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the southern electric system

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05 APR 21 09:59

April 17, 1986

United States Nuclear Regulatory Commission
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File: X7BD102
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Reference: Vogtle Electric Generating Plant - Unit 1, 50-424,
NRC Report No. 50-424/86-03

Attention: Mr. J. Nelson Grace

Attached is the Georgia Power Company response to the two new unresolved items identified in the NRC Inspection Report 50-424/86-03 regarding Module 8, Structural Steel.

Unresolved item (URI) 86-03-02 pertains to design calculations for the polar crane loading conditions. The response to URI 86-03-02 also contains clarifications which address concerns identified by the NRC inspection teams as a result of the initial response review on February 11, 1986.

Unresolved item URI 86-03-03 pertains to the possible overtightening of high strength bolts in structural steel connections.

The technical explanations and clarifications are provided in the attached response to address the concerns expressed and to clarify the issues raised. Based upon the project engineering technical evaluations, VEGP concludes that the design, hardware, and installation of the polar crane and structural steel bolted connections are adequate and meet the design requirements.

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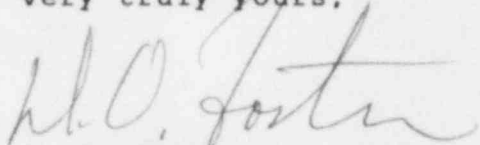
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Mr. J. Nelson Grace
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The design calculations and computer output sheets pertaining to polar crane loading conditions are included in the attachments (1 through 5) to URI item 86-03-02. These calculations are only a part of the complete design for the system they concern, and are subject to being taken out of context, misinterpreted, or misconstrued if used without Bechtel Power Corporation's direct participation. As such, please contact us should the NRC reviewer have any questions pertaining to these attachments. We will arrange to have appropriate personnel from Bechtel address any questions you may have.

This response contains no proprietary information and may be placed in the NRC Public Document Room.

Very truly yours,



D. O. Foster

MRT/DOF/bjd

cc: See Attachment 1

Attachment 1

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Response: As stated on page A2-2 of the seismic report (Reference 1), the "load-up-position" causes the maximum stresses in the polar crane components (girder, girder end connection, rope, and girder pin). The "load-down-position" did not cause the maximum stress in any of the crane components. The reasons are the following:

1. The Y-component of an earthquake along the tangential direction contributes predominantly to the stresses in the crane girder (pages A4-2 and A4-3 of Reference 1). The rope and the lifted load acting as a pendulum with a frequency of about 0.1 hz or less do not contribute much to the crane girder stresses. Thus, the crane girder stresses are not significantly affected by the position of the lifted load (page A4-1 of Reference 1).

The above conclusion is also applicable to the stresses in the girder end connections.

2. The static load and the Z-component of earthquake along the vertical direction contribute predominately to the stresses in the rope. The up-position of the lifted load causes the maximum stress because the corresponding structural frequency of about 2 hz corresponds to the peak accelerations of the vertical input seismic response spectra (Reference 1). Any other position of the lifted load will reduce the structural frequency, thus moving away from the peaks of the spectra.
3. Similar to the rope, the girder pin is affected predominantly by the static load and the vertical component of an earthquake when the lifted load is in the up-position (pages A4-4 through A4-9 of Reference 1).

From the above, it can be concluded that the worst vertical loading condition corresponds to the up-position of the lifted load. An elaborate evaluation has been done for the polar crane under the worst loading condition by performing 20 nonlinear time history analyses using the Engineering Analyses Systems (ANSYS) computer program.

Whiting Corporation (supplier of the polar cranes) completed the seismic qualification by September 1979 and Bechtel approved the qualification by February 1980. The governing specifications were X4AL01, Revision 0, January 10, 1977, and X4AL01, Revision 1, December 14, 1978. These specifications did not specify explicitly the load positions that were required to be considered. Whiting Corporation has considered the following nine load positions: Three trolley positions were considered; mid-span, quarter-span, and end position. For each of the three trolley positions, three lifted load cases were considered; no lifted load, load-up-position and load-down-position. Both Whiting Corporation and Bechtel considered the above nine load positions to be adequate, based on the reasons explained earlier and the experience gained with this type of analysis through the years. However, when the specification X4AL01 was revised later in 1983 as a result of an internal audit, a different set of eight load positions was inadvertently added in paragraph 4.A.3. This will be corrected to be consistent with the seismic report in the next revision to the specification.

The response to URI 424/86-03-02 described above was provided earlier to US NRC Inspectors during their inspection visit of February 10 through 14, 1986, at the Vogtle plant site. As a result of their reviews of the response, the NRC Inspectors made the following comments and requested additional clarifications. The responses to these comments are provided as follows:

Commitment 1:

Statement in item 1 of the Bechtel response on the subject vendor calculation indicates that the rope and lifted load do not contribute much to the crane girder stresses. The statement in item 2 of the Bechtel response indicates that the seismic vertical load is a predominant contributor to the stresses in the rope. Please explain why the load in the rope is not seen by the girder, especially considering the fact that the structural frequency of about 2 hz is so close to the peak seismic response spectra acceleration, shown on page D-12 of 35 of the subject calculations.

Response: Item 1 of the Bechtel response above, concerned itself solely with the Y-component (horizontal component along tangential direction) of the seismic dynamic load and the response was, therefore, correct in stating that the lifted load contributes little to the girder stress. Only the vertical (Z-direction) component of loads will contribute predominantly to rope stresses as stated in Item 2 of URI 424/86-03-02 response above. The Z-direction load is indeed transferred from the rope to the girder. Bending stress corresponding to the vertical seismic load from the rope contributes very little to the total stress in the girder as explained below in response to Comment 2.

