NEDO-21326

APPENDIX B-2

FOUNDATION INVESTIGATION

DECEMBER 1967

Morris Operation
General Electric Company
Morris, Illinois

DAMES & MOORE

309 WEST JACKSON BOULEVARD
PARTNERS
ASSOCIATES

December 13, 1967

The Fluor Corporation, Ltd.
P. O. Box 7030 East Los Angeles Branch
Los Angeles, California 90022

Attention: Mr D C. Van Dine,
Project Manager

Gentlemen:

Ten copies of our "Report of Foundation Investigation, Proposed FRO Plant Project, Near Morris, Grundy County, Illinois for the General Electric Company" are herewith submitted.

The scope of our studies was planned in collaboration with Messrs. D. C. Van Dine, R. V. Bierman, D. C. Morrison, and F. Burrows of The Fluor Corporation, Ltd. Our work was performed under The Fluor Corporation, Ltd. Purchase Order No. 4204-0-003 dated October 20, 1967 and Supplement No. 1 to Purchase Order No. 4204-0-003 dated November 1, 1967.

A rough draft of this report was submitted to you for your review and comments on December 1, 1967. Should you have any questions regarding the contents of this report, subsequent to your final review of it, please do not hesitate to contact us.

Yours very truly,

DAMES & MOORE

James B. Thompson

James B. Thompson

JBT:MLK:ac

REPORT OF FOUNDATION INVESTIGATION

PROPOSED FRO PLANT PROJECT

NEAR MORRIS, GRUNDY COUNTY, ILLINOIS

FOR THE

GENERAL ELECTRIC COMPANY

SCOPE

This report presents the results of our Foundation Investigation performed at the site of the Proposed FRO Plant Project to be constructed near Morris in Grundy County, Illinois for the General Electric Company. The site is located in Section 35 of Township 34 North-Range 8 East and is shown with respect to the center of Section 35 and the existing Dresden Power Plant on the Key Plan shown on Plate 1, Plot Plan. The locations of the proposed structures are also shown with respect to the center of Section 35 on Plate 1.

The purposes of our foundation investigation were as follows:

1 - To explore the subsurface soil, rock and ground water conditions at the site of the proposed construction to the depths which will be significantly affected by the proposed construction.

2 - To evaluate, by the performance of laboratory tests, the strength and compressibility characteristics of the soil and rock which underlie the site that will influence foundation design and construction.

3 - To select the type or types of foundations which, in our opinion, will provide satisfactory support for the proposed construction.

4 - To provide the data required for the design of suitable foundation types including allowable bearing pressures and anticipated movements.

5 - To provide data regarding lateral soil and water pressures to be utilized in design of substructures.

6 - To provide recommendations regarding earthwork operations that will be required at the site including stripping, excavating, and the placement and compaction of fill materials.

7 - To provide data required for the design of pavements.

8 - To investigate possible sources of backfill material and obtain preliminary cost data for suitable backfill material.

9 - To provide our opinion as to the feasibility of the utilization of a leach field in connection with a sewage treatment plant.

The results of our field explorations and laboratory tests, which form the basis of our recommendations, are presented in Appendix A of this report.

STRUCTURAL CONSIDERATIONS

The Proposed FRO Plant Project will include the following facilities: Main Process Building with attached vaults, Sand Filter, Exhaust Fan and Emergency Equipment Building, Shop and Warehouse Building, Utility and Service Building, Fluorine Building, Administration Building, Stack, miscellaneous appurtenant facilities, railroad, roadways and parking lot. The location of the

various facilities are shown on Plate I. A summary of the approximate plan dimensions and foundation elevations are presented below:

FACILITY	APPROXIMATE PLANT DIMENSIONS FEET	APPROXIMATE FOUNDATION ELEVATION* FEET
Main Process Building	85 X 180	500 to 530
Cladding Vault	30 X 80	476
Asphalt Vault	55 X 85	476
Chemical Vault	30 X 35	483
Exhaust Fan and Emergency Equipment Building	15 X 75	**
Sand Filter	75 X 75	518 to 522
Shop and Warehouse Building	50 X 70	**
Utility and Service Building	50 X 70	**
Fluorine Building	35 X 70	**
Administration Building	60 X 70	**

*All Elevations presented in this report refer to U.S.G.S. Datum.
**Established near the final grade.

We understand that the final grade at the site may be raised by approximately four feet by the placement of controlled compacted granular fill.

We understand that the Main Process Building will consist of waste and fuel storage basins, a central process area called the "canyon" and personnel or operating areas surrounding the "canyon." The waste and fuel storage basins will be filled with water and will have foundation loads ranging from approximately 6,000 to 8,000 pounds per square foot. The "canyon" is a heavy walled

concrete structure approximately 30 feet by 100 feet in plan dimensions and 90 feet in height. The foundation loading in the "canyon" will range from approximately 8,000 to 10,000 pounds per square foot. Operating levels surrounding the "canyon" will be of conventional steel and concrete construction with column loads on the order of 50,000 to 200,000 pounds.

A one hundred twenty-five ton cask unloading crane will be installed on the northwest corner of the Main Process Building. The crane will ride on rails approximately 40 feet above the ground and will deliver loads on the order of 200,000 to 300,000 pounds to the frame support foundations.

The storage vaults which will be utilized for storage of radioactive waste material will have foundation loads on the order of 6,000 to 8,000 pounds per square foot. We understand that it is presently planned to over excavate the rock adjacent to the storage vaults to facilitate the enlargement of the vaults at a later date. The over excavated area will be backfilled at the present time during construction of the presently proposed storage vaults.

The sand filter which will be utilized for final filtration of air prior to exiting the plant will impose a foundation pressure of approximately 2,000 pounds per square foot to the supporting rock.

The remainder of the structures will be of steel construction, will be one-story in height, and will be relatively light. There will also be miscellaneous foundations for equipment and tanks.

We understand that a stack to be constructed at approximately coordinates South 950 feet and East 410 feet will be approximately 300 feet in height and will be approximately four feet in diameter. The stack will be designed to resist wind loads of up to approximately 110 miles per hour and will be guyed.

We also understand that the proposed pavements will be subjected to H-20 loading.

SITE CONDITIONS

The ground surface at the site is relatively level with some minor surface undulations except in the south-west corner of the site where the topography slopes gently to the southwest. The ground surface elevation ranges from approximately 530 to 534 and slopes downward to approximately elevation 527 in the southwest corner. The surface vegetation consists of grass and weeds east of approximately coordinate East 350 feet. The portion of the site which is west of this coordinate is under cultivation and the soybean crop had been harvested prior to our field investigation. A few trees are located approximately along coordinate East 350 feet.

The subsurface conditions at the site were investigated by drilling ten test borings and by excavating 55 additional test borings at the locations shown on Plate 1. A summary of the ground surface elevation, thickness of soil, elevation of the bedrock surface, and the type of rock encountered at each boring location is also presented on Plate 1.

The test borings revealed that the site is blanketed by a layer of soil which ranges in thickness from approximately 4 to 42 inches and which has an average thickness of approximately 12 inches. The soil rests on an undulating bedrock surface which has resulted from erosion. The soil generally consists of topsoil which has low strength characteristics and is compressible. Where a greater thickness of soil is present, the soil generally grades with depth to moderately firm, compressible silty clay.

At the location of Boring 61 (near the intersection of the Proposed Railroad and the County Road), a 6.8 foot thick layer of soil cover was encountered which was moderately firm and was underlain by shale. Boring 60 was approximately ten feet south of Boring 61 and sandy limestone was encountered at a shallow depth (0.5 feet) in Boring 60. The difference in the thickness of the overburden soils and in the type of bedrock encountered in Borings 60 and 61 can be explained by erosion since the shale in Boring 61 is encountered at approximately the same elevation as in certain other borings at the site.

The upper 70 feet of bedrock is of primary concern for the proposed construction. The rock strata encountered in our field investigation are in descending order the Pottsville (?) sandstone of Pennsylvanian age, the Divine limestone member of the Maquoketa Formation which is of Ordovician age, and the Maquoketa shale which is also of Ordovician age.

Sandstone was encountered near the surface in Borings 1, 6, 11, and 34. This sandstone is predominantly composed of subangular fine to medium quartz grains which are poorly to moderately cemented by calcium carbonate. In Boring 1, six feet of sandstone was encountered and the sandstone extended to approximately elevation 523.

The Divine limestone was encountered near the surface in most of the remaining test borings. The upper one to three feet of limestone is generally weathered, fractured, sandy, slightly pitted due to ground water, and in most cases interlayered with one to two inch thick seams of sandy clay. The lower unweathered portion of this material is a coarse crystalline, partly fossiliferous, dense limestone. The maximum thickness of limestone was measured in Boring 1 where 46 feet was encountered. The limestone generally extends to approximately

elevation 508 to 524 with the exception of Boring 1 were it extends to approximately elevation 471. The limestone is underlain by the Maquoketa shale. The Maquoketa shale is, for the most part, a dense, gray calcareous shale. The upper one to three feet of shale is generally greenish gray which characterizes the transitional zone between the shale and the overlying limestone and generally contains a layer of clay (weathered shale) which ranges in thickness from 1 to 21 inches in the test borings. The Maquoketa shale extends to the depths penetrated by the test borings. A more detailed description of the subsurface conditions is presented on the Log of Borings in Appendix A.

In summary, most of the site is underlain at shallow depths by limestone which is approximately 6 to 24 feet in thickness with the exception of the southwest portion of the site, vicinity of Boring 1, where a six foot layer of sandstone is present near the ground surface. Occasional pockets of sandstone may be present throughout the site as indicated by Boring 34. The sandstone in Boring 1 is in turn underlain by approximately 46 feet of limestone. The difference in the surface bedrock type and the difference in thickness of the limestone between Boring 1 and the remainder of the borings can be explained by normal faulting which is also prevalent at other locations in the vicinity of the site. Based on the results of the test borings and several additional test pits, we have found that an inactive fault is present within this portion of the site and have been able to establish the approximate location of the fault. The existence of this fault is further substantiated by a careful study of areal photographs of the area. The approximate location of the fault is shown on Plate 1. The fault appears to be an inactive, small normal fault which trends in a northwest-southeast direction, dips down to the southwest and has undergone approximately 40 feet of vertical displacement. To assist in the visualization

of the relationship between the various rock strata to the fault, a subsurface section has been prepared and is presented on Plate 2, Subsurface Section A-A.

Previous geologic studies performed in this region indicate that this region has undergone a complex series of faults and that the faulting had occurred prior to the deposition of the Pottsville (?) sandstone during the Pennsylvanian age since there is generally no evidence of faulting within the Pottsville (?) sandstone. This geologic evidence indicates that these faults are inactive and have been inactive for millions of years. Therefore, it can be concluded that the fault which is present on the site is also inactive and has not undergone any movement for millions of years.

Surface drainage is relatively poor at the site since the bedrock surface is undulating and entraps the surface water. A perched water condition exists at the site due to the relatively impermeable limestone and shale which underlie the site. The perched water condition is present within the upper fractured and weathered limestone that contains seams of sandy clay and is generally encountered above a depth of four to five feet below the existing ground surface. The true ground water level was not encountered within the depths of the test borings.

The maximum depth of frost penetration in the vicinity of the site is on the order of four feet. Discussions with a representative of the water department in Morris, Illinois, indicated that all water lines in Morris are installed at depths of approximately 4.5 to 5 feet below the ground surface to protect them against freezing. The City Engineer of Joliet indicated that all water lines in Joliet are installed at a depth of five feet below the ground surface to protect them against freezing.

DISCUSSION AND RECOMMENDATIONS

GENERAL:

It is recommended that the proposed facilities be supported on either conventional spread foundations and/or mat foundations established within the bedrock. Recommendations and data pertaining to the design and installation of foundations to resist vertical and horizontal loads and the design of substructure walls are presented in subsequent sections, FOUNDATION DESIGN DATA and DESIGN OF SUBSTRUCTURE WALLS.

Earthwork operations at the site will consist of clearing, grubbing, stripping and filling. Approximately four feet of fill may be required to attain the plan grade. In addition it will be required to excavate rock to attain the plan grades. A guide specification for rock excavation is presented in Appendix B of this report. Detailed recommendations for the earthwork operations and rock excavation are presented in the following section.

EARTHWORK OPERATIONS AND ROCK EXCAVATIONS:

Clearing and Grubbing - Clearing and grubbing operations will be required to remove a row of trees located on approximately coordinate East 350 feet.

Stripping - It is recommended that all soil be stripped down to the bedrock surface from areas to be occupied by structures and pavements. These soils contain organic material, are generally soft to moderately firm and would not provide adequate support for structures or pavements. It is estimated that the depth of stripping may range from approximately 4 to 42 inches and will have an average thickness on the order of 12 inches. The thickness of soil at each boring location is tabulated on Plate 1. The stripped soils should not be

utilized as fill within areas which will be subjected to structural and traffic loads. These soils may be used as general site fill in areas which are being filled for general site grading purposes only.

Rock Excavation - It will be necessary to make rock excavations ranging in depth from approximately zero to 30 feet for the proposed Main Process Building, 50 to 60 feet for the proposed storage vaults, and 10 to 15 feet for the Sand Filter. We also understand that additional rock excavation will be performed adjacent to the vaults to facilitate future expansion of these vaults. We understand that the remainder of the structures will not require extensive rock excavation since they will be established near the existing grade and that only minor rock excavation will be required in connection with installation of foundations and utilities.

Since the bedrock at the site is dense and hard, it will be necessary to blast to facilitate the excavation of the rock. The sides of the excavations can be excavated vertically and appropriate blasting operations such as pre-splitting or smoothing operations should be performed to insure a clean vertical excavation.

The rock obtained from the excavations will not be suitable for use as fill within the proposed building and paved areas. The excavated rock will be suitable for use as fill in general site grading operations.

It is anticipated that seepage water will enter the excavations through the upper one to three feet of fractured limestone which is interlayered with thin seams of soil. We do not anticipate that any significant seepage water will be encountered within the lower bedrock strata since the limestone and shale is relatively impermeable.

It is recommended that shale exposed in the bottoms of excavations be protected against freezing, surface water and air exposure by pouring a mud mat immediately upon completion of the excavation. It is anticipated that the banks of the excavations may also be effected by water and air exposure if they are left open for a long period of time. Therefore, it would be necessary to scrape the banks and remove any of the shale which has softened prior to pouring concrete adjacent to the shale. The scraping would be necessary to insure a good bond between the shale and the concrete to prevent a build up of hydrostatic pressure on the substructure walls. As an alternate to scraping the banks, it would be possible to cover the banks with plastic, gunite, or an asphalt sealer to minimize the exposure of the shale to water and air.

Filling - We understand that the final grade may be established approximately four feet above the existing grade. It is recommended that all fill or backfill placed to attain the plan grades within the proposed building and pavement areas consist of clean granular fill materials. This includes backfill over foundations, in utility trenches, and in areas in which the rock has been over excavated to facilitate future expansion of the vaults. It is recommended that the clean granular materials be placed in layers approximately six to eight inches in loose thickness. Each layer should be uniformly compacted to a dry density of at least 95 percent of the maximum dry density as determined by the American Association of State Highway Officials Test Designation T 180-57.

Since there was not any on-site soils which would be suitable for use as fill materials, we contacted local contractors to investigate possible sources of fill materials and to obtain representative cost data The following prices were quoted to Dames & Moore for estimating purposes only and should not be considered as firm bid prices:

1 - Bohac's Sand Pit
John F. Kennedy Road
Braidwood, Illinois
Contact: Mr. George Bohac
Phone: 458-6443

MATERIAL	COST DELIVERED TO SITE
Clean Fine Sand	$1.60 Per Cubic Yard
No.7 Road Gravel*	$1.60 to $1.75 Per Ton

*As specified in the State of Illinois Standard Specifications for Road and Bridge Construction, ART. 118.5, Coarse Aggregate for Gravel or Crushed Stone Base and Surface Course.

2 - Leo Fatlan Trucking
Gardner, Illinois
Phone: 237-2212

MATERIAL	COST DELIVERED TO SITE
Pit Run Sand	$2.00 Per Cubic Yard

3 - Material Services
Division of General Dynamics
Lockport, Illinois
Phone: 838-0600

MATERIAL	COST DELIVERED TO SITE
Pit Run Sand	$1.80 Per Ton

It is recommended that all earthwork operations be performed under the technical supervision of a qualified soils engineer.

FOUNDATION DESIGN DATA:

It is recommended that the proposed structures be supported on conventional spread foundations and/or mat foundations established within the bedrock. The foundations can be established at any convenient depth below the final proposed grade since the rock will not be affected by frost action. The foundations may be proportioned utilizing the bearing pressures which are tabulated below or any value up to those listed. These bearing pressures refer to the total of all dead and live loads and are net values. Since these are net values, the weight of the backfill over the foundations and the weight of the concrete in the foundations may be ignored in proportioning the foundations. These bearing pressures include a factor of safety against failure of the rock in excess of three.

SUPPORTING BEDROCK	MAXIMUM RECOMMENDED BEARING PRESSURE LBS./SQ. FT.
Weathered and Fractured Limestone with soil seams	10,000
Sandstone (Vicinity Boring 1)	20,000
Sound Limestone and Shale	70,000*

*This bearing pressure should be reduced to approximately 10,000 pounds per square foot if foundations are established at or within five feet above the clay layer (weathered shale) encountered at the interface between the limestone and the shale.

It is our opinion that any movement of the proposed structures will be the result of elastic compression of the supporting bedrock. A review of current published literature and unpublished data indicates that the modulus of elasticity of limestone is on the order of 1×10^7 pounds per square inch, shale is on the order of 5×10^6 pounds per square inch, and concrete is on the order of 3×10^6 pounds per square inch. Analyses utilizing the above moduli of elasticity indicate that movement of the foundations would be of no significance to the structure.

It is recommended that all loose or disturbed rock be removed from the foundation excavations and that the foundations be poured neat. Proviced that the foundations are poured neat, the horizontal loads acting on the foundations will be resisted by passive resistance of the bedrock immediately adjacent to the foundation and frictional resistance on the base of the foundation. It is recommended that the allowable passive resistance of the rock adjacent to the foundation be considered as 10,000 pounds per square foot, provided that rock is continuous for a horizontal distance of approximately one foundation width from the foundation, is sound, and bears directly against the foundation. If the foundation is established within the upper zone of weathered and fractured bedrock, the passive resistance of the rock should be reduced to 2,000 pounds per square foot. It is recommended that a coefficient of friction of 0.15 be utilized between concrete and the bedrock. The total frictional resistance will be the product of the coefficient of friction and the total load on the foundation. The recommended lateral resistances should provide a factor of safety in excess of three against failure of the rock.

ANCHOR RODS:

Structural resistance to the uplift forces resulting from the guy wires on the stack may be provided by a system of steel anchor rods, having a plate attached to the end, grouted into holes drilled into the limestone bedrock.

Resistance to uplift will be provided through bond between the grout and the surrounding limestone. It is recommended that the uplift resistance furnished by the upper three feet of limestone be neglected since it contains seams of soil and is weathered. In areas where sandstone is present at the surface such as southwest of the fault shown on Plate 1, the anchor rods should be installed into the limestone and the resistance provided by the sandstone should be neglected. The diameter of the drilled holes into which the anchor rods will be placed should be at least three times the diameter of the anchor rod. The minimum penetration of the drilled holes into the limestone should be at least three feet plus 60 times the diameter of the anchor rods. The plate attached to the bottom of the rod should be slightly smaller than the drilled hole to allow the grout to surround the plate. It is recommended that the allowable bond between the limestone and the grout be considered as 40 pounds per square inch.

