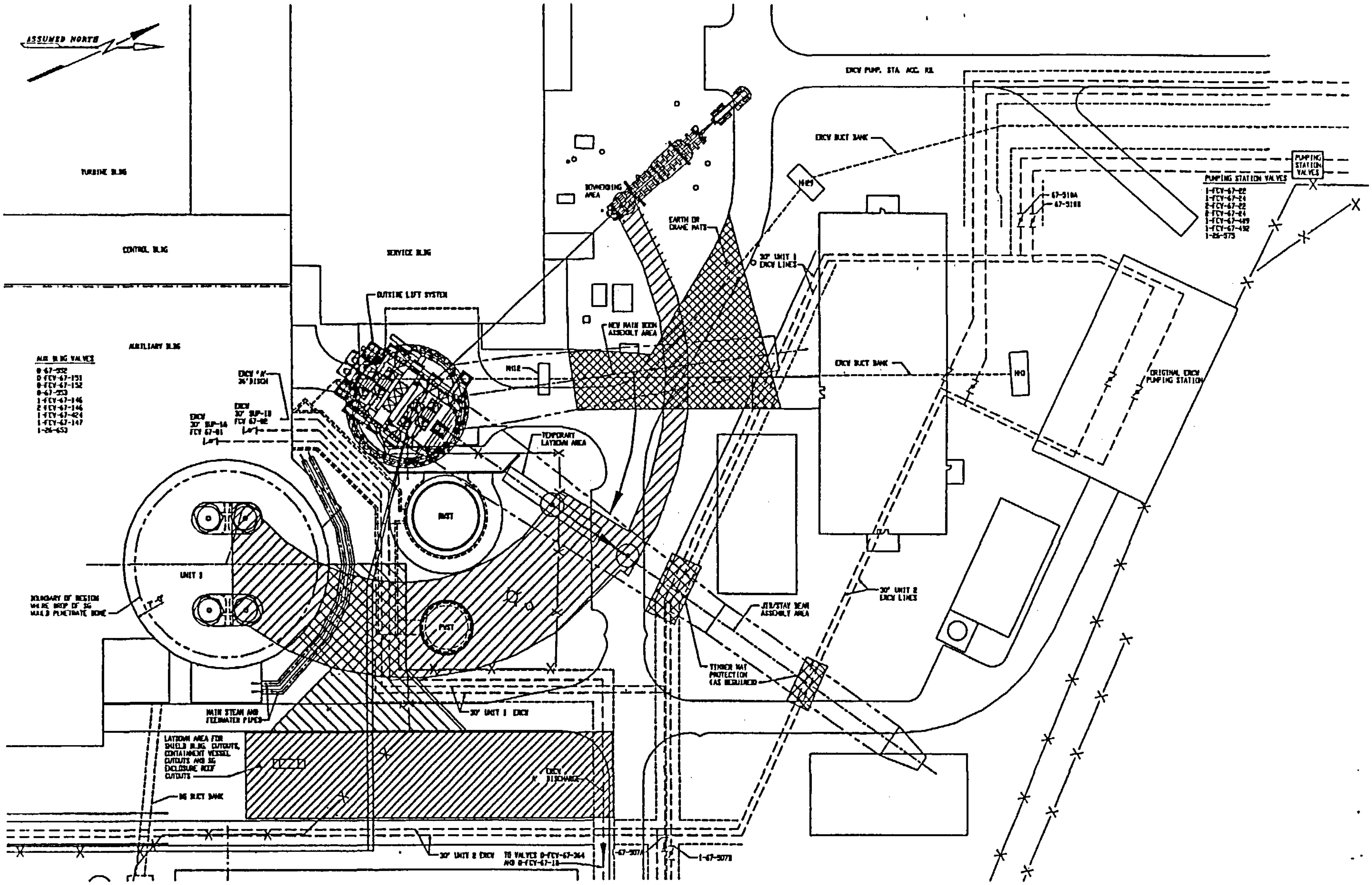

Figure 3 – PRHD Boom Assembly Location

Summary and Conclusion:

The PRHD and PTC cranes are very comparable in design, construction, lift capacity (better for the PRHD), operation and dynamic response under seismic loads. The PRHD crane base ring will be supported on 48 jacks as against 24 jacks for the PTC, providing an improved distribution of bearing pressures at the base. The seismic evaluation of the PRHD Crane established that the dynamic response characteristics of the PRHD crane are similar and further an improvement over that for the PTC crane. The margins of safety available for the structural components of the PTC crane were large enough that due to similarity in responses of the PTC and PRHD cranes, the stress levels in these elements will remain below yield stress for the PRHD crane. The evaluation determined that the factor of safety against overturning (critical failure mode) for the PRHD crane for the most critical configuration improved over that for the PTC crane. This is as expected since the two cranes have approximately the same mass (majority of which is located near the base), very similar dynamic response characteristics, and the center of gravity of the PRHD is lower than that of the PTC. Therefore, the results and conclusions of the evaluation for the PTC crane remain valid for the PRHD crane. Thus, PRHD crane may be used as an alternate crane in lieu of the PTC crane for the Sequoyah Unit 1 SGRP.

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TVA Letter to NRC dated February 19, 2003, Steam Generator Replacement Project (SGR) – Topical Report No. 24370-TR-C-002, "Rigging and Load Handling Topical Report," Commitment Information

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Tennessee Valley Authority, Post Office Box 2000, Soddy-Daisy, Tennessee 37384-2000

February 19, 2003

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555

Gentlemen:

In the Matter of	)	Docket No. 50-327
Tennessee Valley Authority	)	

SEQUOYAH NUCLEAR PLANT - STEAM GENERATOR REPLACEMENT (SGR) PROJECT - TOPICAL REPORT NO. 24370-TR-C-002, "RIGGING AND HEAVY LOAD HANDLING TOPICAL REPORT," COMMITMENT INFORMATION

Reference: TVA letter to NRC dated January 15, 2002, "Sequoyah Nuclear Plant (SQN) - Steam Generator Replacement Project - Topical Report No. 24370-TR-C-002, "Rigging and Heavy Load Handling Topical Report," Response to NRC Request for Additional Information

The purpose of this submittal is to provide TVA commitments to three specific compensatory actions delineated in the subject topical report. As we understand it, establishing these actions as specific NRC commitments is desired as they are considered to be risk significant activities intended to mitigate the risks of a potential heavy load drop.

TVA understands that the commitment information will allow the staff to complete their review of the subject topical report. The approval of the topical report supports Sequoyah's Unit 1 steam generator replacement outage that is scheduled to begin on March 16, 2003.

It should be noted that Item 7 on page A1-3 of the reference letter contains an incorrect valve number. Valve number 0-67-553 is incorrectly listed as 0-67-551. This error is a typographical error that was found during development of plant procedures.

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Enclosed are the TVA commitments. This letter is being sent in accordance with NRC Regulatory Issue Summary 2001-05.

If you have any questions about this change, please telephone me at (423) 843-7170 or J. D. Smith at (423) 843-6672.

Sincerely,

Pedro Salas
Licensing and Industry Affairs Manager

I declare under penalty of perjury that the foregoing is true and correct. Executed on this 19 day of February, 2003.

Enclosures
cc (Enclosures):
Mr. Raj K. Anand, Senior Project Manager
U.S. Nuclear Regulatory Commission
MS 0-8G9
One White Flint North
11555 Rockville Pike
Rockville, Maryland 20852-2739

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ENCLOSURE
TENNESSEE VALLEY AUTHORITY
SEQUOYAH NUCLEAR PLANT (SQN)
UNIT 1
DOCKET NO. 327

TVA COMMITMENTS

The following TVA commitments are compensatory actions that will be in effect when making the 22 lifts described in Topical Report 24370-TR-C-002 during the Unit 1 steam generator replacement outage. These commitments will be specifically proceduralized in our procedure for compensatory measures written for this evolution.

1. Any load drop in the zone where the potential for piping damage exists, TVA will commence an orderly shutdown of Unit 2 in accordance with technical specifications.

2. In the event of a load drop in the zone where the potential for piping damage exists, the four alternate supply valves for the standby emergency diesel generator will be opened immediately to preclude damage to the diesel generators if a start signal is generated due to some reason unrelated to the load drop.

3. In the event of a load drop in the zone where the potential for piping damage exists, an immediate operator action is to ensure that there are at least two running ERCW pumps per train.

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