Commitment 2:

Refer to the Whiting calculations, page A4-1. Explain why there is no change in the girder stress tabulated in the summary of modal analyses for the (Up and Down) load cases versus no load (NL) cases for both Safe Shutdown Earthquake (SSE) and Operating Basis Earthquake (OBE) conditions. It would be expected that the vertical seismic load in the rope is transferred to the girder, whereby changes in the girder stress should occur.

Response: As mentioned in response to comment 1 above, the vertical or Z-component of load on the rope induces bending of the girder in the vertical plane. However, as pointed out in (A) earlier, the Y-component of the earthquake is the dominant source of stress in the girder; dynamic stress caused by the load along the Y (tangential) direction is greater than that caused by the vertical load. For example, vertical dynamic load induced stress is 8,644 psi which is 31 percent of the tangential dynamic load induced stress of 27,717 psi in the girder under SSE loads. The total dynamic stress, based on the Square Root of the Sum of the Squares (SRSS) combination of stresses from the longitudinal (1,169 psi), tangential, and vertical earthquake components, is 29,057 psi; just 5 percent more than the stress from the tangential load alone. With the addition of static stress (7,808 psi) to the overall dynamic stress, the impact of load position and the

contribution of the vertical dynamic load induced stress on the total stress in the girder becomes less significant.

The seismic report provides the breakdown of computed stresses for the trolley at midspan/load-up case. The computer print-outs pertaining to polar crane design (from Whiting Corporation), which formed the basis for the seismic report, provides the breakdown of stresses for all cases. Attachments 1 and 2 are excerpts from these computer print-outs. Attachment 1 shows the dynamic stresses obtained numerically by analysis of the polar crane with load at midspan/load-down position. Girder stresses induced by longitudinal, tangential, and vertical dynamic loads are 1,194 psi, 27,718 psi, and 6,819 psi, respectively. Those values compare with 1,169 psi, 27,717 psi, and 8,644 psi for the load-up case. Only the vertical load contributions to stress show any appreciable difference; lowering the load reduces the vertical dynamic load induced stress by 21 percent. After SRSS combination of the dynamic stresses, the load-down value is 28,569 psi versus 29,057 psi for the load-up case, only a 2 percent difference. The static load contribution of 7,808 psi is the same for both load positions. Therefore, the total stress adds up to 36,865 psi for load-up and 36,377 psi for load-down, a difference of 1 percent. Thus, the load-up or load-down positions have little influence on the total girder stress as tabulated on page A4-1 of Reference 1.

There is a greater difference between the total stress in the loaded and unloaded cases; total stress is 5 to 15 percent lower in the unloaded case (Reference 1, page A4-1). This comes about largely as a result of the reduction in the static load. Again, the overall dynamic stress is dominated by the tangential contribution and will not be much affected by the absence of the lifted load.

Attachment 2 gives the stresses from the computer runs of the trolley at midspan/no-load case for SSE excitation. The dynamic beam stresses for longitudinal, tangential, and vertical load contributions are 1,292 psi, 27,718 psi, and 7,167 psi, respectively. This is little different from 1,169 psi, 27,717 psi,

and 8,644 psi which are the dynamic stresses for the load-up case. The overall dynamic stresses by SRSS are virtually equal, 28,659 psi for no-load and 29,057 psi for up-load. The significant difference between those two cases comes in the static stresses where the no-load stress of 4,156 psi is 53 percent of the load-up value of 7,808 psi. The resulting total stresses, reported on page A4-1 of Reference 1 are 32,815 psi for no-load stress, which is 11 percent lower than 36,865 psi for load-up stress.

Whereas, there is some contribution to the total midspan stress from the vertical excitation and static load, the contribution due to the vertical loads to the end girder stresses and hence, end connection stresses are insignificant. The reason for this is that the ends of the girder are pinned for vertical loading. The crane girder system acts as a frame in the horizontal plane as can be seen from the sketch of the computer model in Attachment 5. Almost all of the end girder stress results from the tangential excitation. Therefore, the end girder stresses and end connection stresses are insensitive to the presence of lifted load.

Commitment 3:

The natural frequency is stated to be approximately 2 hz for the structure. This 2 hz is in the peak acceleration region of the response spectrum of the lifted load, pages D-11 and D-12, of the report. The statement is made in your response dated February 11, 1986, that lowering of the load will reduce the natural frequency, thereby reducing the seismic acceleration. However, if the natural frequency is to right of the peak, a lowering of the load (and thus, natural frequency) will increase the seismic acceleration. Please provide a calculation on the 2 hz frequency for the Up position and further verify that a reduction in the frequency does reduce the seismic acceleration in the time history analyses (which is the basis of design of the subject polar crane).

Response: The natural frequencies of the polar crane system were obtained by numerical analysis (finite element modal analysis). Those frequencies were summarized by Whiting Corporation and are reproduced in Attachment 3. In addition, an approximate hand calculation of frequencies is provided in Attachment 4 for the load at midspan. In the latter case, the crane girder is modeled as a simple supported beam and the rope is modeled as a spring. The numerical analysis results and hand calculated results are in agreement. In the case where there is no load (trolley at midspan), the numerical analysis results give a fundamental vertical frequency of 4.9 cps; the hand calculated results give 4.6 cps. Adding load to the system increases its mass and reduces the frequency to roughly the 2 cps level. In the loaded condition with the load up, the numerical analysis results give 1.9 cps while the hand calculation give 1.7 cps. When the load is down, both the numerical analysis and the hand calculated results show a reduction in frequency to a value of 1.3 cps.

It should be pointed out that the response spectra on page D-11 and D-12 of Reference 1 represent the response of a single-degree-of-freedom oscillator mounted on the lifted load. Those spectra do not represent the response of the polar crane structure. In fact, the presence of a peak response near 2 cps occurs as a result of the polar crane structure having a vertical mode at around 2 cps.