In placing the anchorage system, it is recommended that the drilled hole be thoroughly cleaned with compressed air subsequent to drilling the holes. The anchor rods with the steel plate at the end should then be installed and the hole should be filled with expanding grout.

DESIGN OF SUBSTRUCTURE WALLS:

It is our opinion that all concrete substructure walls should be poured neat against the sound bedrock. If the substructure walls are poured neat, the bedrock will not exert any significant active pressure on the walls. Water pressures against concrete substructure walls which are poured against the bedrock will be negligible.

We understand that in certain areas, such as adjacent to the storage vaults, the bedrock will be over excavated at the time of the present construction and that the over excavation will be backfilled with controlled compacted granular backfill. Since these soils will become saturated and submerged during the life of the plant, it is recommended that these walls be designed to resist

lateral pressures due to backfill soils and water. The compacted fill material and water may be assumed to act as an equivalent fluid having a unit weight of 80 pounds per cubic foot.

It would be possible to reduce the lateral pressures acting on the walls if a continuous concrete cut-off wall was installed within the sound bedrock around the periphery of the over excavated area. These cut-off walls should extend to a depth of approximately five feet below the existing ground surface through the upper zone of weathered and fractured bedrock and should encircle the over excavated areas. Extreme care would have to be initiated to insure that this proposed cut-off system was installed as proposed above and within sound rock. It would also be necessary to seal the surface of the area within the cut-off wall to prevent direct percolation of water into the over excavated area. If this procedure was followed, the lateral pressures acting on the wall would then consist of only soil pressures, and the soil could be assumed to act as an equivalent fluid having a unit weight of 40 pounds per cubic foot.

It is our opinion that the substructure walls will not have to be designed to resist the horizontal components of adjacent foundation loads if the foundations are established on sound rock.

FLOOR SLABS:

It is recommended that all floor slabs established near the existing ground surface be underlain by a minimum of six inches of compacted, clean granular materials. It will not be necessary to place granular materials below floor slabs established within the lower bedrock strata.

PAVEMENT DESIGN DATA:

We understand that the paved areas will be subjected to H-20 loading. It is recommended that a flexible pavement be utilized for the proposed paved area. The pavement section should consist of a minimum of six inches of either

Grade 7, 8, or 9, Coarse Aggregate for Gravel or Crushed Stone Base and Surface Courses as defined in Article 118.5 of the State of Illinois Standard Specifications for Road and Bridge Construction, a prime coat, 1.5 inches of bituminous concrete binder course, and 1.5 inches of bituminous concrete surface course. The bituminous concrete binder and surface courses should consist of Fine Dense-Graded Aggregate Type, Class I, Subclass I-II as defined in Section 46 of the State of Illinois Standard Specifications for Road and Bridge Construction. All soil should be stripped from the paved areas prior to placing the base course, and the rock surface should be relatively rough to prevent slipping of the pavement at the interface of the rock and base course. The base course should be compacted to a dry density of at least 100 percent of the maximum dry density as determined by the American Association of State Highway Officials Test Designation T 180-57. All road construction should be performed in accordance with the State of Illinois Specifications.

It is further recommended that the shoulders and ditches adjacent to the pavements be graded such as to prevent the accumulation of water adjacent to the paved areas. In order to facilitate drainage, it may be desirable to raise the grade of the proposed pavements to insure that the surface water will drain out of the base course into the ditches.

SEWAGE DISPOSAL SYSTEM:

It is our opinion that it will not be feasible to utilize a leach field in connection with a sewage treatment plant since the soils blanketing the site are relatively impermeable and the bedrock is also relatively impermeable.

PROPOSED RAILROAD:

The portion of the proposed railroad which lies north of the existing County Road was investigated by walking the approximate route. We were unable to dig test pits in this area since the surface soils were soft and would not permit access of a tractor-mounted backhoe.

Our surficial inspection, supplemented by shallow probes, indicated that the low area immediately north of the County Road may contain black organic silts to depths ranging up to at least three feet. The high area farther north along the proposed route is underlain at a relatively shallow depth by sand and sandstone. The sand and sandstone is overlain by up to 1.5 feet of topsoil. The thickness of the sandstone is unknown.

It is recommended that test pits be excavated along the proposed railroad subsequent to the finalization of the plant location, to define the soil conditions more accurately.

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The following Plates and Appendices are attached and complete this report:

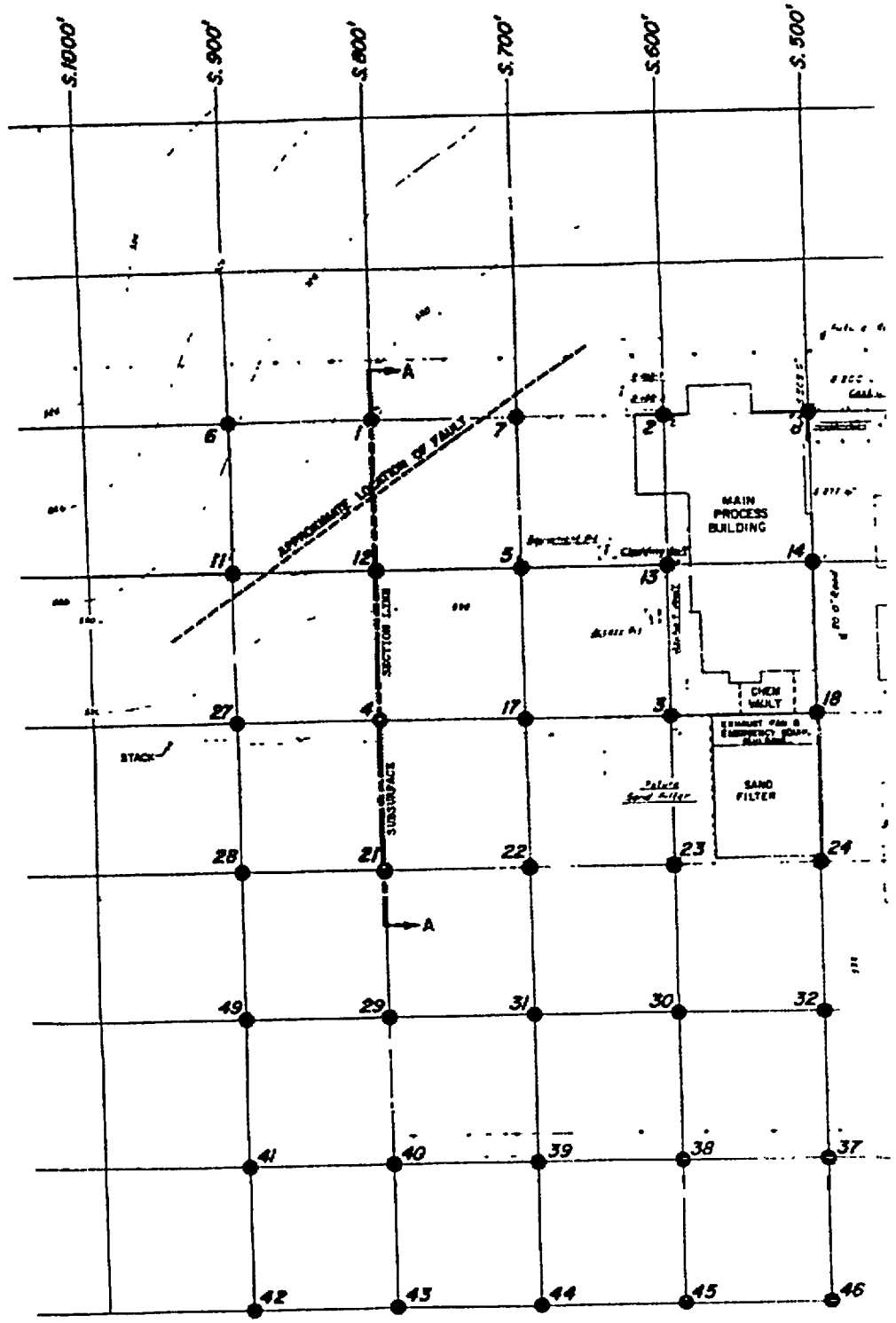
Plate 1	-	Plot Plan
Plate 2	-	Subsurface Section A-A
Appendix A	-	Exploration and Laboratory Tests
Appendix B	-	Guide Specifications For Rock Excavation

Respectfully submitted,

DAMES & MOORE

James B. Thompson

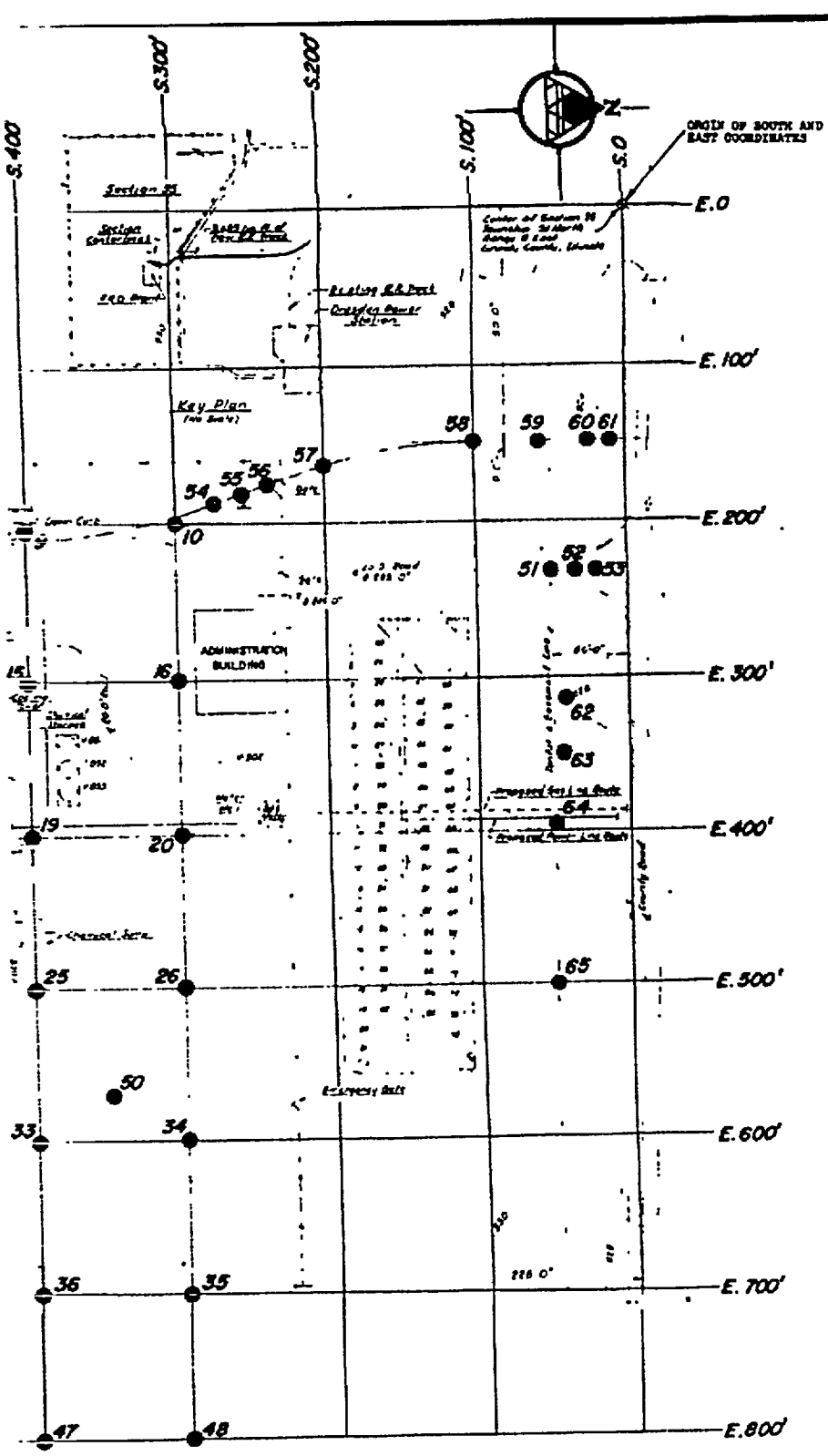
James B. Thompson



DRAWING REFERENCE:
THE FLUOR CORPORATION LTD.
JOB NUMBER 4204
DRAWING NUMBER SK5- 47 9
SOIL TEST BORING PLAN
PRO PLANT PROJECT
GENERAL ELETRIC CO.
MORRIS.ILLINOIS
DATED 11/27/67

REVISIONS
BY DATE
BY DATE
BY DATE
PLATE BY

FILE 1674-074
BY DATE 12-30-67
CHECKED BY DATE



BORING NUMBER	SURFACE ELEVATION FEET	THICKNESS OF SOIL FEET	ELEVATION TOP OF ROCK FEET	TYPE OF ROCK
1	530.6	1.3	529.3	Sandstone
2	532.7	2.0	530.7	Limestone
3	532.6	0.5	532.1	Limestone
4	533.7	0.3	533.4	Limestone
5	531.5	2.8	528.7	Limestone
6	526.9	0.7	526.2	Sandstone
7	531.6	1.5	530.1	Limestone
8	531.2	0.9	530.3	Limestone
9	531.2	0.7	530.5	Limestone
10	530.8	0.9	529.9	Limestone
11	530.1	2.0	528.1	Sandstone
12	531.6	1.2	530.4	Limestone
13	531.1	1.6	529.5	Limestone
14	531.3	1.0	530.3	Limestone
15	531.2	0.9	530.3	Limestone
16	530.8	1.0	529.8	Limestone
17	532.3	2.0	530.3	Limestone
18	532.1	1.6	530.5	Limestone
19	531.8	1.0	530.8	Limestone
20	531.4	1.0	530.4	Limestone
21	531.4	0.4	531.2	Limestone
22	532.5	0.5	532.0	Limestone
23	533.3	0.4	532.9	Limestone
24	533.2	0.5	532.7	Limestone
25	532.2	1.0	531.2	Limestone
26	531.4	1.7	529.7	Limestone
27	533.5	0.8	532.7	Limestone
28	533.9	0.3	533.6	Limestone
29	532.9	0.7	532.2	Limestone
30	533.1	0.7	532.4	Limestone
31	532.8	0.7	532.1	Limestone
32	533.4	0.3	533.1	Limestone
33	531.8	1.1	530.7	Limestone
34	531.7	2.5	529.2	Sandstone
35	531.1	1.3	529.8	Limestone
36	531.8	3.5	528.3	Limestone
37	532.4	0.8	531.6	Limestone
38	531.3	2.5	528.8	Limestone
39	534.1	0.3	533.8	Limestone
40	539.0	2.1	530.9	Limestone
41	532.9	0.3	532.6	Limestone
42	532.0*	0.9	531.2	Limestone
43	532.8*	0.6	532.4	Limestone
44	532.6*	1.8	530.8	Limestone
45	531.8*	0.8	531.0	Limestone
46	531.1*	0.5	530.6	Limestone
47	531.9*	0.9	530.0	Limestone
48	530.3*	2.6	527.6	Limestone
49	533.0	0.6	532.4	Limestone
50	532.0**	2.0	530.0	Limestone
51	528.5**	0.5	527.8	Limestone
52	528.0**	0.5	527.5	Limestone
53	527.7**	0.5	527.2	Limestone
54	530.5**	1.8	528.7	Limestone
55	530.5**	2.5	528.0	Limestone
56	530.5**	2.5	528.0	Limestone
57	530.2**	2.6	527.6	Limestone
58	528.7**	1.5	526.9	Limestone
59	528.0**	0.5	527.5	Limestone
60	528.0**	0.5	527.5	Limestone
61	527.0**	6.8	520.2	Shale
62	529.0**	0.1	528.9	Limestone
63	529.0**	2.0	527.0	Limestone
64	529.5**	1.3	528.2	Limestone
65	530.0**	1.0	529.0	Limestone

*These elevations were determined by Dames & Moore by leveling from adjacent borings where the elevations were established

**These elevations were determined by interpolating from the elevations provided to us by The Fluor Corporation, Ltd on Drawing No. SK 5-47 13, Grid Elevations, FRD Plant Project, General Electric Company, Morris, Illinois.

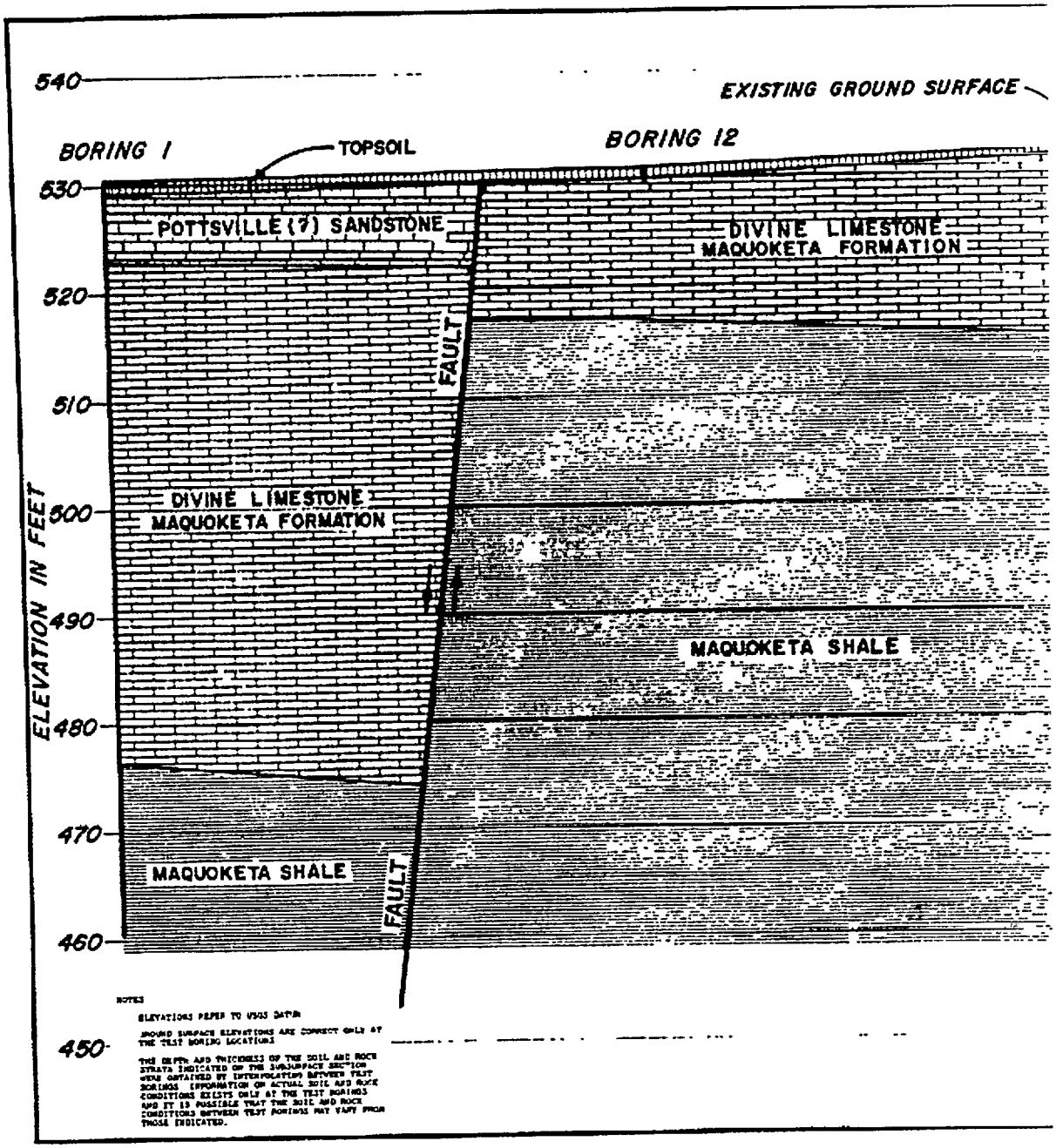
PLOT PLAN

FEET

50 0 50 100

DAMES & MOORE

PLATE 1

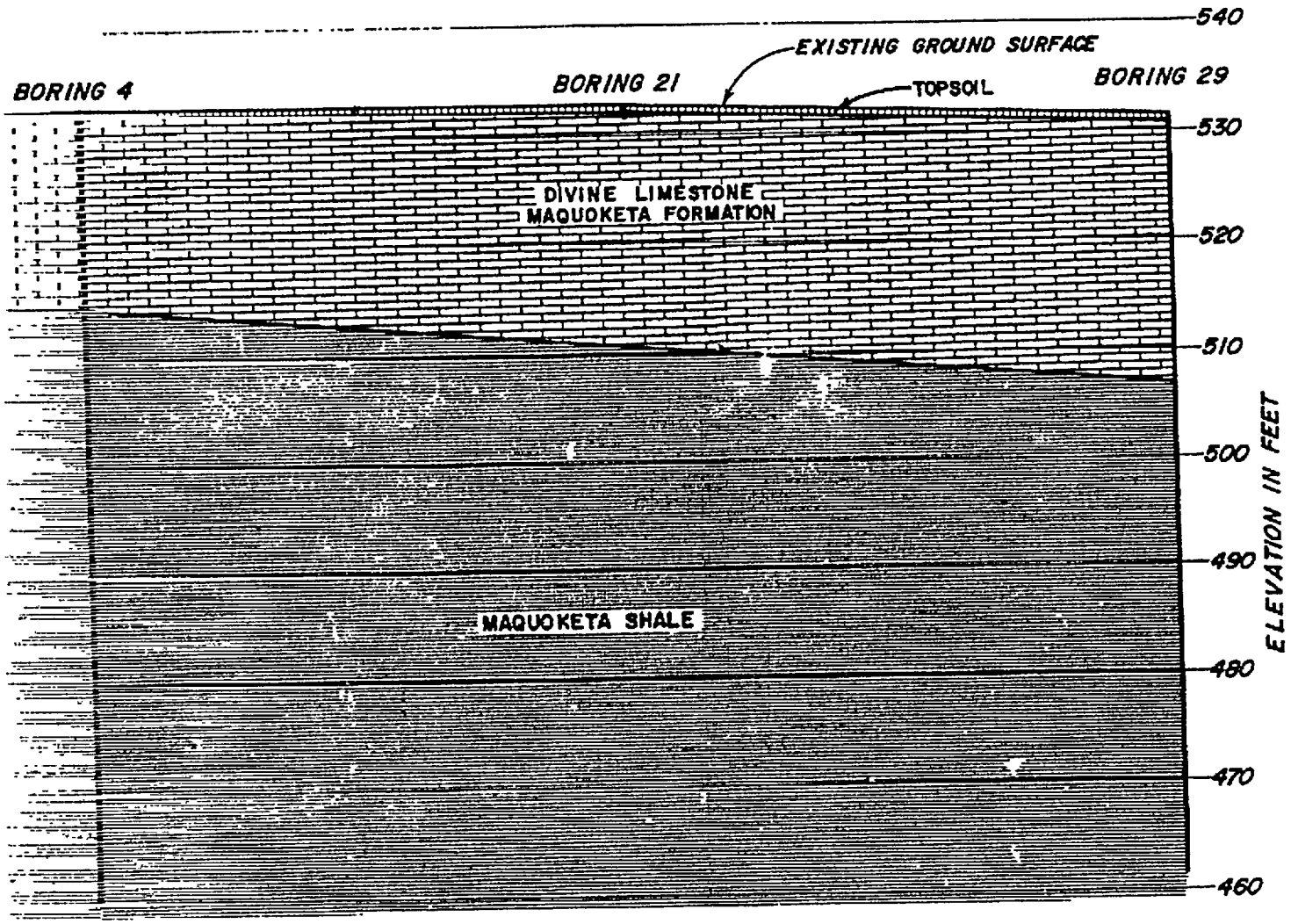


NOTES

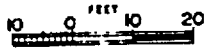
ELEVATIONS REFER TO USGS DATUM

GROUND SURFACE ELEVATIONS ARE CORRECT ONLY AT THE TEST BORING LOCATIONS

THE DEPTH AND THICKNESS OF THE SOIL AND ROCK STRATA INDICATED ON THE SUBSURFACE SECTION WERE OBTAINED BY INTERPOLATION BETWEEN TEST BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT THE TEST BORINGS AND IT IS POSSIBLE THAT THE SOIL AND ROCK CONDITIONS BETWEEN TEST BORINGS MAY VARY FROM THOSE INDICATED.



SUBSURFACE SECTION A-A



450

DAMES & MOORE

PLATE 2

APPENDIX A

EXPLORATIONS AND LABORATORY TESTS

EXPLORATIONS:

The subsurface conditions at the site were investigated by drilling ten exploration test borings with rotary-wash type drilling equipment to depths ranging from approximately 20 to 73 feet below the existing ground surface. An additional 55 shallow test borings were excavated with a tractor-mounted backhoe to depths ranging from approximately one to seven feet below the existing ground surface to define the upper surface of the bedrock at the site. Graphical representations of the soils and rock encountered in Borings 1 through 5, and 9, 14, 19, 29, and 30 are shown on Plates A-1A through A-1H, Log of Borings. The results of the remaining borings are summarized on Plate 1 presented in the text of the report. The method utilized in classifying the soils is defined by the Unified Soil Classification System presented on Plate A-2. Undisturbed samples of the subsurface soils were not obtained since the bedrock was encountered at a very shallow depth. Continuous cores of the underlying rock strata were obtained by diamond core drilling methods.

The locations of Borings 1 through 28 were staked in the field by a survey crew from Reiter & Associates of Joliet, Illinois. The locations of Borings 29 through 65 were staked in the field by the Dames & Moore Field Engineer. The elevations of Borings 1 through 41 and 49 were determined by Reiter & Associates and were provided to us on Drawing Number SK 5-47.13, Grid Elevations, FRD Plant Project, General Electric Co., Morris, Illinois. The elevations of Borings 42 through 48 were obtained by the Dames & Moore Field Engineer by leveling from the borings located on coordinate East 700.0 feet (Borings 35 through 41) to the locations of these borings. The elevations of Borings 49 through 65 were obtained by interpolating between the elevations

presented on the Drawing Number SK 5-47.13, Grid Elevations, FRO Plant Project, General Electric Co., Morris, Illinois. These elevations refer to U.S.G.S. Datum.

LABORATORY TESTS:

Since the soil cover was very thin and generally consisted of topsoil, there were no laboratory tests performed on the soils at the site.

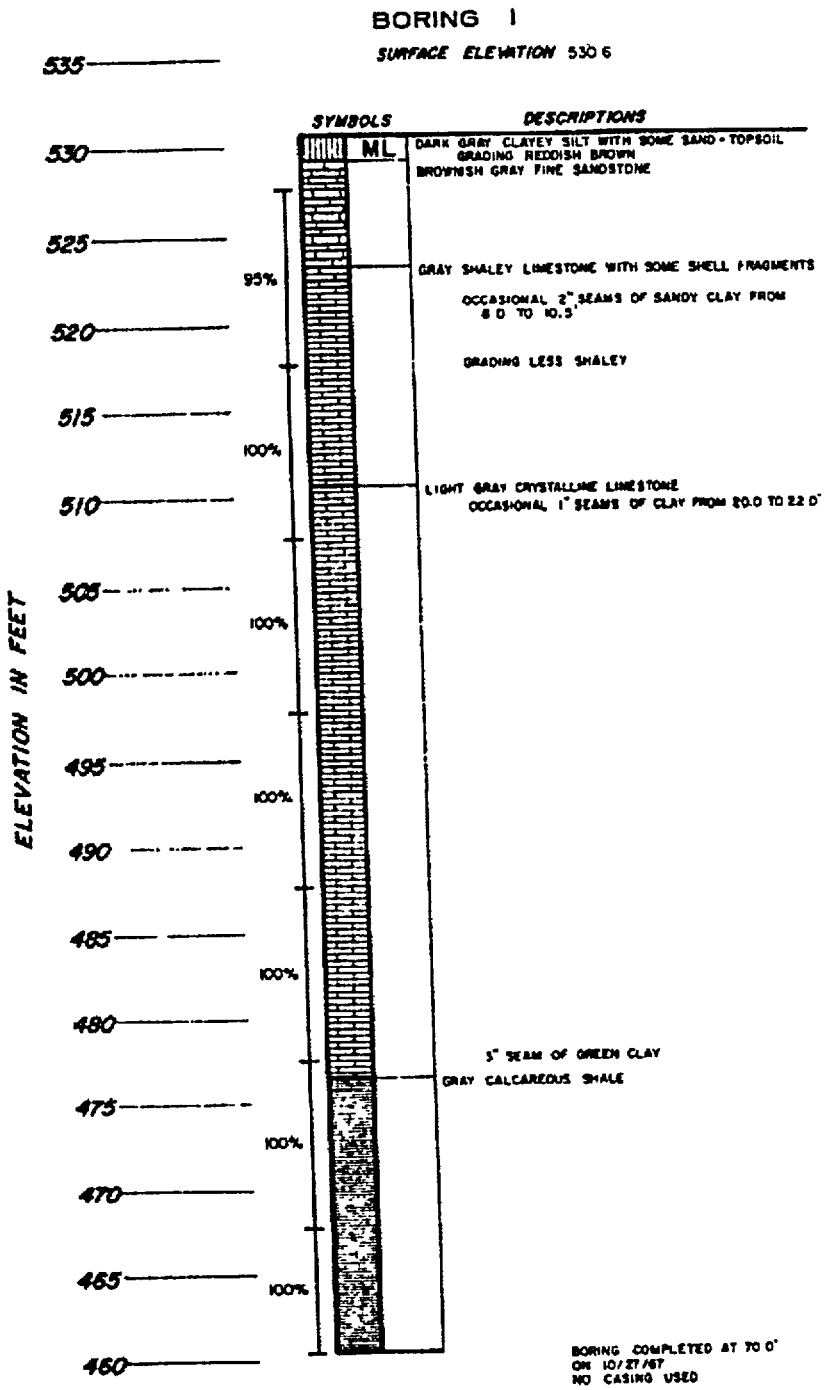
Unconfined compression tests were performed on selected rock cores to determine the strength characteristics of the various rock strata penetrated by the borings. The unconfined compression tests were performed by subjecting cylinders of rock approximately four inches in height by two inches in diameter to an incremental loading rate until failure of the sample occured. Specific gravity tests were also performed on selected samples to determine the density of the various rock strata. The results of the rock compression tests and the density tests are tabulated below.

BORING NUMBER	ELEVATION OF CORE, FEET	ROCK TYPE	ULTIMATE COMPRESSIVE STRENGTH, LBS./SQ. IN.	DENSITY LBS./CU. FT.
3	525.5	Limestone	20,127	-
3	512.5	Shale	8,854	-
4	511.5	Shale	10,127	-
4	501.5	Shale	6,497	-
4	481.0	Shale	8,471	-
4	471.5	Shale	8,662	-
5	525.5	Limestone	10,573	-
14	527.0	Limestone	11,548	-
14	507.0	Shale	9,286	-
14	596.0	Shale	6,242	-
14	467.5	Shale	7,738	-
19	523.5	Limestone	6,786	-
19	517.5	Shale	10,268	-
29	527.0	Limestone	12,675	168
29	509.0	Limestone	11,656	-
29	506.0	Shale	8,662	157
29	484.0	Shale	5,096	-
29	467.0	Shale	6,401	160
30	527.5	Limestone	14,204	164
30	516.0	Limestone	16,561	-
30	510.5	Limestone	14,013	171
30	507.0	Shale	12,038	161

--oOo--

The following Plates are attached and complete Appendix A:

Plate A-1A	-	Log of Borings (Boring 1)
Plate A-1B	-	Log of Borings (Boring 2 and 3)
Plate A-1C	-	Log of Borings (Boring 4)
Plate A-1D	-	Log of Borings (Borings 5 and 9)
Plate A-1E	-	Log of Borings (Boring 14)
Plate A-1F	-	Log of Borings (Boring 19)
Plate A-1G	-	Log of Borings (Boring 29)
Plate A-1H	-	Log of Borings (Boring 30)
Plate A-2	-	Unified Soil Classification System



LOG OF BORINGS

NOTES:

ELEVATIONS REFER TO USGS DATUM

100% INDICATES DEPTH, LENGTH AND PERCENT OF CORE RUN RECOVERED.

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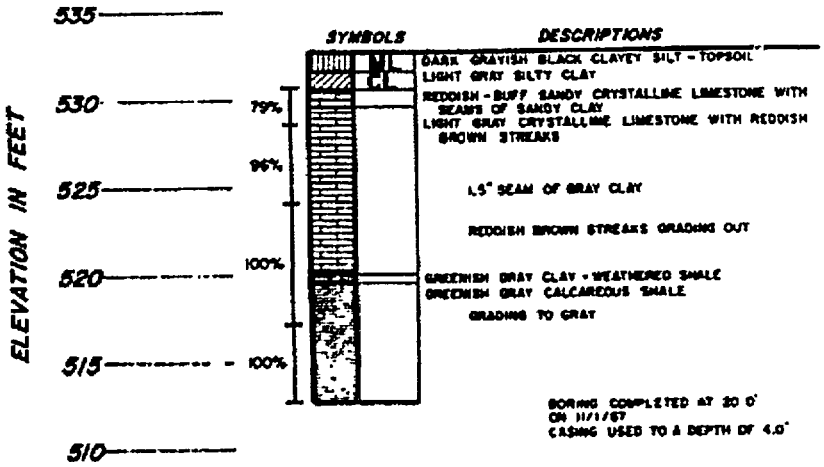
PLATE A-1A