The basis for seismic analysis of the polar crane girders is the set of in-structure response spectra shown in Appendix B of Reference 1. The spectra were developed from a time-history analysis of the building and were given to Whiting for use in polar crane analysis. Ten sets of time history records were developed to analyze the polar crane structure (page 5-12 of the project specification X4AL01). Collectively, they represent the amplification and frequency content of the spectra in Appendix B. The vertical response spectra are on page B-1 and B-3. They show the peak response to lie in the range from 2 cps to 10 cps. Below 2 cps, the response is a

descending ramp. As the load is lowered and frequency is reduced, the crane will experience progressively lower levels of amplification in response.

Reference:

1. Whiting Corporation Seismic Report
Summary of Seismic Analysis and Supplementary
Calculations, Containment Building Polar
Crane, Bechtel Log No. AX4AL01-46-2

Response: The following evaluation is provided in response to NRC concern expressed relative to possible overtorquing of high strength bolts in a structural steel connection during the site inspection visit of January 1986.

The evaluation includes a response to the specific concerns noted by the Quality Control (QC) inspector due to a lack of conformance to the turn-of-nut requirements. The evaluation also includes a project engineering review of the effects of overtorquing on design requirements.

A) Response to QC inspector concerns on conformance to turn-of-nut method:

A statement was made by the Georgia Power Company (GPC) structural steel inspector that at one time, he had witnessed some irregularity in the installation of high strength bolts in a steel connection. He had observed that iron workers installed bolts in the connection and then improperly placed match marks on the steel to indicate a properly installed bolt. The subject connection was redone in his presence. As the result of the incident, GPC inspection supervisory personnel contacted the Ingalls Ironworks, supervisory personnel. It was decided, in the meeting among the supervisory personnel that immediate steps would be taken to ensure that the Ingalls Ironworks' bolt installation program was upgraded to eliminate any practices that would result in improper installation of high strength bolts. The incident, according to the recollection of the GPC inspector, had occurred around January 1982.

Based on the discussion as stated above, the NRC inspector has raised a concern in this unresolved item that inspection program for high strength bolting was inadequate to identify potential overtorquing of high strength bolts. In response to this concern a review of procedures and inspection reports relative to bolting was initiated. The results of this review are discussed as follows:

A review of procedure CD-T-16 revealed that the turn of the nut method for structural steel bolting was initiated on Revision 3 dated July 23, 1982. Since this is several months later than the time frame noted in the inspectors statement, a review was performed of the installation records which noted that girder No. 5 in the Control Room, elevation 240 ft, was installed in August 1982. To verify that training was conducted as noted by the inspectors statement, a review was performed of the Ingalls Ironworks training records which noted that all Ingalls iron workers were given a training course on turn-of-nut installation methods September 8, 1982.

This review has concluded that the situation noted in the inspectors statement occurred within the first month of site initiation of the turn-of-nut method for structural steel bolting. Therefore, it is not surprising that problems were noted. The fact that problems were noted is indicative that GPC QC was closely monitoring the application of the new method to ensure that compliance was adequate. The corrective action taken by site management and Ingalls to replace the identified bolts is indicative of the site concern for ensuring that bolts were properly installed in accordance with the technical requirement.

The time frame from July 23, through September 8, 1982, indicates a period prior to Ingalls' training program. This clearly makes the application of the turn-of-nut method suspect and overtorque a potential problem. However, as indicated in the second paragraph, it is obvious that a close surveillance and inspection was being performed by GPC QC to ensure the proper application of the turn-of-nut methods by the craft. To further support that overtorque was not a problem during this time frame, if a bolt was overtorqued to a point that the proof load was not obtained, then the verification performed by GPC QC, with a calibrated torque wrench, would identify this. This identification would be made by noting the bolt turning prior to reaching the setting on the torque wrench and all bolts would then be verified with the torque wrench. Also, if additional torque is applied to the bolt, it will either fail or will continue not to pass the torque wrench verification.

It must also be noted that during the use of an impact wrench, when the failure range of a high strength bolt is approached, the tension applied decreases. This increases the speed at which the impact wrench operates to a point that the failure of a bolt would proceed so rapidly that the craft would not have time to react and terminate the application of torque prior to failure.

A further review was performed of site audits by GPC Quality Assurance (QA), NRC, and other groups such as Institute of Nuclear Power Operations (INPO) and Self-Initiated Evaluation (SIE) to determine whether the structural steel bolting installation program was conducted and whether problems with the craft's application of the turn-of-nut installation methods were in compliance with American Institute of Steel Construction (AISC) requirements. Several audits were noted as being conducted, but none cited problems with craft practice for conformance with the turn-of-nut method. Rather, their application to be in compliance was noted. These audits were conducted on a continuing basis from 1982 until 1986. It should also be noted that GPC QC noted no further problems with the craft incorrectly applying the turn-of-nut verification marking as noted in the QC inspectors statement. The GPC QC program consisted of a 100 percent verification of the proper torquing marks, a torque wrench verification of two bolts or 10 percent of the bolts in each connection to ensure minimum torque, and a surveillance program to ensure craft compliance with the turn-of-nut method during the installation process.

Therefore, it is the Project conclusion that the structural steel bolts, as installed, are in compliance with the turn-of-nut method, as stated in AISC requirements, and that overtorquing is not a concern.

B) Project engineering evaluation of effect of overtorquing on design requirements:

General:

High strength bolts mainly of the types A-325 and A-490 are used in structural steel connections (joints). The design is in accordance with the 1969 AISC specification (Reference 1) that implies factors of safety of at least 3 and 5, respectively, against ultimate tension and shear failures in the bolts. Most commonly used bolts are 7/8-in.

diameter A-325 type. The connected steel surfaces are blast-cleaned and coated with inorganic zinc paint.