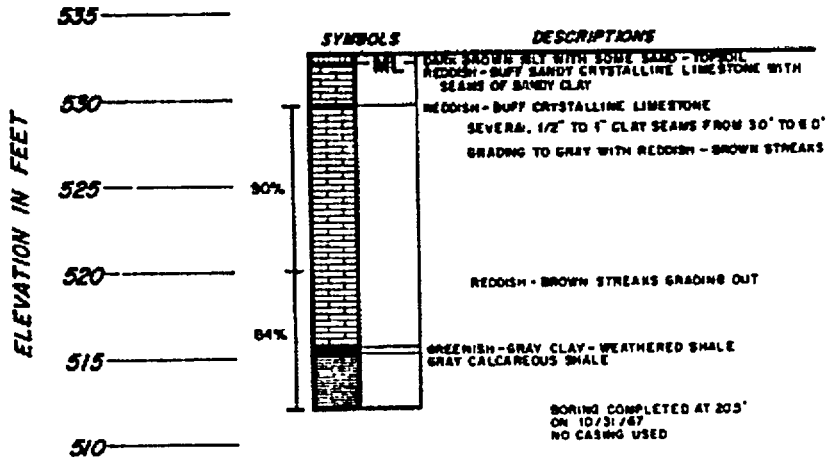
BORING 2

SURFACE ELEVATION 532.7



BORING 3

SURFACE ELEVATION 532.6



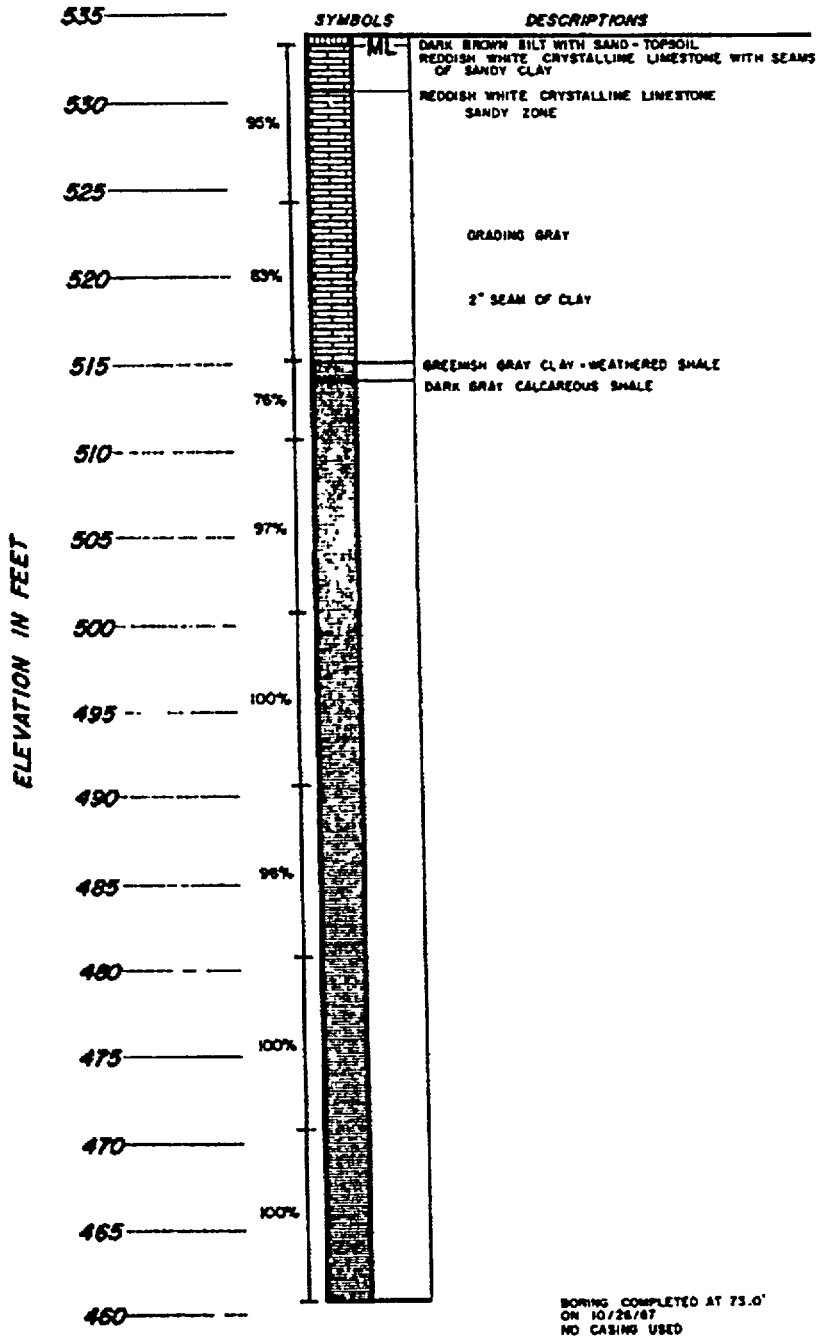
LOG OF BORINGS

DAMES & MOORE

PLATE A-1B

BORING 4

SURFACE ELEVATION 533.7



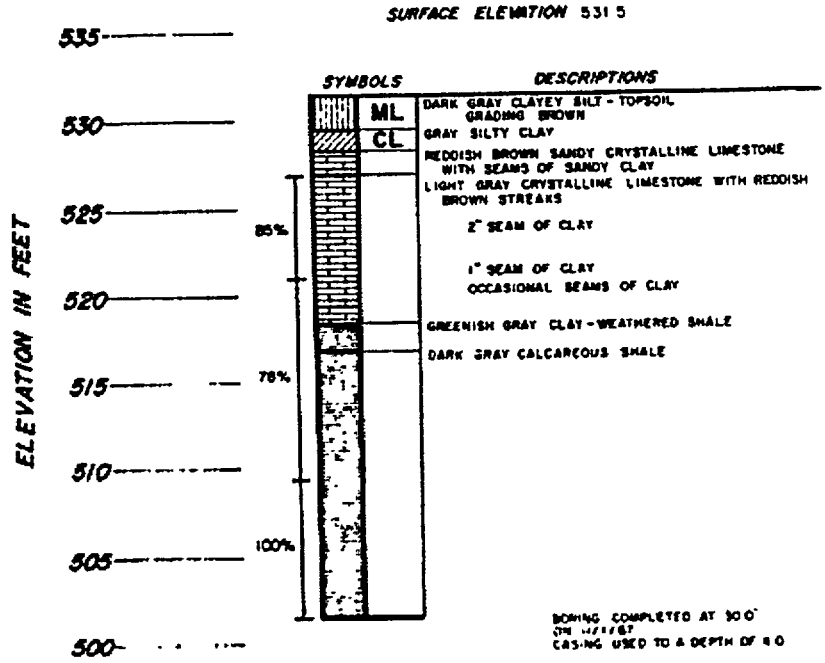
LOG OF BORINGS

DAMES & MOORE

PLATE A-1C

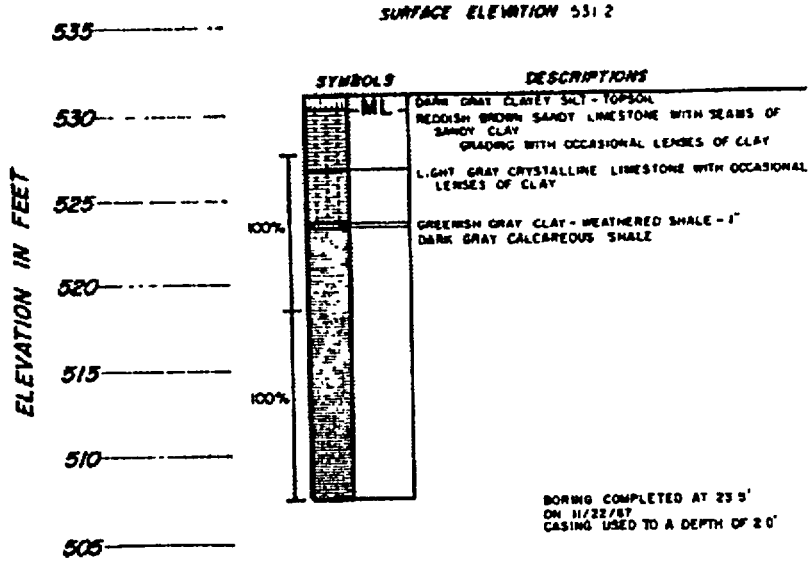
BORING 5

SURFACE ELEVATION 531.5



BORING 9

SURFACE ELEVATION 531.2



LOG OF BORINGS

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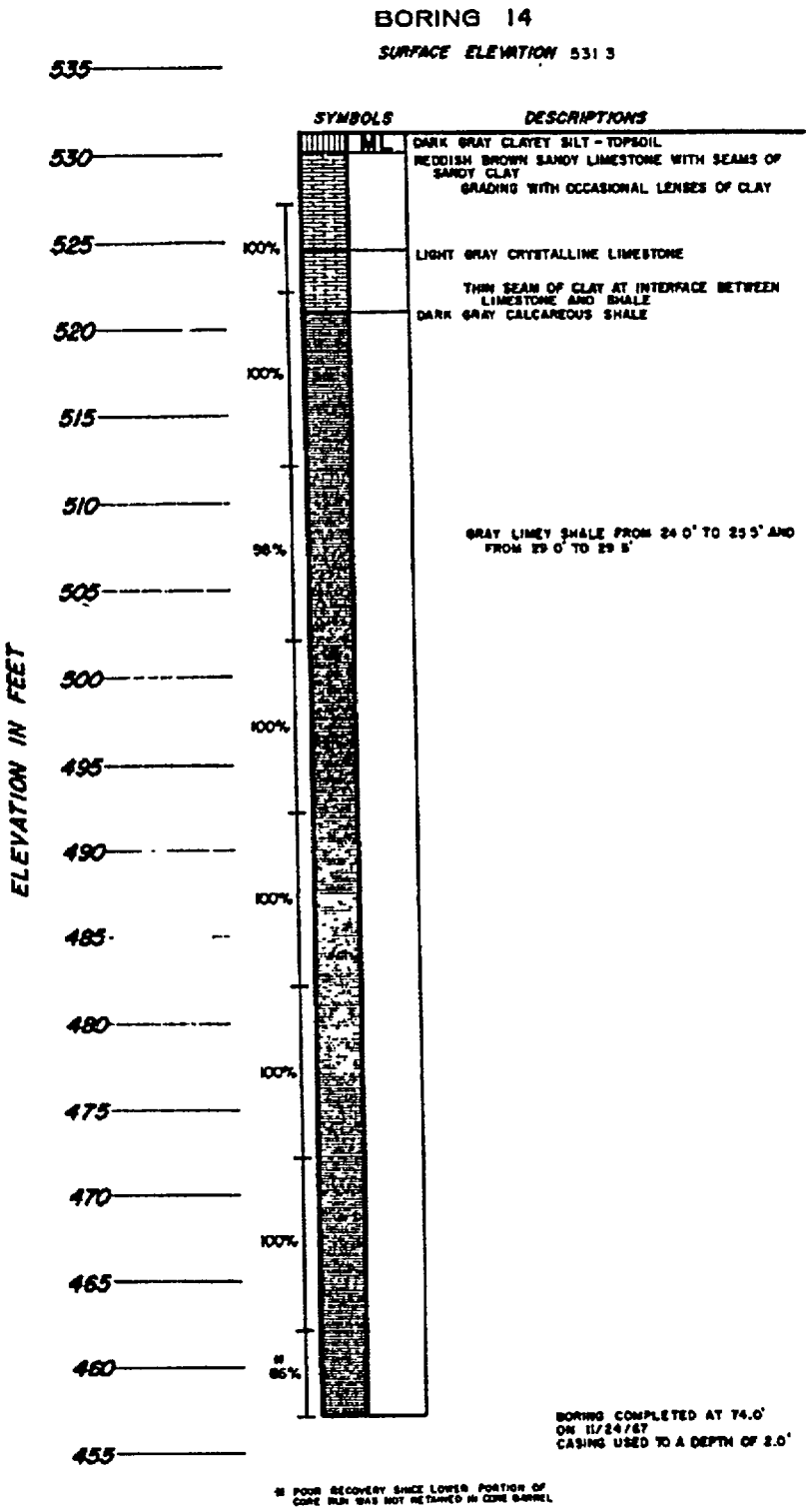
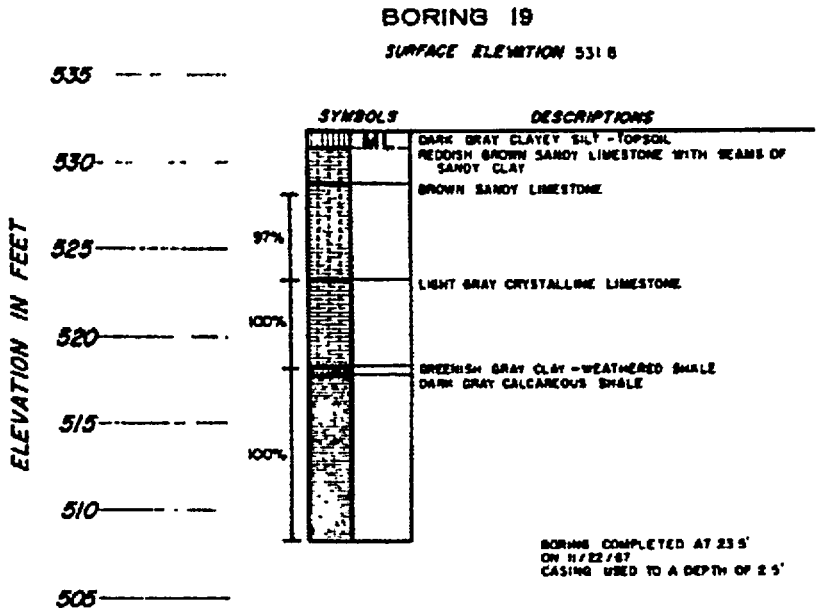
PLATE A-10



LOG OF BORINGS

DAMES & MOORE

PLATE A-1E



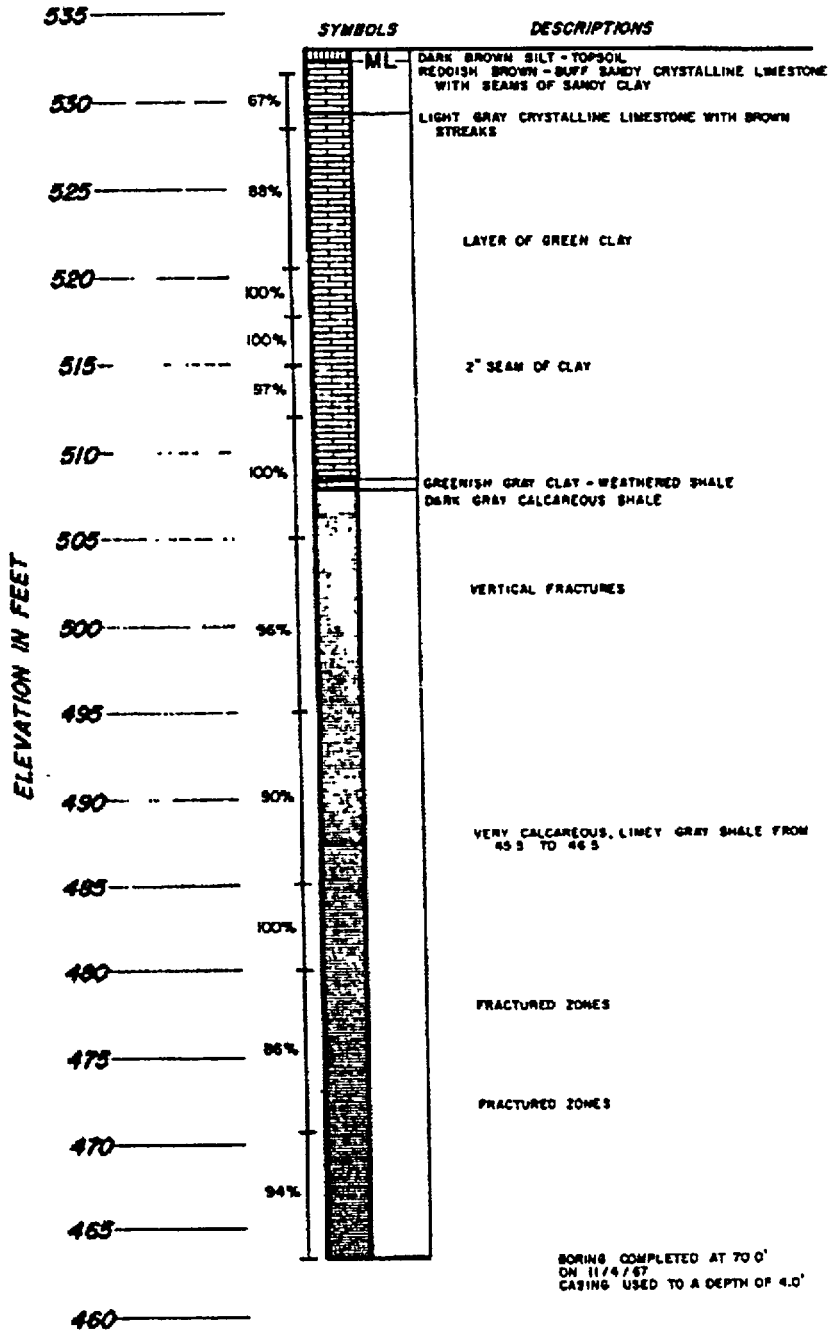
LOG OF BORINGS

DAMES & MOORE

PLATE A-1F

BORING 29

SURFACE ELEVATION 532.9



LOG OF BORINGS

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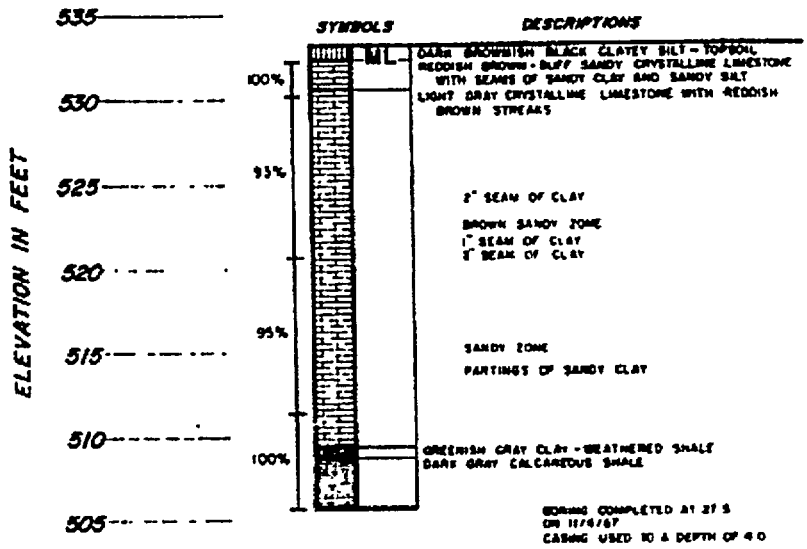
PLATE A-16

BORING 30

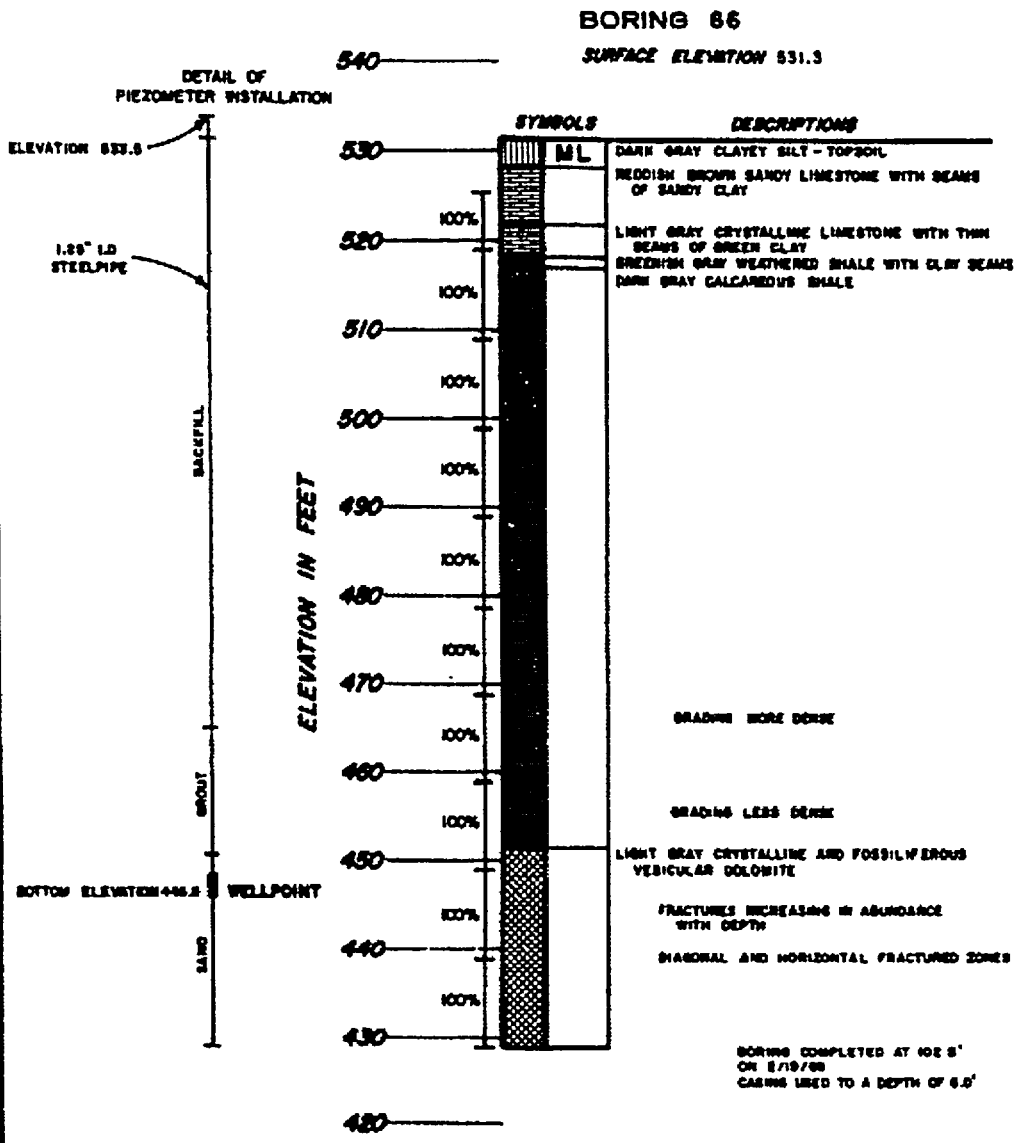
SURFACE ELEVATION 533.1



LOG OF BORINGS

DAMES & MOORE

PLATE A-1H



LOG OF BORINGS

NOTE:
ELEVATIONS REFER TO U.S.G.S. DATUM.

DAMES & MOORE

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILT AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

SOIL CLASSIFICATION CHART

UNIFIED SOIL CLASSIFICATI

DIVISIONS		GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
50% FRACTION SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
50% FRACTION	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

SOIL CLASSIFICATION CHART

DIRECT SHEAR AND FRICTION TESTS

TESTS AT FIELD MOISTURE — TESTS AT ARTIFICIALLY CHANGED MOISTURE

TEST NORMAL PRESSURE IN POUNDS PER SQUARE FOOT
PER CENT FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT
PER CENT MOISTURE WHEN TESTED EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL

SHEARING STRENGTH IN POUNDS PER SQUARE FOOT
FRICTION OF SOIL ON STEEL IN POUNDS PER SQUARE FOOT
FRICTION OF SOIL ON WOOD IN POUNDS PER SQUARE FOOT
FRICTION OF SOIL ON CONCRETE IN POUNDS PER SQUARE FOOT

UNCONFINED COMPRESSION TESTS

PER CENT FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT

SHEARING STRENGTH IN POUNDS PER SQUARE FOOT

TRIAXIAL COMPRESSION TESTS

CONFINING PRESSURE IN POUNDS PER SQUARE FOOT
PER CENT FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT

½ DEVIATOR STRESS IN POUNDS PER SQUARE FOOT

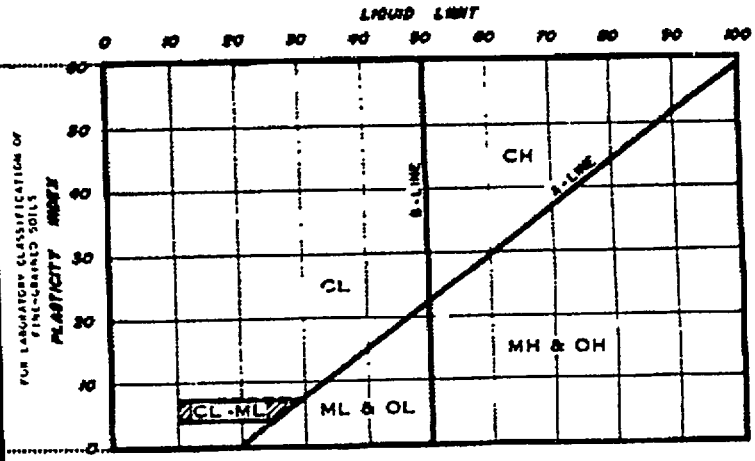
ROCK COMPRESSION TESTS

COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH

KEY TO TEST DATA

INDICATES DEPTH OF UNDISTURBED SAMPLE
INDICATES DEPTH OF DISTURBED SAMPLE
INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY
INDICATES DEPTH OF SPLIT-SPOON SAMPLE
INDICATES DEPTH AND LENGTH OF CORING RUN

KEY TO SAMPLES



PLASTICITY CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE
SOIL MECHANICS ENGINEERS

PLATE A-2

APPENDIX B

GUIDE SPECIFICATION FOR ROCK EXCAVATION

1 - CLASSIFICATION

Rock excavation includes all solid rock in place which cannot be removed until loosened by blasting, barring, or wedging and all boulders or detached pieces of solid rock more than one cubic yard in volume. Solid rock under this class, is defined as sound rock of such hardness and texture that it cannot be loosened or broken down by hand-drifting picks. Solid rock would include sandstone, shale, and limestone which are hard and dense. (All of the excavations in rock at the site would be classified as rock excavation.)

2 - EXCAVATION

When indicated or approved elevations have been reached, rock bottom surfaces shall be leveled off to a tolerance of plus zero, minus 4 inches, after all loose rock is removed. Where overbreak of rock results in dropping elevations of rock bottom surfaces below 4 inch tolerance, overbreak shall be filled, by Contractor, up to indicated or approved elevations with the same concrete as specified for the foundations. All labor, tool, equipment, and material costs, associated with such concrete fill, shall be for Contractor's account and shall not be reimbursed by Engineer-Designer.

Vertical rock faces, or faces steeper than 45 degrees from the horizontal, shall be held to the lines shown on the drawings with zero underbreak tolerance and 6 inches overbreak tolerance. Should an overbreak of any vertical face exceed the 6 inch tolerance, Contractor shall, from monies due him, reimburse the Engineer-Designer at the rate of _____ per cubic yard for the total number of cubic yards of excessive overbreak as determined by actual survey.

If, in the opinion of Engineer-Designer, the uppermost (1 to 3 feet) stratum of rock is considered excessively weathered, the above six inches overbreak tolerance will not apply.

Contractor shall carefully scale all vertical rock surfaces, or surfaces sufficiently steep to cause rolling or falling, to remove all loose material regardless of size. Survey to determine overbreak (or underbreak) will be performed only after the removal of all loose material.

All shale foundation surfaces shall be protected from freezing, water and air-slaking immediately upon completion of the excavation to the plan grades. The protection should consist of the placement of a concrete mud mat over the bottom of the excavation.

It will also be necessary to protect the side slopes established within the shale from exposure to water and air-slaking. If the Contractor selects not to protect the side slopes, the Contractor must scrape the shale banks prior to pouring concrete adjacent to the shale to remove any shale which has softened. The scraping is required to insure a good bond between the shale and the concrete. The method of protection or scraping should be proposed by the Contractor and approved by the Engineer-Designer prior to .

3 - Blasting

3.1 Blasting shall be performed only when proper precautions are taken for protection of persons, the work, private property, etc. Caps or other exploders or fuses shall in no case be stored or transported in same place in which dynamite or other explosives are transported. Location of storage magazines, methods of transportation and, in general, precautions taken to prevent accidents shall, at all times, be subject to approval of the Engineer-Designer, but the Contractor shall at all times be liable for any injuries to person or property caused by explosives.

3.2 Every possible precaution shall be taken in blasting operations to preserve rock outside lines of excavation in the soundest possible condition. As an excavation approaches its final lines, the depth of the holes for blasting, the amount of explosives used per hole, and the volume of rock blasted shall be reduced progressively. Blasting shall be done only to lines and grades indicated on drawings or as approved by the Engineer-Designer.

3.3 Explosives shall be of such quantity and power and shall be used in such locations as will not tend to open seams, or to crack or damage rock outside prescribed limits of excavation. If needed, firing of blast shall be controlled by use of delayed explosives. Whenever, in the opinion of the Engineer-Designer, continuation of blasting may injure rock on which or against which concrete is to be placed, use of explosives shall be discontinued and excavation shall be completed by wedging, boring, channeling or other suitable means.

3.4 Contractor shall submit plans and methods of operation for rock excavation work before work is started. Approval of method of blasting or of strength and amount of explosives used will not relieve Contractor of responsibility for blasting operations.

3.5 For blasting, Contractor shall employ a supervisor thoroughly experienced in this type of work and shall at all times maintain rigid inspection to see that intent of these requirements are fully complied with.

3.6 Contractor shall maintain a complete and detailed record of blasting operations, in a form approved by the Engineer-Designer and shall submit copies of such records to the Engineer-Designer.

3.7 All blasting shall conform to applicable requirements of the Health and Safety regulations of the Industrial Commission of the State of Illinois.

4. ROCK DISPOSAL

Contractor shall dispose of the rock obtained from the excavation in the respective areas indicated on Drawing No. _____.

DAMES & MOORE
CONSULTANTS IN APPLIED EARTH SCIENCES
SOIL MECHANICS ENGINEERING GEOLOGY GEOPHYSICS

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PARTNERS JAMES B THOMPSON • GEORGE D LEAL
ASSOCIATE WILLIAM G PARATORE

February 29, 1968

The Fluor Corporation, Ltd.
P. O. Box 7030 East Los Angeles Branch
Los Angeles, California 90022

Attention: Mr. D. C. Van Dine
Project Manager

Gentlemen:

Supplement - Report of Foundation Investigation
Proposed FRO Plant Project
Near Morris, Grundy County, Illinois
For The General Electric Company

This supplement presents the results of additional site investigations conducted subsequent to our Foundation Investigation performed at the site of the Proposed FRO Plant Project to be constructed near Morris in Grundy County, Illinois for the General Electric Company. The report of our Foundation Investigation was submitted to The Fluor Corporation, Ltd. on December 13, 1967. The scope of our additional investigations was planned in collaboration with Mr. R. V. Bierman of The Fluor Corporation, Ltd.

Scope - Subsequent to the completion of our foundation investigation, we received verbal information from The Fluor Corporation, Ltd. concerning a possible change in the depth and configuration of the cladding and asphalt vaults. Originally the vaults were to be rectangular and were to extend to a depth of approximately 50 feet. For this reason, the deepest boring previously drilled at the site penetrated to a depth of 70 feet. We understand that these vaults may now be approximately 50 feet in diameter and may extend to a depth of approximately 85 feet below final grade. Since we anticipated a change in the type of rock from shale to dolomite near the proposed base of the vaults, we recommended that an additional boring be drilled to a depth of about 100 feet near the location of our previous Boring 13. The purpose of the additional boring was to define the quality and type of rock near the proposed base of the vaults. It was proposed that a piezometer be installed in the boring to establish the water conditions near the proposed base of the vaults.

In addition, it was requested that we make a visual inspection of the portion of the route of the proposed railroad which lies north of the existing County Road. The purpose of this inspection was to determine if there were surface indications of any relatively deep soil deposits which would not adequately support the proposed railroad and which should be investigated by test pits. We understand that relatively shallow organic silts a few feet in depth are anticipated along the route and will not require subsurface investigation.

Subsurface Conditions - The soil and rock penetrated by the boring was essentially the same as revealed by the previous borings which extended to a depth of 70 feet. A thin layer of surface soil was underlain by the Divine Limestone which was underlain by the Maquoketa shale. At a depth of 80 feet, the Galena dolomite formation was encountered. The dolomite extended to the depth penetrated by the boring.

The lower four feet of the Maquoketa shale, between elevations 451.3 and 455.3 is not as sound as the shale above. It contains seams of weak shale interbedded with dense shale. The Galena dolomite is a dense, light gray crystalline and fossiliferous dolomite with some vesicles. The dolomite contains some moderately fractured zones below about elevation 444.

A piezometer was installed in Boring 66 as shown on the sketch to the left of the log of boring on Plate A-11. The bottom of the well point was placed within the dolomite stratum at a depth of about 84.5 feet below the existing ground surface. The water level in the piezometer tip was isolated from the water level above the shale stratum. Thus, water level measurements taken in the piezometer reflect the water condition in the dolomite above the piezometer tip elevation. Measurements recorded at various time intervals are as follows:

ELAPSED TIME AFTER SETTING PIEZOMETER (HOURS)	DEPTH OF WATER LEVEL BELOW GROUND SURFACE (FEET) (GROUND SURFACE IS AT ELEVATION 531.3)
Initial	11.0
1	63.5
47	82.6
110	Below piezometer tip depth of 84.5

The water in the piezometer is the water that was in the borehole during piezometer installation. The water level dropped in the piezometer to below the piezometer tip elevation as equilibrium was established. Based on these measurements, it is our opinion that the present static water level in the dolomite stratum is below about elevation 447 at the location of the proposed vaults. If there is any significant fluctuation of the water level in the dolomite stratum, the water movement will take place primarily along joints or fractured zones. Thus, it is conceiveable that the water level could periodically rise above the present level. For this reason, it is recommended that additional readings be taken periodically, prior to and during construction operations atthe site.

Recommendations - The proposed vaults may be established on conventional spread foundations or mat foundations established within the shale or the dolomite. We understand that the maximum bearing pressures for the vault foundations will be on the order of 10,000 pounds per square foot. If the vault foundations are established in the shale between elevations 451 and 460, it is recommended that the maximum bearing pressure not exceed 10,000 pounds per square foot. If the foundations are established within the dolomite stratum, the maximum bearing pressure can be increased, but should not exceed 70,000 pounds per square foot. These bearing pressures refer to the total of all dead and live loads.

Our previous recommendations pertaining to rock excavation and design of substructure walls presented in the subject report are applicable for the revised vaults. The excavations for the vault can be successfully accomplished by blasting while maintaining vertical sides on the excavation. It is anticipated that there will be no significant seepage water entering the excavations from the dolomite stratum.

We made a visual inspection of the proposed railroad alignment north of the County Road between stations 0 + 00 and 24 + 83. There is no surface evidence of any deep zones of soil which will not adequately support the proposed railroad. Previous surficial inspection, supplemented by shallow probes, has indicated the presence of organic silts to depths of at least three feet. It is possible that there could be deeper zones of unsuitable soil, but there is no surface evidence to indicate they would extend over a large area.

Explorations - The additional test boring, Boring 66, was drilled immediately adjacent to Boring 13, which is shown on Plate I, Plot Plan, of the subject report. The boring was drilled with rotary wash type drilling equipment to a depth of 102.5 feet. Continuous cores of the rock were obtained by diamond core drilling methods. A graphical representation of the soils and rock encountered in Boring 66 is shown on Plate II, Log of Borings.

A piezometer was installed in the boring as shown on Plate A-11. Upon completion of the test boring, the hole was flushed with water in order to remove cuttings from the shale which may have partly sealed the sides of the hole in the dolomite. The hole was then bailed to a depth of about 30 feet where the capacity of the bailing equipment was reached. A one and one-quarter inch inside diameter steel pipe with a 30-inch long well point attached was placed into the hole and the hole was backfilled to elevation 450.5 with clean sand. A 100 pound bag of cement grout mix was placed in the hole to form a seal around the piezometer pipe in the shale and to prevent surface water from entering the dolomite stratum. The hole was backfilled to the ground surface with soil.

--oOo--

It is requested that this supplement be bound with and become a part of the subject report.

Yours very truly,

DAMES & MOORE

James B. Thompson

JBT:PSF:ac
Ten Copies Submitted
Attachment: Plate A-11 - Log of Borings

END

NEDO-21326

APPENDIX B-3

EARTHQUAKE AND WIND ANALYSIS

PLANT EFFLUENT STACK

NOVEMBER 1968

Morris Operation
General Electric Company
Morris, Illinois

REPORT ON THE EARTHQUAKE AND WIND ANALYSES OF THE PLANT EFFLUENT STACK FOR THE MIDWEST FUEL RECOVERY PLANT

This report presents the results of a dynamic earthquake analysis and wind analysis of the plant effluent stack. In the earthquake analysis, a mathematical model of the stack is subjected to the design earthquake and the time-history of the stack's response is determined with the aid of an electronic computer system. The same mathematical model is subjected to a set of static forces representing the design wind and the stack's response is determined. The influence of wind induced vibrations is discussed.

ANALYTICAL CRITERIA

The analysis was based on the following data taken from Reference 1:

1. Design Earthquake

   North-South component of the 1940 El Centro earthquake normalized to 0.10g.

2. Design Wind

   110 miles per hour.

3. Damping

   Plant effluent stack – 0.01

STACK DESCRIPTION

The plant effluent stack is a guyed, steel stack 299 feet in height. The stack is supported at the base by a concrete footing and is laterally restrained by two layers of cable guys as shown in Figures 1 and 2. The cross section of the stack consists of 3/8-inch thick steel plate and 16 gage stainless steel liner as shown in Figure 3. Only the steel plate is included in the section property calculations. Data for the stack was taken from Reference 2.

ANALYTICAL METHOD

The steel stack was idealized as a model consisting of 29 lumped masses supported by weightless elastic columns and lateral springs representing the cable guys. The elastic properties of the columns were determined by computing the moment of inertia and effective shear area of the stack elements between mass points.

The cable guys laterally supporting the stack are nonlinear springs, their actual restraining effect being a function of the cable sag, geometry, and also the direction of the applied forces if the initial prestress force is exceeded in the "compression" cables. Since the analyses used for this report assume a linear, elastic behavior of the structure, the effect of the cable guys nonlinear behavior is approximated by determining the stack's response for a range of linear cable spring values.

The mathematical model used for the analyses is shown in Figure 4. Cross sectional properties, weights, and cable stiffness details are presented in the calculations.

1. Earthquake Analysis

The analysis used to obtain the earthquake response of the mathematical model is based on standard equations of motion for damped linear systems. Specifically, the matrix equations used to find the periods and mode shapes of the system and the time history response can be found in Reference 3 under the heading of "Exact Analysis".

A total of eight modes was considered in the analysis with a damping value of one percent of critical assumed for each mode. A complete time history of response was determined using an IBM 360/65 computer system, and the maximum results plotted.

2. Wind Analysis

The wind analysis was performed on a static basis using the complete range of cable spring values, and the loading pattern shown in the calculations.

DISCUSSION OF RESULTS

1. Lateral Vibrations

The lateral periods of vibration are summarized in Figure 9 for the most flexible and most stiff cable support conditions. The ovaling frequency of vibration was found to be about 22 cycles per second and is shown in the calculation.

2. Shears, Moments and Displacements

The envelope of maximum shears, moments and displacements are shown in Figures 5, 6 and 7 for the design seismic condition; and Figures 10, 11 and 12 for the 110 MPH wind condition. The seismic results should be doubled when reviewing for the 0.20g ground acceleration earthquake.

3. Cable Guy Forces

The maximum forces in the cable guys are summarized in Figure 9 for the design seismic condition and Figure 13 for the 110 MPH wind condition. The cable forces shown in Figure 9 should be doubled when reviewing for the 0.20g ground acceleration earthquake.

4. Horizontal Accelerations

The envelope of maximum, seismically induced absolute accelerations are shown in Figure 8 for the design seismic condition and should be doubled when reviewing for the 0.20g ground acceleration earthquake.

5. Wind Induced Vibrations

Lateral and ovaling vibrations produced by the shedding of the Karman vortex was investigated and the summary of this investigation is presented in the calculations. Based on the results shown, it is concluded that wind induced vibrations will not be a significant factor in the structural design of the Plant Effluent Stack.

MIDWEST FUEL RECOVERY PLANT

PLANT EFFLUENT STACK

REFERENCES

1. General Electric Company

   Design and Analysis Report for the Midwest Fuel Recovery Plant, including Amendments 1 and 3.

2. The Fluor Corporation Ltd.

   Plant Effluent Stack Drawings 5-2196 through 5-2199, all revision 1.

3. Clough, R. W., Earthquake Analysis by Response Superposition, Bulletin of the Seismological Society of America, Volume 52, No. 3, July, 1962.

MIDWEST FUEL RECOVERY PLANT

PLANT EFFLUENT STACK

LIST OF FIGURES

Description	Figure No.
East Elevation	1
Plan of Cable Guys	2
Stack Sections	3
Mathematical Model	4
Seismic Shears	5
Seismic Moments	6
Seismic Displacements	7
Seismic Accelerations	8
Periods and Seismic Guy Forces	9
Wind Shears	10
Wind Moments	11
Wind Displacements	12
Wind Guy Forces	13

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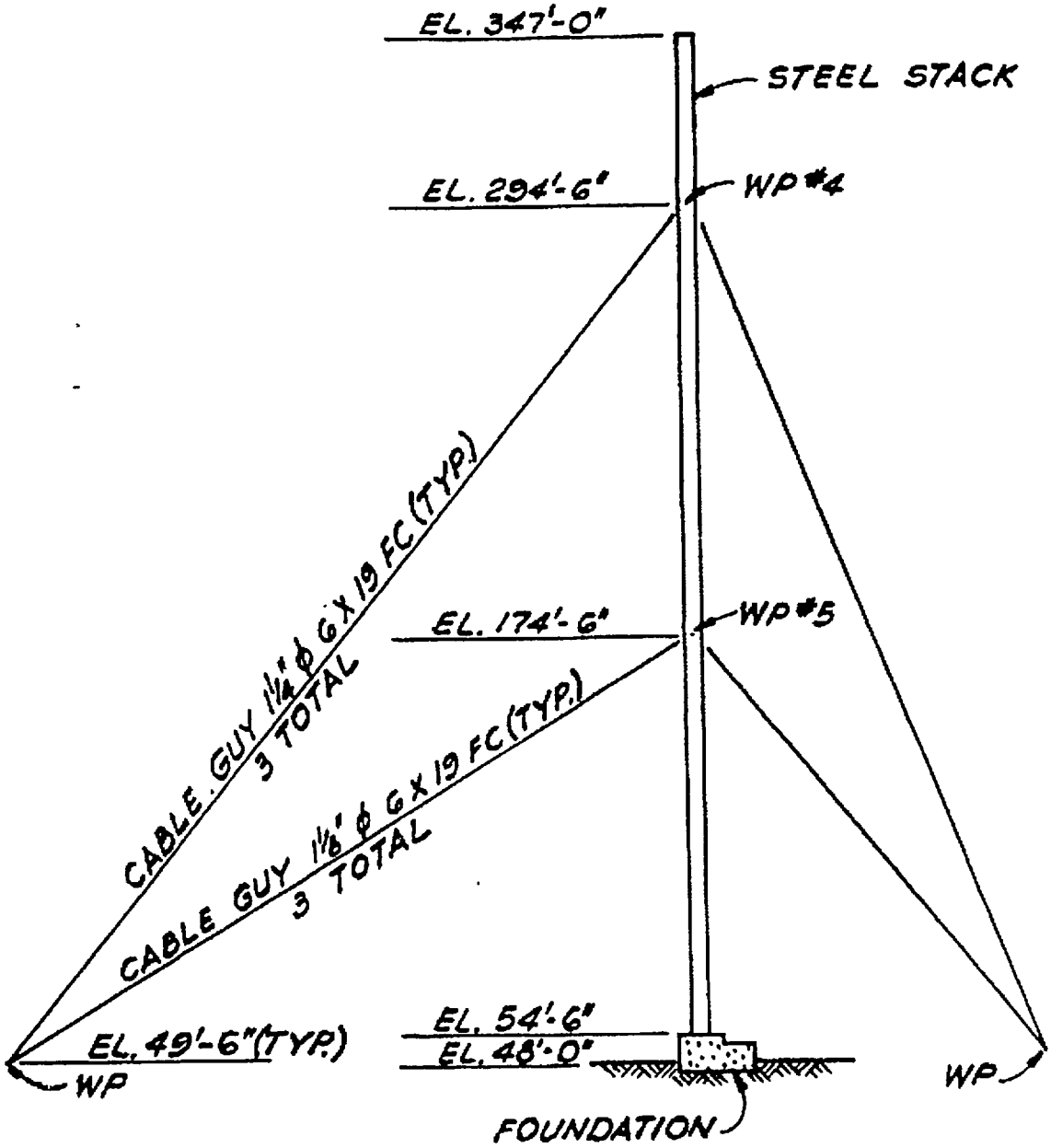
MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK



EAST ELEVATION

FIGURE 1

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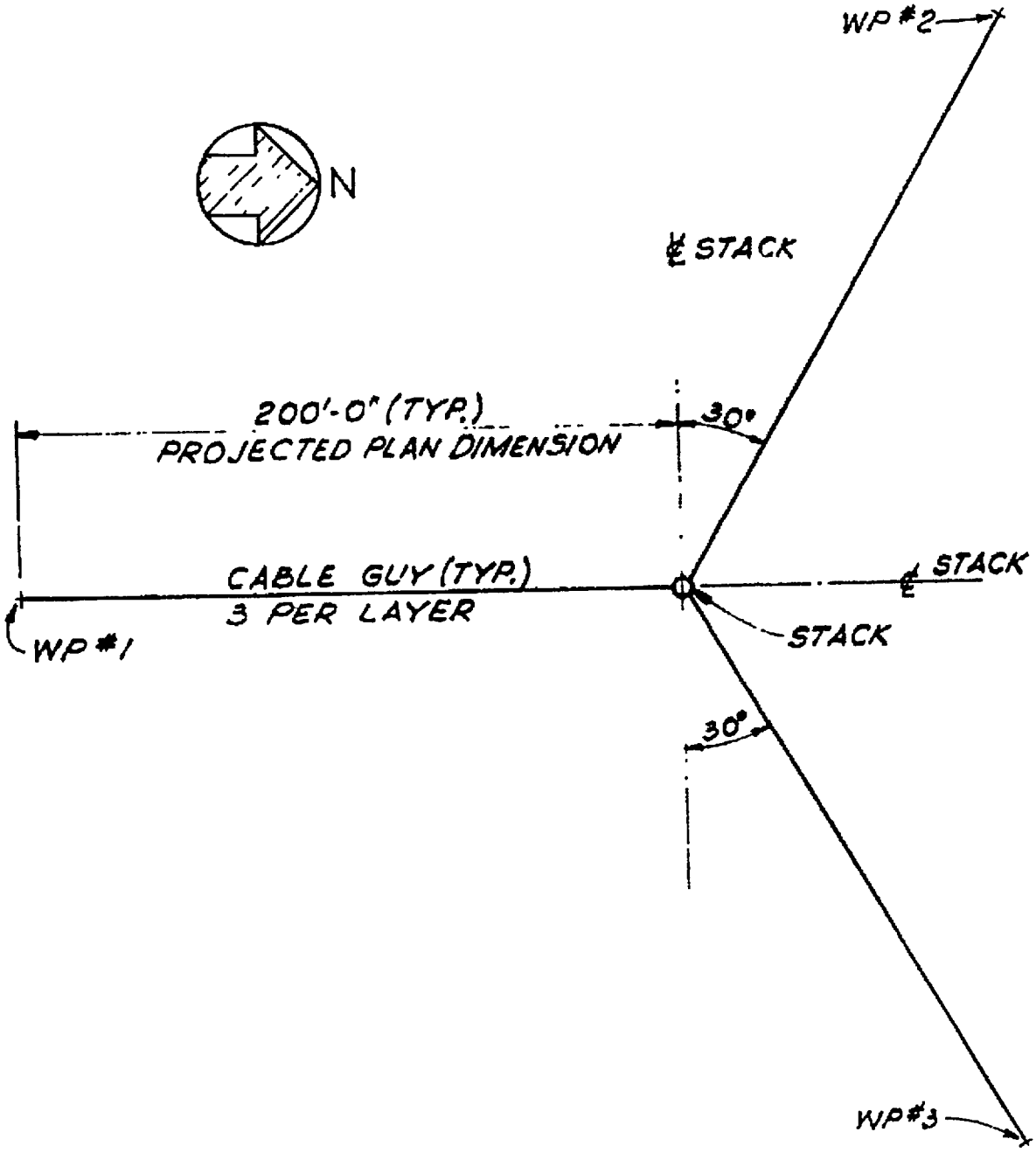
PLANT EFFLUENT STACK



PLAN OF CABLE GUYS

FIGURE 2

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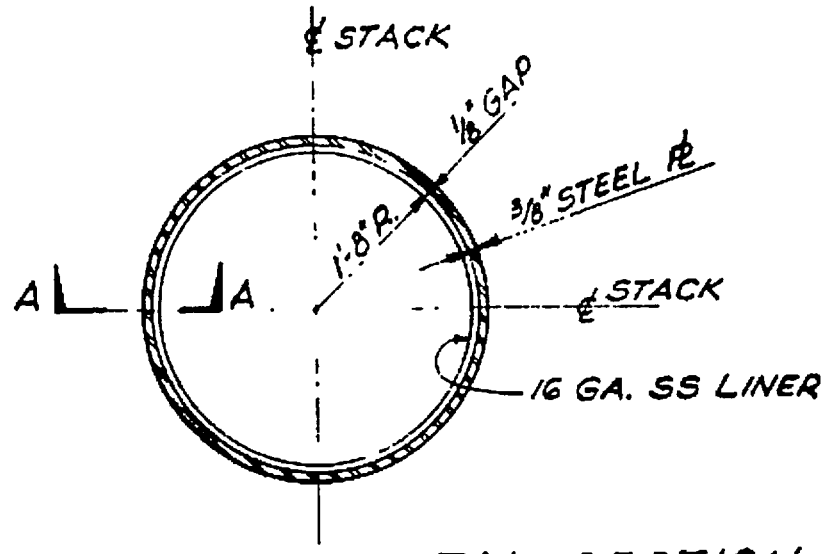
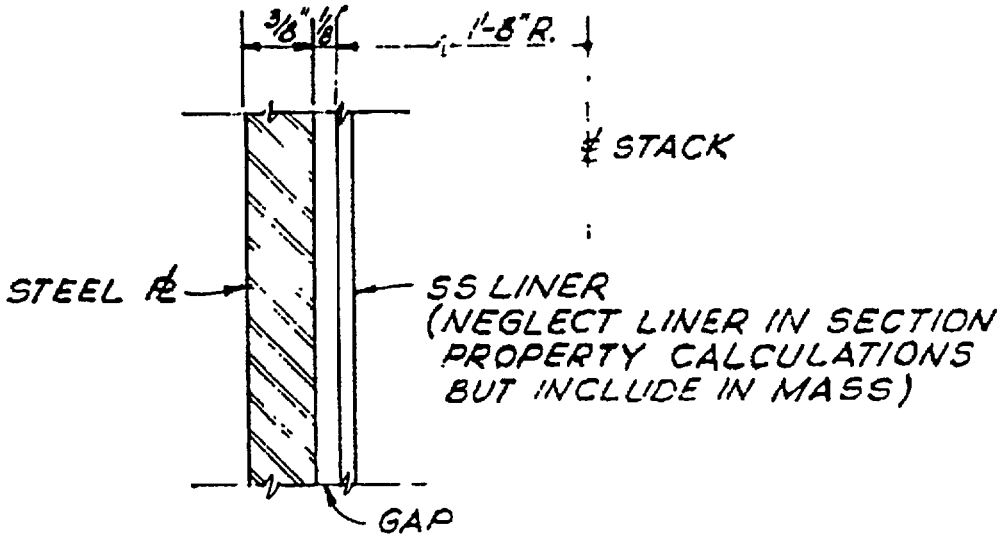
PLANT EFFLUENT STACK



TYPICAL HORIZONTAL SECTION THROUGH STACK



SECTION A-A

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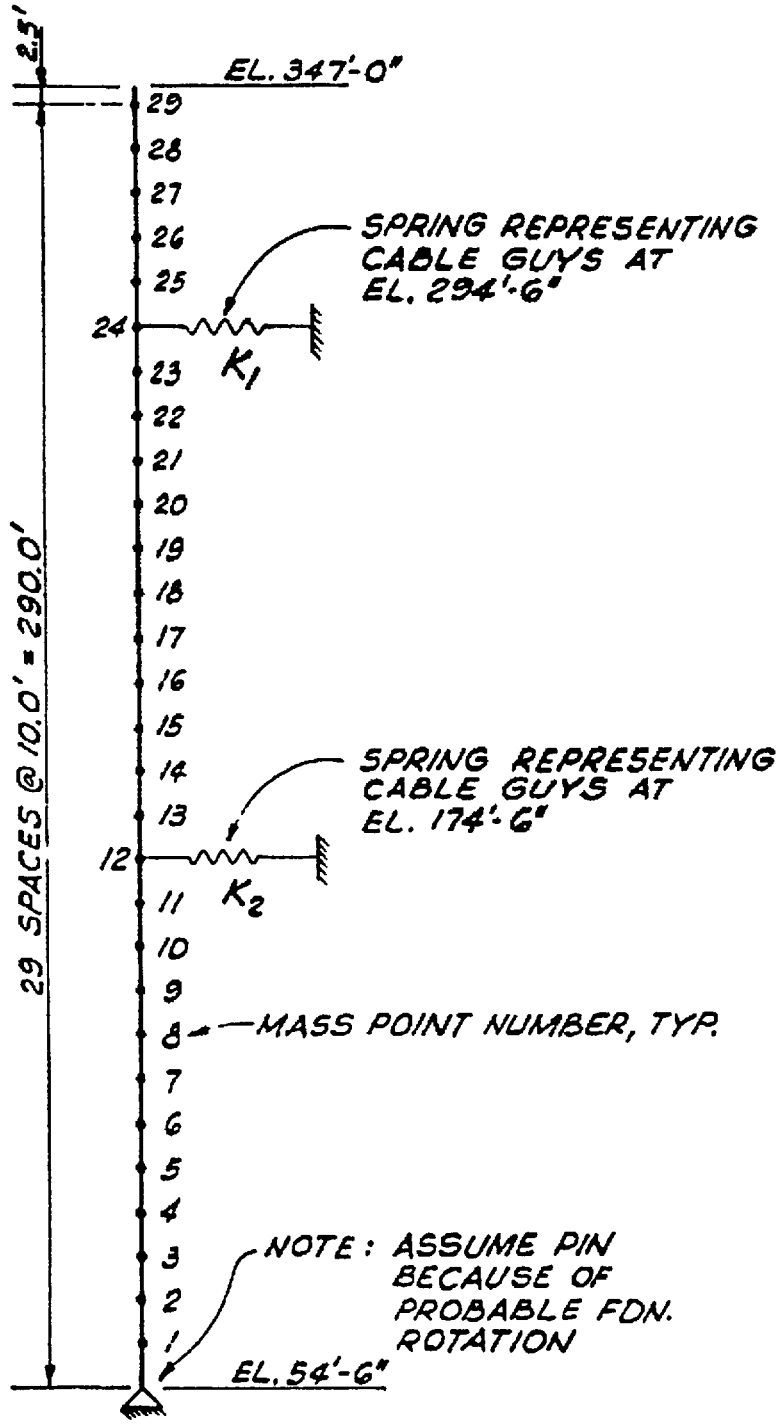
FIGURE 3

MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK



MATHEMATICAL MODEL

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FIGURE 4

SEISMIC RESULTS

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MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK
SEISMIC SHEARS

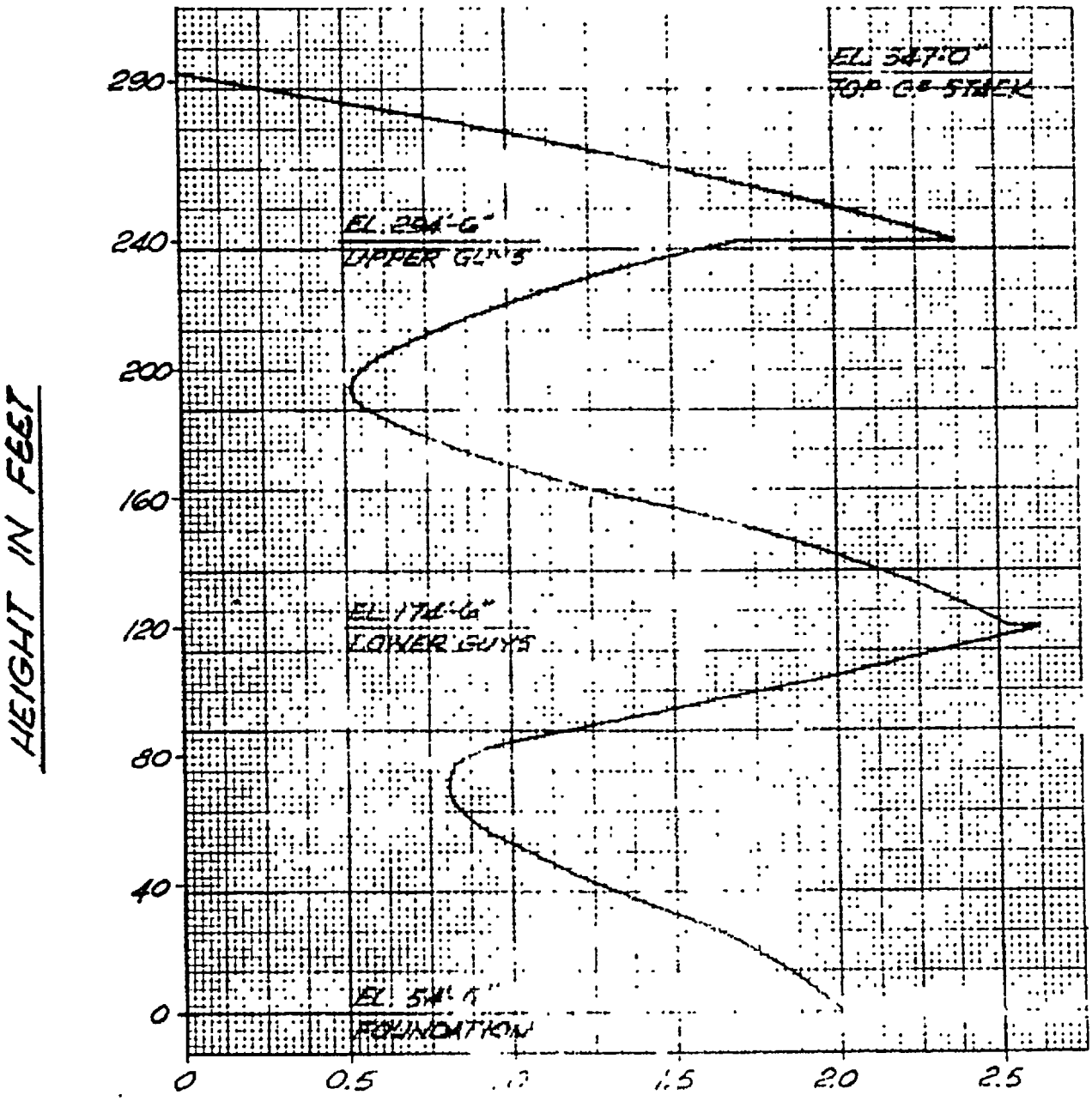

FIGURE 5

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MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK
SEISMIC MOMENTS

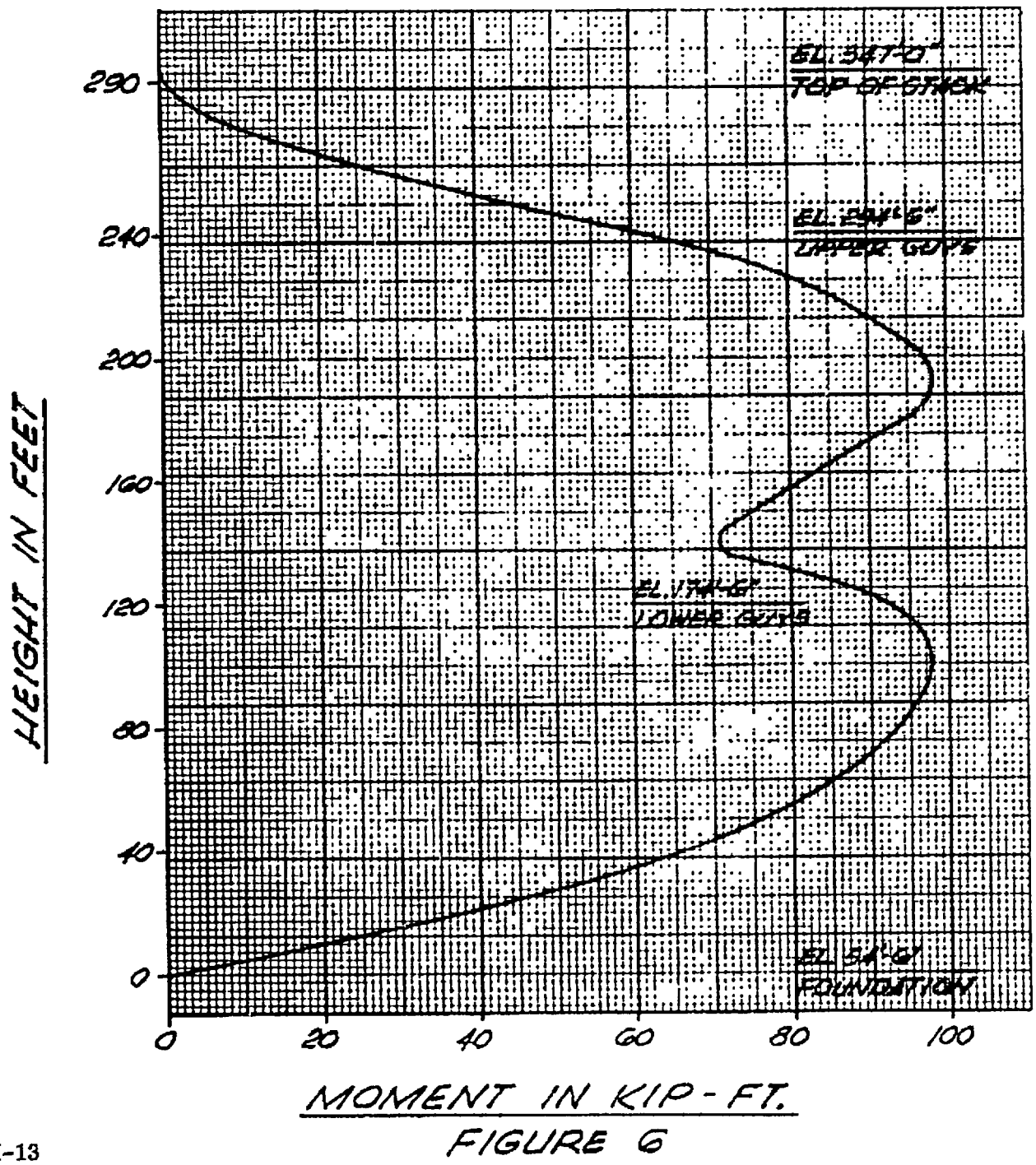

FIGURE 6

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MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK
SEISMIC DISPLACEMENTS