The bolts are installed by the turn-of-the-nut-method (Reference 2). Accordingly, a tensile preload corresponding to the proof load is applied to each bolt by turning the nut after snug tight by 1/3 to 2/3 turns, depending on the grip length of the bolted connection. This write up provides the evaluation of possible overtorquing (turning the nut more than the specified amount) which may have taken place during a period from July 23, through September 8, 1982.

Possible Consequences of Overtorquing:

A treatise on bolted joints is provided in the 1974 book by J. W. Fisher and J. A. Struik (Reference 3). Chapter 4 of this book provides the discussion on behavior of bolts including the significance of torquing. Review of this reference, and others, indicates that the ultimate strength of the joint under external loads is not affected by overtorquing. (Of course, if sufficient overtorquing is applied, the bolt would fracture and would be replaced.) This conclusion about the joint strength is discussed separately below for tension and shear.

Tension:

When a bolt is torqued, the induced tensile force in the bolt will reach a peak value after about 3/4 nut rotation. This peak torque induced tension is, on the average, about 15 percent below the ultimate direct tension value. The reduction may be attributed to combined torsional and tensile stresses in the bolt while torquing. If torquing is continued, the torque induced tension decreases. The probability of overtorquing a bolt, so that the induced tension is below the proof load, is very remote. In such a case, the bolt would probably be torqued to failure since the tension in the bolt (and thus the torque required to generate that force) decreases as the torquing continues. In addition, the bolt would not have passed the inspection program which verifies the preload using a calibrated torque wrench.

However, if the bolt is loaded in direct tension after tightening the nut, the ultimate tensile strength of the bolt will not decrease as illustrated in Figures 4.7 and 4.8 of Reference 3. The reason for developing the full tensile strength (even after overtorquing) may be attributed to lack of torsional stresses when the bolt is loaded in direct tension. (Any torsional stress that may be locked-in as a result of torquing does not appear to have any impact.) This means that bolts installed by torquing can sustain direct tension loads without any apparent reduction in their ultimate tensile strength.

Shear:

Tests performed on A-325 and A-490 bolts torqued to various degrees of tightness have shown that the initial clamping force had no significant effect on the ultimate shear strength. (Reference 3, Figure 4.16) The following two reasons are given in Reference 3 for this phenomenon: (1) The critical shear plane is often through the bolt shank, while the critical plane for tension is through the threaded portion. Because of the increased area, the tensile stress in the shank is significantly lower than that in the threaded portion; consequently, the tensile stress has minimal effect on the shear strength. (2) Measurements on test specimens have shown that at the ultimate shear load level there is little clamping force left in the bolt. Thus, the joint can sustain the design shear forces, regardless of the magnitude of the pretension induced by torquing.

Conclusion:

The ultimate tensile and shear strengths of a bolt are not affected by the magnitude of preload and deformation from overtorquing. Thus, the integrity of the bolted connections and adequate factors of safety against tensile and shear failures are ensured. Therefore, the 1966 version of Reference 2 states, "...if the fastener does not fail while being installed, it will not fail thereafter...". The later versions (1972 and later) state the same with an editorial change as, "...if the fastener does not fail while being installed, it should not fail thereafter..." It appears that the editorial change is made to be consistent with the standard specification language. Also, there is no known instances of problems reported in the technical literature with overtorqued bolts.

Therefore, it is concluded that any inadvertent overtorquing of bolts in the VEGP structural steel connections has not compromised the integrity and design adequacy of structural steel connections.

References:

- 1) Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, adopted February 12, 1969, and includes supplements 1, 2, and 3, American Institute of Steel Construction (AISC).
- 2) Specification for Structural Joints Using ASTM A-325 or A-490 Bolts, Research Council on Riveted and Bolted Structural Joints.
- 3) Fisher, J. W. and Struik, J. H. A., Guide to Design Criteria for Bolted and Riveted Joints, John Wiley & Sons, 1974.

71136-BECHTEL FOR GEORGIA POWER-TROLLEY @ MID SPAN-LOAD DOWN (150 FT) SSE

SRSS OF LOAD STEP STRESSES AT I		SRSS			
LOAD STEP		1	2	3	
ELEM	TYPE				
3	4	1.0684	54.981	1.5600	55.016
4	4	0	74.832	1.2549	74.843
5	4	544.00	40129.	216.28	40133.
6	4	164.90	1537.7	46.946	1547.3
7	4	186.07	745.92	1160.3	1391.8
8	4	187.96	756.36	1212.2	1441.1
9	4	1.0018	0.	0.	1.0018
10	4	8.7887	0.	3.6099	9.5012
11	4	19.254	33.784	59.757	71.295
12	4	13.535	37.716	66.225	77.404
13	4	19.622	25.754	59.618	67.842
14	4	14.353	45.524	66.519	81.873
15	4	6.6172	210.16	15.229	210.82
16	4	18.135	9.1395	20.494	28.852
17	4	4.9203	6.5195	15.092	17.160
18	4	18.424	208.17	21.252	210.06
20	8	2013.2	3.0710	33293.	33353.
22	4	0.	0.	0.	0.
24	4	1114.4	30817.	597.47	30843.
25	4	1063.6	29530.	478.36	29553.
26	4	1066.5	6859.9	4583.3	8318.7
27	4	1166.2	5211.0	4655.6	7084.4
28	4	1194.1	27718.	6818.9	28569.
29	4	1209.1	23043.	6867.4	24075.
30	4	1037.5	24612.	6683.2	25525.
31	4	1031.8	24603.	6714.9	25524.
32	4	1023.7	22967.	6786.5	23970.
33	4	940.19	27603.	6786.9	28440.
34	4	858.97	4495.2	4341.5	6308.3
35	4	788.20	6664.4	4317.3	7979.6
38	4	0.	72.335	0.	72.335
39	4	1.0508	58.063	0.	58.072
40	4	344.02	37287.	91.670	37288.
41	4	164.94	1775.4	81.317	1784.9
42	4	164.55	790.56	1204.1	1449.8
43	4	167.77	789.96	1216.5	1460.1

ATTACHMENT 1
 SHEET 1 OF 2

NOTICE
 THESE DESIGN CALCULATIONS ARE ONLY AN ISOLATED PART OF THE COMPLETE DESIGN FOR THE SYSTEM THEY CONCERN, AND ARE SUBJECT TO BEING TAKEN OUT OF CONTEXT, MISINTERPRETED OR MISCONSTRUED IF USED WITHOUT BECHTEL POWER CORPORATION'S DIRECT PARTICIPATION.

71136-BECHTEL FOR GEORGIA POWER-TROLLEY MID SPAN-LOAD DOWN (150 FT) SSE

SRSS OF LOAD STEP STRESSES AT J					
LOAD STEP		1	2	3	SRSS
ELEM	TYPE				
3	4	0.	37.396	1.4833	37.426
4	4	0.	57.643	1.1428	57.654
5	4	164.93	1536.2	46.953	1545.8
6	4	362.22	37230.	207.57	37233.
7	4	186.04	742.22	1160.2	1389.8
8	4	187.95	752.66	1212.2	1439.2
9	4	8.7880	0.	3.4328	9.4347
10	4	1.0011	0.	0.	1.0011
11	4	19.194	25.802	59.753	67.857
12	4	12.251	37.678	65.888	76.882
13	4	19.622	25.754	59.618	67.842
14	4	13.065	37.688	66.152	77.248
15	4	8.4204	221.42	20.939	222.56
16	4	22.636	12.850	27.611	37.946
17	4	6.7773	8.9801	20.788	23.637
18	4	22.962	218.06	28.375	221.10
20	8	2013.2	3.0710	33293.	33353.
22	4	0.	0.	0.	0.
24	4	1066.4	6859.9	4583.1	8318.7
25	4	1147.0	4975.0	4594.0	6868.2
26	4	1194.1	27718.	6818.9	28569.
27	4	1209.1	23043.	6867.4	24075.
28	4	1182.7	24611.	6773.1	25554.
29	4	1140.9	24639.	6799.7	25585.
30	4	1115.0	22969.	6760.8	23469.
31	4	1047.5	27605.	6745.6	28436.
32	4	858.97	4495.2	4341.5	6308.3
33	4	788.20	6664.4	4317.3	7979.6
34	4	293.10	29879.	91.021	29881.
35	4	390.90	31474.	157.52	31476.
38	4	0.	54.948	0.	54.948
39	4	0.	40.373	0.	40.373
40	4	164.98	1777.8	80.937	1787.3
41	4	538.85	40705.	243.72	40710.
42	4	164.54	786.96	1204.1	1447.9
43	4	167.74	786.20	1216.5	1458.1

11136-BECHTEL FOR GEORGIA POWER-TROLLEY MID SPAN-NO LOAD 170 FT SSE

SRSS OF LOAD STEP STRESSES AT 1					
LOAD STEP ELEM	STEP TYPE	1	2	3	SRSS
3	4	1.1105	54.983	1.3821	55.012
4	4	0	74.832	1.0970	74.840
5	4	555.59	40129.	322.73	40134.
6	4	180.75	1537.7	100.17	1551.6
7	4	201.69	746.01	1232.7	1454.9
8	4	204.23	756.27	1291.2	1510.2
9	4	1.0023	0.	0.	1.0023
10	4	8.7926	0.	3.8912	9.6152
11	4	19.590	33.790	47.377	61.401
12	4	14.102	37.723	57.793	70.440
13	4	19.937	25.748	46.837	57.045
14	4	14.920	45.517	58.516	75.621
15	4	7.8666	210.16	11.901	210.65
16	4	18.197	9.1415	19.984	28.532
17	4	5.0469	6.5180	11.856	14.441
18	4	19.860	208.17	20.649	210.13
20	4	0	0	0	0
22	4	1147.7	30817.	714.13	30847.
23	4	1080.7	29530.	558.51	29555.
24	4	1149.3	6859.5	4963.0	8544.4
25	4	1224.3	5211.3	5058.9	7365.4
26	4	1292.3	27718.	7166.8	28659.
27	4	1314.6	23042.	7225.7	24185.
28	4	1142.5	24613.	6998.1	25614.
29	4	1137.7	24603.	7039.7	25615.
30	4	1131.4	22967.	7136.2	24077.
31	4	1047.2	27602.	7119.2	28525.
32	4	926.45	4494.9	4693.0	6564.1
33	4	864.17	6664.7	4646.4	8170.3
36	4	0	72.335	0.	72.335
37	4	1.0919	58.063	0.	58.073
38	4	349.57	37287.	141.31	37289.
39	4	180.72	1775.4	110.59	1788.0
40	4	182.60	790.64	1282.1	1517.3
41	4	185.57	789.87	1296.6	1529.6

NOTICE

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71136-BECHTEL FOR GEORGIA POWER-TROLLEY @ MID SPAN-NO LOAD (170 FT) SSE