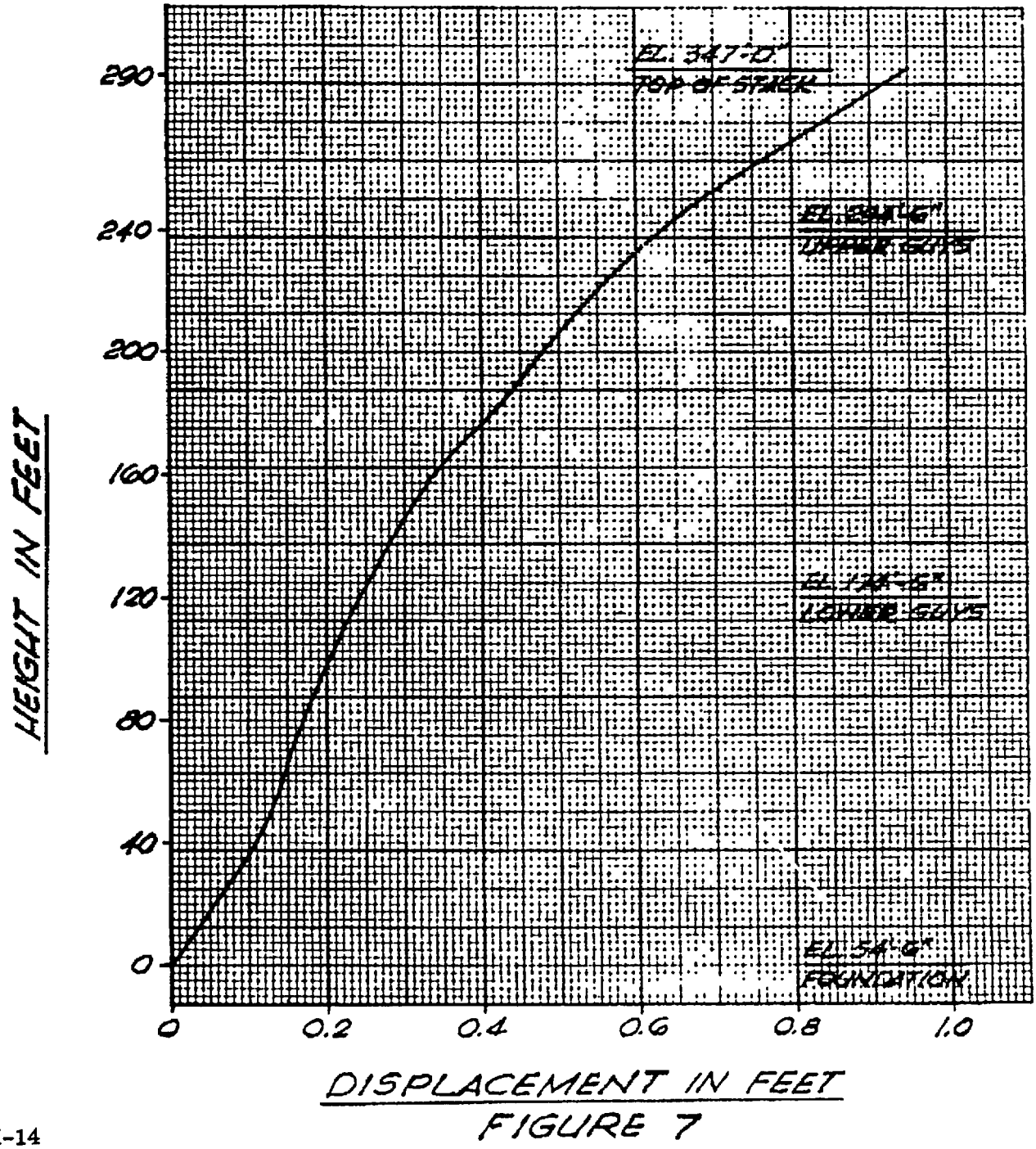

FIGURE 7

MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK
SEISMIC ACCELERATIONS

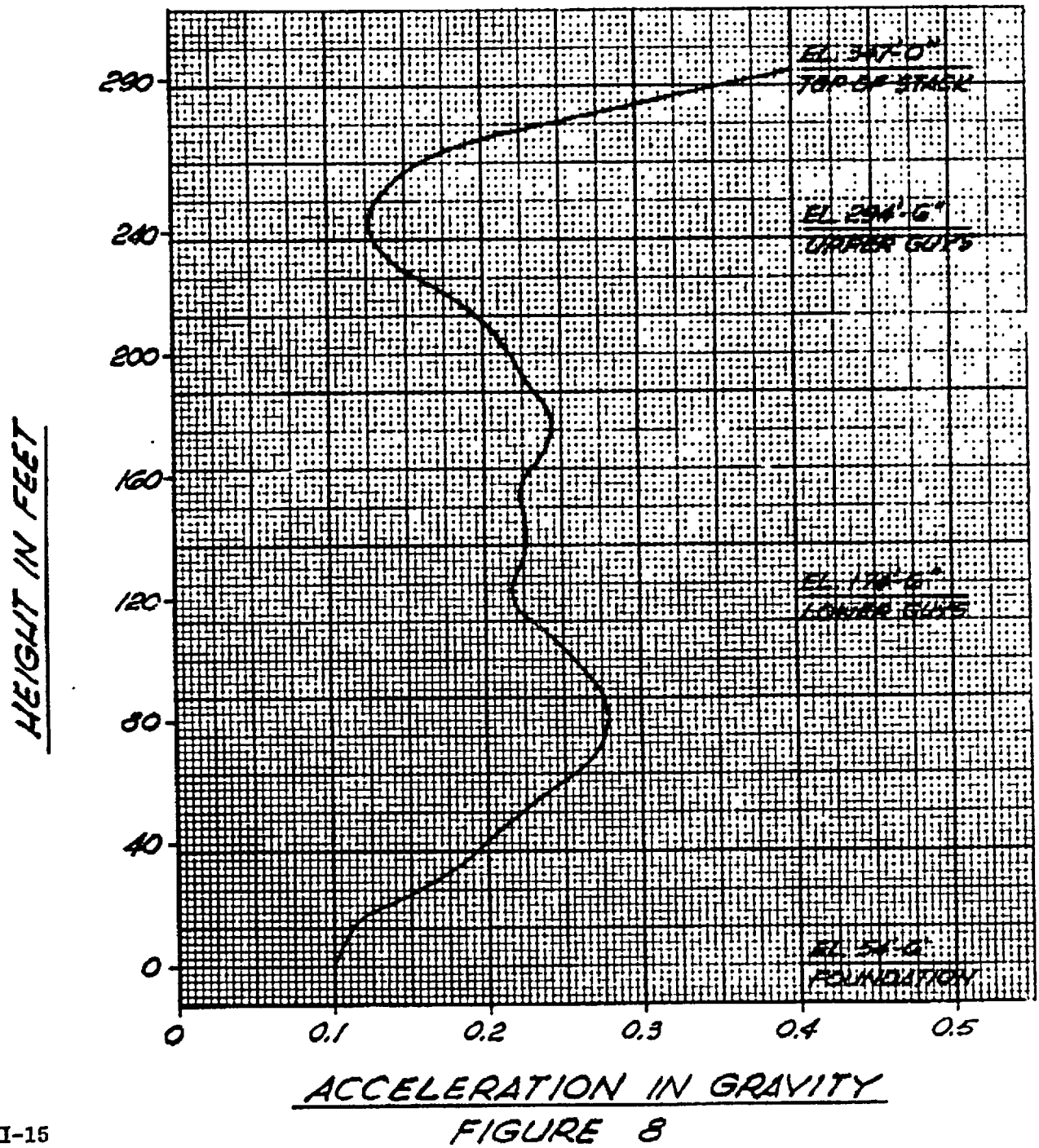

FIGURE 8

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MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK

SEISMIC RESULTS

PERIOD RANGE		
MODE	PERIOD (SECOND)	
	FLEXIBLE CASE	STIFF CASE
1	3.13	1.48
2	1.46	1.03
3	0.58	0.57
4	0.28	0.28
5	0.17	0.17
6	0.11	0.11
7	0.08	0.08
8	0.06	0.06

MAXIMUM FORCE IN EACH GUY	
UPPER GUY	9.50 KIPS
LOWER GUY	8.78 KIPS

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NOTE: ABOVE FORCES INCLUDE ORIGINAL PRETENSION FORCE OF 6.81 KIPS AND 5.67 KIPS FOR THE UPPER AND LOWER GUYS, RESPECTIVELY.

FIGURE 9

WIND RESULTS

MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK
WIND SHEARS

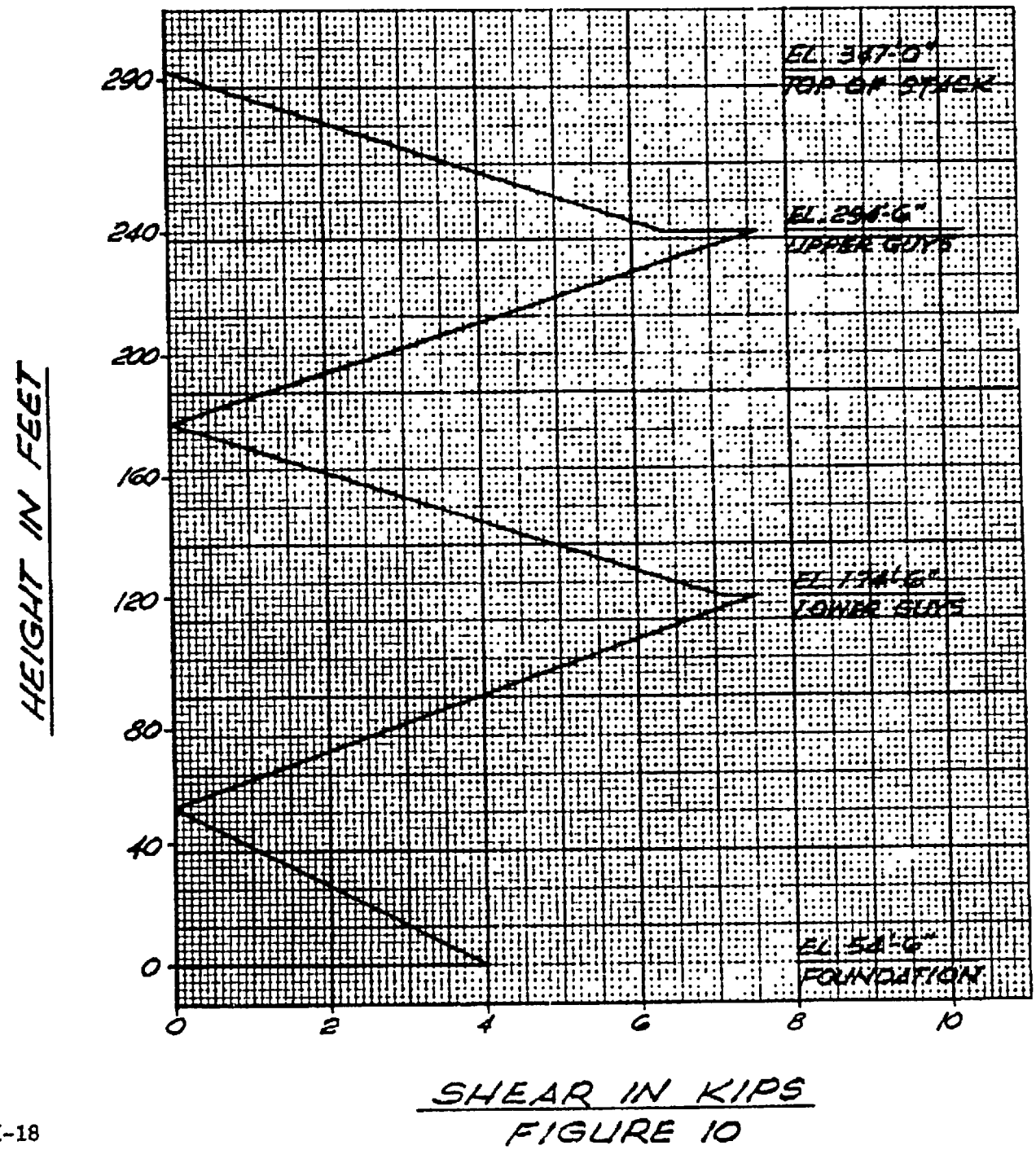

FIGURE 10

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MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK
WIND MOMENTS

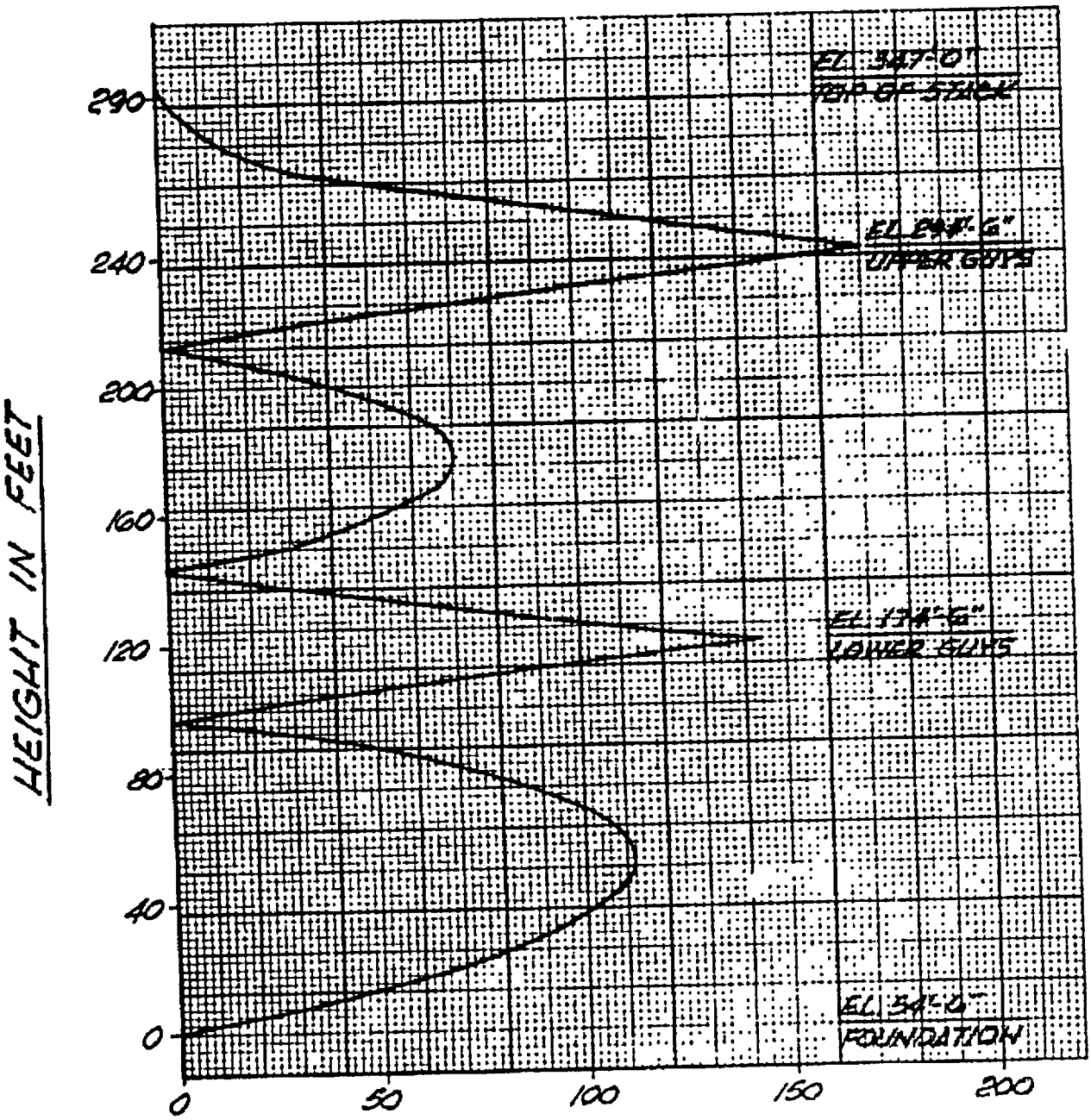

FIGURE 11

MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK
WIND DISPLACEMENTS

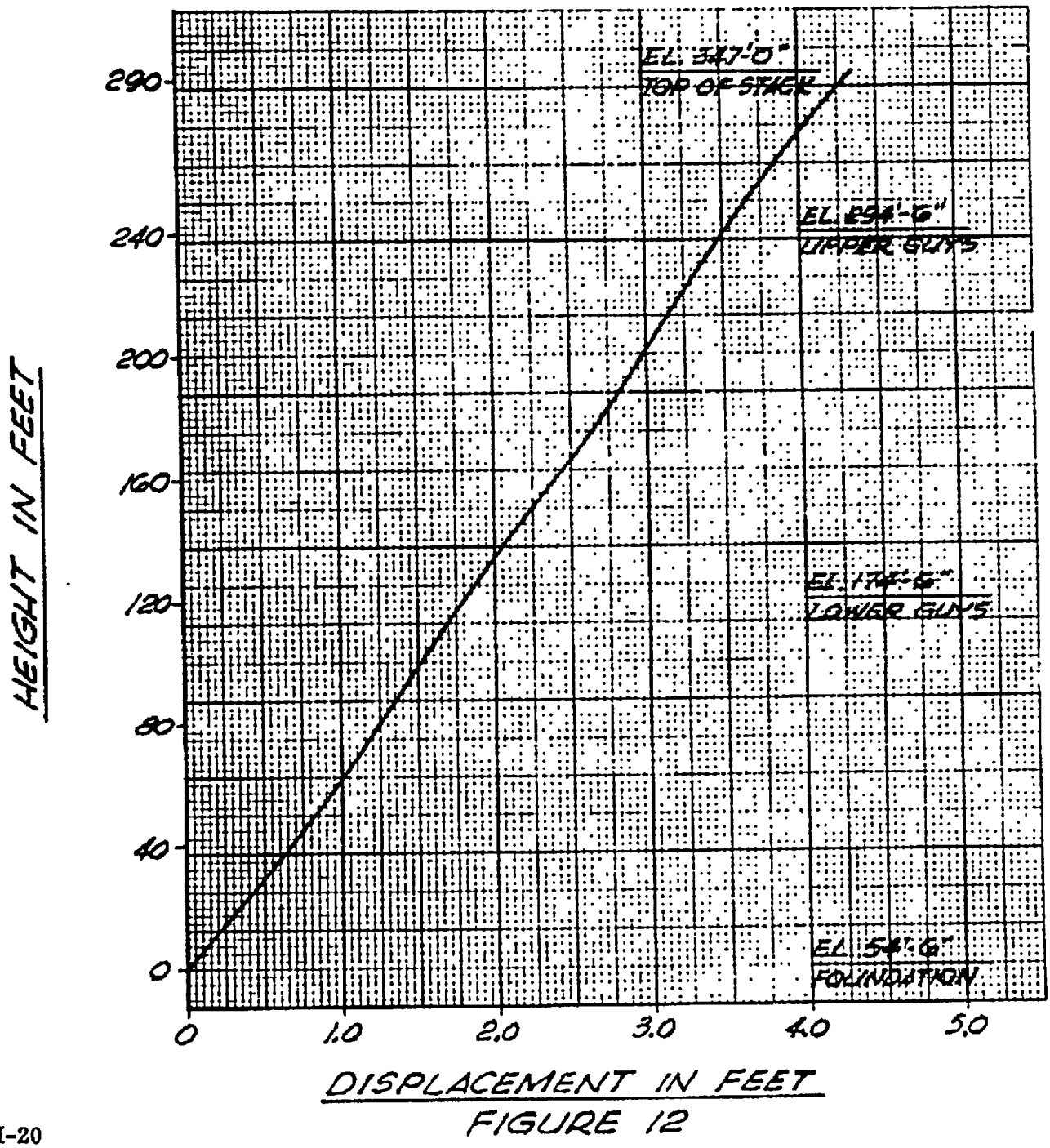

FIGURE 12

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MIDWEST FUEL RECOVERY PLANT
PLANT EFFLUENT STACK

WIND RESULTS

MAXIMUM FORCE IN EACH GUY	
UPPER GUY	21.71 KIPS
LOWER GUY	17.28 KIPS

NOTE: ABOVE FORCES INCLUDE ORIGINAL PRETENSION FORCE OF 6.81 KIPS AND 5.67 KIPS FOR THE UPPER AND LOWER GUYS, RESPECTIVELY.