SRSS OF LOAD STEP STRESSES AT J					
LOAD STEP ELEM	TYPE	1	2	3	SRSS
3	4	0.	37.397	1.3430	37.421
4	4	0.	57.643	1.0194	57.652
5	4	180.67	1536.2	100.19	1550.0
6	4	368.71	3723.0	263.48	3723.0
7	4	201.66	742.31	1232.7	1453.0
8	4	204.22	752.57	1291.2	1508.4
9	4	8.7919	0.	3.6984	9.5381
10	4	1.0016	0.	0.	1.0016
11	4	19.530	25.808	47.373	57.374
12	4	12.866	37.684	57.264	69.748
13	4	19.937	25.748	46.837	57.045
14	4	13.678	37.681	57.965	70.476
15	4	9.8061	221.42	16.433	222.24
16	4	22.726	12.852	26.691	37.337
17	4	6.9517	8.9780	16.331	19.891
18	4	24.476	218.06	27.407	221.13
20	4	0.	0.	0.	0.
22	4	1149.3	6859.6	4962.9	8544.3
23	4	1209.6	4975.3	5029.5	7177.2
24	4	1292.3	27718.	7166.8	28659.
25	4	1314.6	23042.	7225.7	24185.
26	4	1283.7	24612.	7115.7	25652.
27	4	1242.4	24638.	7132.7	25680.
28	4	1220.7	22969.	7102.7	24074.
29	4	1152.8	27604.	7079.2	28521.
30	4	926.45	4494.9	4693.0	6564.1
31	4	864.17	6664.7	4646.4	8170.3
32	4	295.73	29879.	140.77	29881.
33	4	396.63	31474.	232.72	31477.
36	4	0.	54.948	0.	54.948
37	4	0.	40.373	0.	40.373
38	4	180.85	1777.8	110.17	1790.4
39	4	551.90	40705.	342.64	40711.
40	4	182.60	787.05	1282.1	1515.4
41	4	184.57	786.11	1296.6	1527.5

11136-BECHTEL FOR GEORGIA POWER-TROLLEY & MID SPAN-NO LOAD STATIC

SRSS OF LOAD STEP STRESSES AT 1
LOAD STEP 1 SRSS

ELEM	TYPE	SRSS	SRSS
3	4	11.760	11.760
4	4	12.081	12.081
5	4	93.719	93.719
6	4	278.66	278.66
7	4	1007.7	1007.7
8	4	1047.4	1047.4
9	4	0.	0.
10	4	0.	0.
11	4	29.185	29.185
12	4	28.242	28.242
13	4	28.242	28.242
14	4	29.186	29.186
15	4	7.4433	7.4433
16	4	7.1399	7.1399
17	4	7.1492	7.1492
18	4	7.4526	7.4526
20	4	0.	0.
22	4	39.536	39.536
23	4	41.790	41.790
24	4	2778.7	2778.7
25	4	2820.5	2820.5
26	4	4155.8	4155.8
27	4	4179.9	4179.9
28	4	4170.5	4170.5
29	4	4198.2	4198.2
30	4	4145.5	4145.5
31	4	4177.1	4177.1
32	4	2757.2	2757.2
33	4	2771.6	2771.6
36	4	12.328	12.328
37	4	12.451	12.451
38	4	94.247	94.247
39	4	289.32	289.32
40	4	1013.5	1013.5
41	4	1021.6	1021.6

71136-BECHTEL FOR GEORGIA POWER-TROLLEY & MID SPAN-NO LOAD STATIC

SRSS OF LOAD STEP STRESSES AT J

LOAD STEP	ELEM	TYPE	1	SRSS
	3	4	11.773	11.773
	4	4	12.050	12.050
	5	4	278.66	278.66
	6	4	97.724	97.724
	7	4	1007.5	1007.5
	8	4	1047.2	1047.2
	9	4	0.	0.
	10	4	0.	0.
	11	4	29.183	29.183
	12	4	28.242	28.242
	13	4	28.242	28.242
	14	4	29.183	29.183
	15	4	10.232	10.232
	16	4	9.8407	9.8407
	17	4	9.8474	9.8474
	18	4	10.234	10.234
	20	4	0.	0.
	22	4	2778.6	2778.6
	23	4	2820.4	2820.4
	24	4	4155.8	4155.8
	25	4	4179.9	4179.9
	26	4	4170.5	4170.5
	27	4	4198.2	4198.2
	28	4	4145.7	4145.7
	29	4	4177.0	4177.0
	30	4	2757.2	2757.2
	31	4	2771.6	2771.6
	32	4	7.0606	7.0606
	33	4	6.4226	6.4226
	36	4	12.338	12.338
	37	4	12.435	12.435
	38	4	289.32	289.32
	39	4	80.648	80.648
	40	4	1013.3	1013.3
	41	4	1071.5	1071.5

NATURAL FREQUENCIES
 SSE TROLLEY @ MID

NO LOAD

Frequency Hz	Mode Coefficient		
	X	Y	Z
3.096	.4997	73.79	.1341
4.812	2.814	8.450	.2090
4.917	2.640	.7511	40.66
9.488	.04623	.7685	.00486
10.09	.1242	.09119	.02055
13.13	.2258	1.004	.03405
15.36	.09798	.6050	.01222
16.07	1.194	.03146	.1218
17.75	.01042	3.230	.00030
23.63	.01285	1.871	.00150
23.92	.03689	.6231	.00217

LOAD UP

Frequency Hz	Mode Coefficient		
	X	Y	Z
1.873	10.35	2.197	394.4
3.096	.4994	73.79	.1457
4.812	2.747	8.466	.8570
5.302	2.233	.4679	24.52
9.488	.04666	.7685	.00422
10.09	.1238	.0911	.02107
13.13	.2257	1.009	.03410
15.36	.09749	.6050	.01222
16.08	1.193	.03140	.1223
17.75	.01043	3.230	.00031
23.63	.01289	1.871	.00149
23.92	.03689	.6231	.00217

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WHITING REON 71136 DATE 6/12/78

BY MJM PAGE A-7 OF A-9

NATURAL FREQUENCIES -SSE

Trolley @ Mid Span
Load Down

Frequency Hz	Mode Coefficient		
	X	Y	Z
1.291	9.509	2.023	542.9
3.096	.4996	73.79	.1384
4.812	2.763	8.463	.6017
5.074	2.430	.5545	33.23
9.488	.04641	.7685	.00460
10.09	.1241	.09116	.02077
13.13	.2258	1.004	.03407
15.36	.09777	.6050	.01222
16.07	1.193	.03144	.1220
17.75	.01042	3.230	.00030
23.63	.01285	1.871	.00150
23.92	.03689	.6231	.00217



CALCULATION SHEET

PROJECT VEGP JOB NO. 9510 CALC. NO. _____

SUBJECT _____ SHEET NO. _____

REV	ORIGINATOR	DATE	CHECKER	DATE	REV	ORIGINATOR	DATE	CHECKER	DATE
1	<i>[Signature]</i>	4/14/86	P. Shumway	4/14/86					
2									

NATURAL FREQUENCY OF THE POLAR CRANE

(REF.: POLAR CRANE SEISMIC REPORT, BEC Log No. AX4AL01-46-2)

GIRDER SPAN = 120 FT (REFERENCE, p. A3-3)
 AREA = 474 in² (" " p. A3-9)
 I_{xx} = 1,405,449 in⁴ (" " ")

WEIGHT OF CRANE GIRDER = (474 in²) (12 in) (0.49 k/ft³ / 1728 in³/ft³) = 1.61 k/ft
 (ADD 10% MARGIN)

GIRDER WEIGHT = 1.1 x 1.61 k/ft = 1.77 k/ft ≈ 2 k/ft

WEIGHT OF TRUSS = 229.7 k (REFERENCE, p. A3-11)
 WEIGHT OF LIFT LOAD = 225 TONS (2 k/TON) = 450 k (" "
 WEIGHT OF BLOCK = 5 TONS (2 k/TON) = 10 k (" ")

TOTAL LOAD PER GIRDER = (229.7 k + 450 k + 10 k) / 2 GIRDERS = 345 k

f₁ = FREQUENCY OF GIRDER WITH UNIFORM DEAD LOAD
 f₂ = FREQUENCY OF GIRDER WITH POINT LOAD AT MIDSPAN

$$f_1 = \frac{9.88}{2\pi} \sqrt{\frac{EI_g}{W L^4}} = \frac{9.88}{2\pi} \sqrt{\frac{(30 \times 10^3 \text{ ksi})(1.41 \times 10^6 \text{ in}^4)(386.4 \text{ in/ft}^3)}{(2 \text{ k/ft})(12 \text{ ft/in}) [(120 \text{ ft})(12 \text{ in/ft})]^4}}$$

= 6.4 cps

$$f_2 = \frac{1}{2\pi} \sqrt{\frac{48EI_g}{W L^3}} = \frac{1}{2\pi} \sqrt{\frac{(48)(30 \times 10^3 \text{ ksi})(1.41 \times 10^6 \text{ in}^4)(386.4 \text{ in/ft}^3)}{(345 \text{ k}) [(120 \text{ ft})(12 \text{ in/ft})]^3}}$$

= 3.9 cps

FREQUENCY OF THE LOADED STRUCTURE WITHOUT ROSE EFFECT, LOAD AT MIDSPAN

$$\frac{1}{f^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2} = \frac{1}{(6.4)^2} + \frac{1}{(3.9)^2}$$

f = 3.33 cps

ATTACHMENT 4
SHEET 1 OF 3

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REV. INDICATOR



CALCULATION SHEET

PROJECT VEEP JOB NO. 9510 CALO. NO. _____

SUBJECT _____ SHEET NO. _____

REV	ORIGINATOR	DATE	CHECKER	DATE	REV	ORIGINATOR	DATE	CHECKER	DATE
△	...	4/14/86	P. Stumm...	4/14/86	△				
△					△				

NATURAL FREQUENCY OF THE POLE (CONT'D)

UNLOADED WEIGHT (TRAY AND BLOCK ALONE) = 239.7 K + 10K = 239.7 K
 " " PER GIRDER = 239.7 K / 2 = 120 K

f_L' = FREQUENCY OF GIRDER IN NO-LOAD CONDITION, TRAY AND BLOCK AT MIDSPAN

$$f_L' = \frac{1}{2\pi} \sqrt{\frac{48EI^3}{WL^3}} = \frac{1}{2\pi} \sqrt{\frac{(49)(30 \times 10^3 \text{ ksi})(1.41 \times 10^6 \text{ in}^4)(286.4 \text{ W/SEC}^2)}{(120 \text{ K})[(120 \text{ FT})(12 \text{ IN/FT})]^3}}$$

$$= 6.6 \text{ cps}$$

FREQUENCY OF STEELWORK, "NO LOAD" CONDITION

$$\frac{1}{f^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2} = \frac{1}{(6.4)^2} + \frac{1}{(6.6)^2}$$

$$\underline{\underline{f = 4.6 \text{ cps}}}$$

EFFECT OF ROPE FLEXIBILITY:

ROPE AREA = 11.52 in² (REFERENCE, P. A3-11)
 E = 14 x 10³ ksi (" " "
 ROPE LENGTH,
 LOAD UP L = 50 FT (" " P. A4-4)
 LOAD DOWN L = 150 FT (" " P. A4-5)

ROPE STIFFNESS, $k = \frac{EA}{L}$

f_2 = FREQUENCY OF TANK ROPE IN LONGITUDINAL MOTION



CALCULATION SHEET

Attachment 4 W-16
(Page 3 of 3)

PROJECT VEGP JOB NO. 9510 CALC. NO. _____

SUBJECT _____ SHEET NO. _____

REV	ORIGINATOR	DATE	CHECKER	DATE	REV	ORIGINATOR	DATE	CHECKER	DATE
△	<i>[Signature]</i>	4/14/86	<i>[Signature]</i>	4/14/86	△				
△					△				

REV. INDICATOR

NATURAL FREQUENCY OF THE PULVE CRANE (CONT'D)

WEIGHT OF LIFT LEAD AND BLOCK = 450 K + 10 K = 460 K

Rope frequency,

$$f_3 = \frac{1}{2\pi} \sqrt{\frac{k}{m}} = \frac{1}{2\pi} \sqrt{\frac{EA}{L} \frac{g}{W}} = \frac{1}{2\pi} \sqrt{\frac{(14 \times 10^6 \text{ psi})(11.52 \text{ in}^2)(32.2 \text{ ft/sec}^2)}{(460 \text{ K}) L}}$$

$$= \frac{16.9}{\sqrt{L}}$$

L = 70 ft → $f_3 = \frac{16.9}{\sqrt{70}} = 2.0 \text{ cps}$

L = 150 ft → $f_3 = \frac{16.9}{\sqrt{150}} = 1.4 \text{ cps}$

FREQUENCY OF LOWER STRUCTURE, LEAD UP, LEAD AT MIDSPAN

$$\frac{1}{f^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2} + \frac{1}{f_3^2} = \frac{1}{(6.0)^2} + \frac{1}{(2.9)^2} + \frac{1}{(2.0)^2}$$

f = 1.7 cps

FREQUENCY OF LOWER STRUCTURE, LEAD DOWN, LEAD AT MIDSPAN

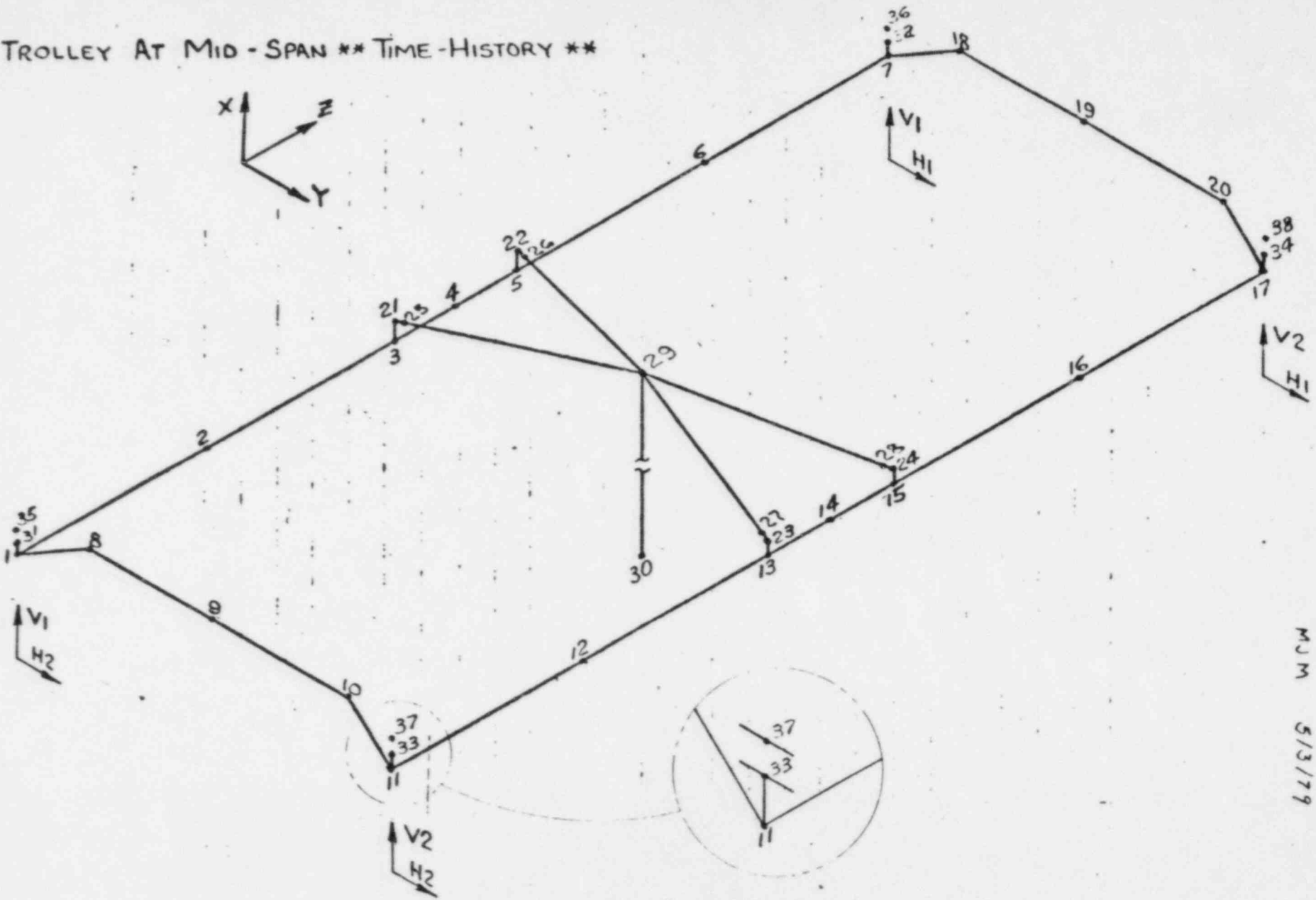
$$\frac{1}{f^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2} + \frac{1}{f_3^2} = \frac{1}{(6.4)^2} + \frac{1}{(3.9)^2} + \frac{1}{(1.4)^2}$$

f = 1.3 cps

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BECHTEL FOR GEORGIA POWER COMPANY

TROLLEY AT MID-SPAN ** TIME-HISTORY **



INTERFACE ELEMENT

ATTACHMENT 5
 SHEET 1 OF 1

WRITING REQN. 71136 ATTACHMENT 5
 BY J. LUSTYK PAGE 3.5 OF 6
 MUM 5/3/79 DATE 1-2-79