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FIGURE 13

CALCULATIONS

LUMPED MASSES

WT. PER FOOT:

3/8" STEEL @ .490 x .332 FT.^{2} = .1627 K/FT.

16 GA. SS LINER $\pi \times \frac{3.33}{1000} \times \frac{40}{16}$ = .0261

.1888 K/FT.

MASS PER FOOT:

.1888/32.2 = .00586 $\frac{K\text{-}SEC^2}{FT.}$

CABLE

1¼"φ HALF WT. $\frac{1}{2} \times \frac{2.63}{1000} \times 316$ FT. = .4155 K

HALF MASS .4155/32.2 = .0129 $\frac{K\text{-}SEC^2}{FT.}$

1⅛"φ HALF WT. $\frac{1}{2} \times \frac{2.13}{1000} \times 235.5$ FT. = .2508 K

HALF MASS .2508/32.2 = .0078 $\frac{K\text{-}SEC^2}{FT.}$

MASS PT. 29 = 7.5 FT. x .00586 = .0440 $\frac{K\text{-}SEC^2}{FT.}$

MASS PT. 24 = 10 FT. x .00586 + 3x.0129 = .0973 $\frac{K\text{-}SEC^2}{FT.}$

MASS PT. 12 = 10 FT. x .00586 + 3x.0078 = .0820 $\frac{K\text{-}SEC^2}{FT.}$

TYPICAL MASS = 10 FT. x .00586 = .0586 $\frac{K\text{-}SEC^2}{FT.}$

SECTION PROPERTIES

DATA

$E = 30 \times 10^3$ KSI

OD = 3.417 FT.

ID = 3.354 FT.

t = .375 IN. = 0.0313 FT. (WALL THICKNESS)

AREA = $A = 0.785(3.417^2 - 3.354^2) = 0.332$ FT.^{2}

MOMENT OF INERTIA = $I = 0.0491(3.417^4 - 3.354^4) = 0.476$ FT.^{4}

SPRINGS REPRESENTING CABLE GUYS

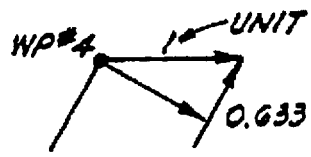
BASIC DATA (TAKEN FROM EDWARD'S WIRE ROPE)

$E = 12{,}000{,}000$ PSI

$A = 0.352\ d^2$

K_1 CALCULATION (UPPER LIMIT)

CABLE FROM WP #1 TO WP #4 (L = 316')



FORCE IN CABLE DUE TO UNIT DISP.

$T_{1-4} = \frac{.633}{316} \times 12 \times 10^3 \times .550 = 13.2^K$

$A = .550\ IN^2$

CABLES FROM WP #2 & #3 TO WP #4 (L = 316')

$T_{\substack{2-4 \\ 3-4}} = \frac{.316}{316} \times 12 \times 10^3 \times .550 = 6.6^K$

HORIZONTAL COMPONENTS IN CABLES

$H_{1-4} = .633 \times 13.2 = 8.35^K$

$H_{2-4} = .316 \times 6.6 = 2.09$

$H_{3-4} = .316 \times 6.6 = 2.09$

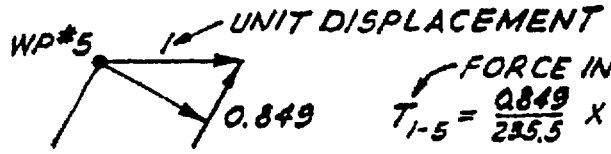
TOTAL HORIZ. FORCE = 12.53 K/FT. = K_1

BASED ON NO SAG ASSUMPTION

REPRESENTS UPPER LIMIT OF SPRING

K_2 CALCULATION (UPPER LIMIT)

CABLE FROM WP #1 TO WP #5 (L = 235.5')

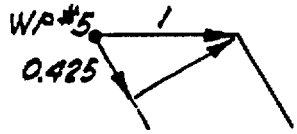


FORCE IN CABLE DUE TO UNIT DISP.

$T_{1-5} = \frac{0.849}{235.5} \times 12 \times 10^3 \times 0.445 = 19.2^K$

$A = .445\ IN^2$

CABLES FROM WP #2 & #3 TO WP #5 (L = 235.5')



$T_{\substack{2-5\\3-5}} = \frac{0.425}{235.5} \times 12 \times 10^3 \times 0.445 = 9.64^K$

HORIZONTAL COMPONENTS IN CABLES

$H_{1-5} = 0.849 \times 19.2 = 16.30^K$

$H_{2-5} = 0.425 \times 9.64 = 4.09$

$H_{3-5} = 0.425 \times 9.64 = 4.09$

TOTAL HORIZ. FORCE $= 24.48\ K/FT. = K_2$

BASED ON NO SAG ASSUMPTION

REPRESENTS UPPER LIMIT OF SPRING

CABLE STIFFNESS VARIATIONS

BASIC DATA

WT. OF UPPER CABLE = .831^K
WT. OF LOWER CABLE = .502^K
LENGTH OF UPPER CABLE = 316'
LENGTH OF LOWER CABLE = 235.5'
UPPER ANGLE = ARC TAN 245/200 = 50°46'
LOWER ANGLE = ARC TAN 125/200 = 32°00'
TENSION IN UPPER CABLE = .06 X 56.8 X 2 = 6.81^K
TENSION IN LOWER CABLE = .06 X 47.3 X 2 = 5.67^K

PROCEDURE

1. ASSUME MINIMUM PRESTRESS FORCE, T_B OF W/2 COS ϕ WHERE W = WT. OF CABLE AND ϕ = ANGLE OF CABLE INCLINATION WITHOUT SAG.

2. CALCULATE STRETCH IN CABLE, Δ

3. INCREASE PRESTRESS, SUCH THAT HORIZONTAL COMPONENT, H IS ONE KIP GREATER.

4. RE-CALCULATE STRETCH IN CABLE.

5. SUBTRACT (2) FROM (4) AND DETERMINE HORIZONTAL COMPONENT OF RESULT.

6. HORIZONTAL EFFECTIVE SPRING CONSTANT FOR CABLE IS THEREFORE THE RECIPROCAL OF (5).

7. REPEAT FOR ONE KIP INCREMENTS UP TO MAXIMUM PRESTRESS FORCE.

CABLE STIFFNESS VARIATIONS (CONT'D.)

UPPER CABLE

$\sin 50°46' = .77458$ $\quad A = .550\ IN^2$

$\cos 50°46' = .63248$ $\quad E = 12 \times 10^3\ KSI$

$\sec 50°46' = 1.5811$ $\quad \tan \rho = \tan \phi + \frac{.831}{2H}$

$$H = \frac{.63248}{2}\left[.831(.77458) + \sqrt{4T_B^2 - (.831 \times .63248)^2}\right]$$

$$= .107 + .316\sqrt{4T_B^2 - .2762}$$

$$\Delta = \frac{200H}{AE}\left(\sec^2\phi + \frac{.831^2}{12H^2}\right)$$

$$AE\Delta = 500H + \frac{11.51}{H}$$

$$H_{INITIAL} = .107^K$$

$$H_{TERMINAL} = .107 + .316\sqrt{4(6.81)^2 - .2762}$$

$$= 4.408^K$$

1.5 CABLES ACTING X COS ρ X K

H (KIPS)	Δ (FT.)	Δ DIFF (FT.)	H DIFF (KIPS)	K=SLOPE (KIPS/FT.)	K, HORIZ (KIPS/FT.)
.107	.0244				
		.0170	.393	23.11	6.66
.500	.0414				
		.0361	.500	13.85	9.08
1.000	.0775				
		.0749	1.000	13.35	10.42
2.000	.1524				
		.0754	1.000	13.26	11.02
3.000	.2278				
		.0757	1.000	13.21	11.34
4.000	.3035				
		.0308	.408	13.21	11.44
4.408	.3343				

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UPPER LIMIT FROM PREVIOUS K_1 CALCS 12.53

CABLE STIFFNESS VARIATIONS (CONT'D.)

UPPER CABLE (CONT'D.)

CONSIDER CASE WHERE TWO CABLES ARE AT MAXIMUM TENSION OF 2 TIMES PRESTRESS, AND OTHER ONE IS SLACK.

$$K_1 = 4.18 \text{ K/FT.}$$

THEREFORE, THE APPARENT NET STIFFNESS RANGE IS BETWEEN ABOUT 4 K/FT. TO A MAXIMUM OF 12.53 K/FT. FOR THIS ANALYSIS USE THREE CASES AND SELECT MAXIMUM RESULTS.

K_1 VALUES USED	
CASE NUMBER	K_1 (K/FT.)
1	4.0
2	8.0
3	12.0

24

CABLE STIFFNESS VARIATIONS (CONT'D.)

LOWER CABLE

$\sin 32°00' = 0.52992$ $\quad A = 0.445\ IN^2$

$\cos 32°00' = 0.84805$ $\quad E = 12 \times 10^3\ KSI$

$\sec 32°00' = 1.17918$ $\quad \tan \rho = \tan \phi + \frac{.502}{2H}$

$$H = \frac{.84805}{2}\left[.502(.52992) + \sqrt{4T_B^2 - (.502 \times .84805)^2}\right]$$

$$= .113 + .424\sqrt{4T_B^2 - .1815}$$

$$\Delta = \frac{200H}{AE}\left(\sec^2\phi + \frac{.502^2}{12H^2}\right)$$

$$AE\Delta = 278H + \frac{4.20}{H}$$

$$H_{INITIAL} = .113^K$$

$$H_{TERMINAL} = .113 + .424\sqrt{4(5.67)^2 - .1815}$$

$$= 4.923^K$$

H (KIPS)	Δ (FT.)	Δ DIFF (FT.)	H DIFF (KIPS)	K = SLOPE (KIPS/FT.)	K_1 HORIZ (KIPS/FT.)
.113	.0128				
		.0148	.387	26.15	12.93
.500	.0276				
		.0245	.500	20.40	20.35
1.000	.0521				
		.0519	1.000	19.27	21.70
2.000	.1040				
		.0520	1.000	19.23	23.10
3.000	.1560				
		.0520	1.000	19.23	23.55
4.000	.2080				
		.0480	.923	19.23	23.80
4.923	.2560				

UPPER LIMIT FROM PREVIOUS K_2 CALCS 24.48

CABLE STIFFNESS VARIATIONS (CONT'D.)

LOWER CABLE (CONT'D.)

CONSIDER CASE WHERE TWO CABLES ARE AT MAXIMUM TENSION OF 2 TIMES PRESTRESS, AND OTHER ONE IS SLACK.

$$K_2 = 8.16 \text{ K/FT.}$$

THEREFORE, THE APPARENT NET STIFFNESS RANGE IS BETWEEN ABOUT 8 K/FT. TO A MAXIMUM OF 24.48 K/FT. FOR THIS ANALYSIS USE THREE CASES AND SELECT MAXIMUM RESULTS.

K_2 VALUES USED	
CASE NUMBER	K_2 (K/FT.)
1	8.0
2	16.0
3	24.0

26

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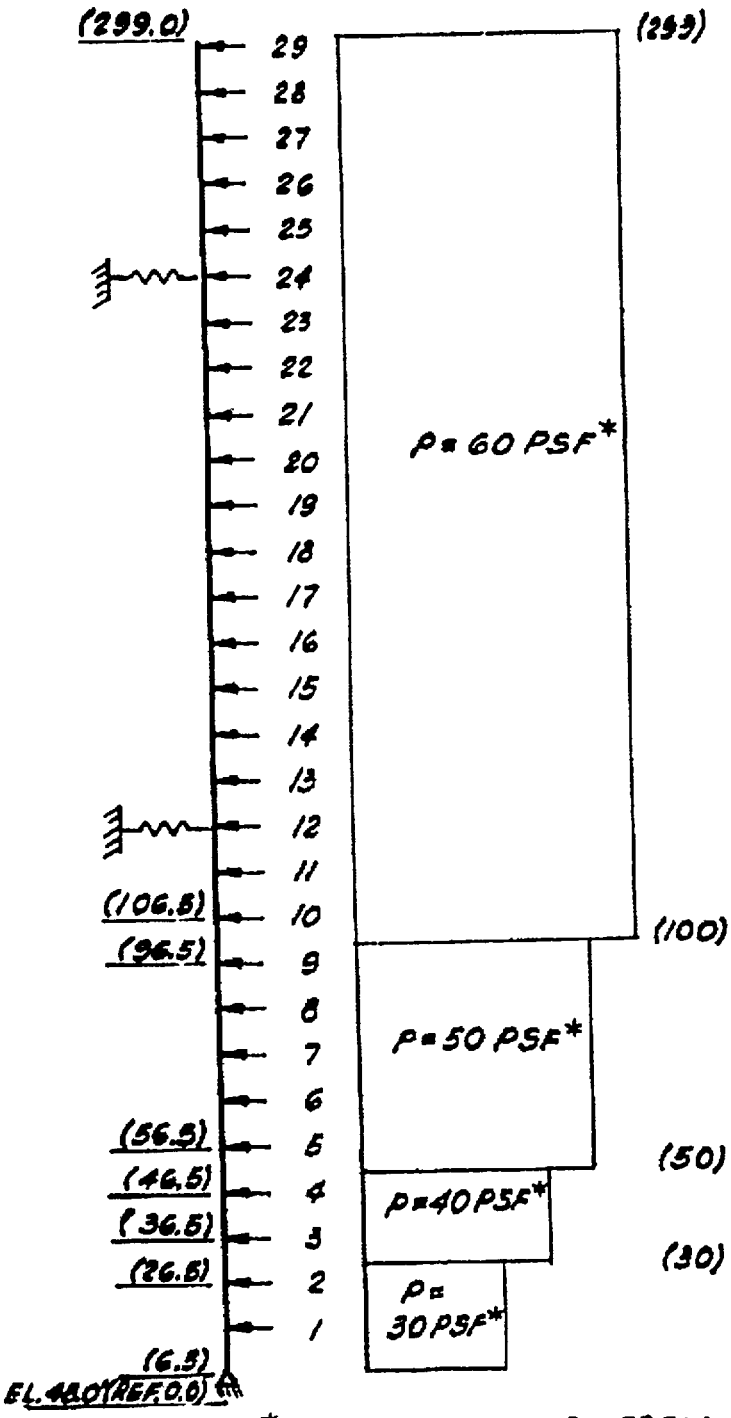
JOB NO. 1301-4 JOB MIDWEST FUEL RECOVERY PLANT BY TeH DATE 9/68
CLI FRO SUBJECT PLANT EFFLUENT STACK CHK'D dB DATE 9/68

WIND LOADING

SHAPE FACTOR = 0.6 FOR CYLINDER FROM UBC

$P_i = rpA_i$ = TRIBUTARY WIND LOAD TO MASS i



$P_1 = 0.6 \times 0.03 \times 3.417 \times 10.0$
$= 0.615^K$

$P_2 = 0.6(0.03 \times 8.5 + 0.04 \times 1.5) \times 3.417$
$= 0.646^K$

$P_3 = 0.6 \times 0.04 \times 3.417 \times 10.0$
$= 0.820^K$

$P_4 = 0.6(0.04 \times 8.5 + 0.05 \times 1.5) \times 3.417$
$= 0.861^K$

$P_5 \sim P_8 = 0.6 \times 0.05 \times 3.417 \times 10.0$
$= 1.025^K$

$P_9 = 0.6(0.05 \times 8.5 + 0.06 \times 1.5) 3.417$
$= 1.056^K$

$P_{10} \sim P_{28} = 0.6 \times 0.06 \times 3.417 \times 10.0$
$= 1.230^K$

$P_{29} = 0.6 \times 0.06 \times 7.5 \times 3.417$
$= 0.922^K$

27

A_i = TRIBUTARY PROJECTED AREA TO MASS POINT i

*PRESSURES TAKEN FROM FLUOR SPECIFICATION NO. SP-4204-00-2 "STRUCTURAL ENGINEERING AND DESIGN CRITERIA" PARAGRAPH 3.7.4 (110 MPH WIND).

WIND VIBRATION ANALYSIS

LATERAL VIBRATIONS DUE TO KARMAN VORTEX SHEDDING

$f = S\frac{v}{D}$

f = FREQUENCY OF VORTEX SHEDDING, IN CYCLES/SEC.
S = STROUHAL NUMBER (0.18-0.22)
v = VELOCITY OF WIND, IN FPS
V = VELOCITY OF WIND, IN MPH
D = DIAMETER OF STACK, IN FT.
$v = \frac{D}{S}f = \frac{D}{ST}$ (IN FPS) $= \frac{3600}{5280}\frac{D}{ST}$ (IN MPH)

$D = 3.417$ FT.
$S = 0.2$
$V = \frac{11.65}{T}$

MODE	MODE PERIOD (SEC.)	RESONANT VELOCITY (MPH)
FLEXIBLE CASE (CASE 1 CABLE STIFFNESS)		
1	3.13	4.0
2	1.46	8.0
3	0.58	20.0
4	0.28	41.0
STIFF CASE (CASE 3 CABLE STIFFNESS)		
1	1.48	8.0
2	1.03	11.0
3	0.57	20.0
4	0.28	42.0

28

WIND VIBRATION ANALYSIS (CONT'D.)

SUMMARY OF CONTROLLING RELATIVE ENERGY INPUTS FOR RESONANT WINDS

(BASED ON ENERGY INPUT OF 8 MPH = 1.00)

MODE NO.	CRITICAL RESONANT WIND VELOCITY (MPH)	RELATIVE ENERGY
1	8.0	1.00
2	11.0	9.16
3	20.0	5.63
4	42.0	0.09

FROM THE ABOVE TABLE, IT APPEARS THAT THE CRITICAL MODE FOR WIND VIBRATIONS IS THE SECOND MODE WHERE THE RESONANT WIND (11 MPH) WILL IMPART TO THE STACK ABOUT 9 TIMES THE ENERGY AS WOULD THE FIRST MODE'S 8 MPH RESONANT WIND. HOWEVER, THIS ENERGY LEVEL IS QUITE LOW. SINCE GUYED STACKS HAVE PERFORMED QUITE SATISFACTORILY AS FAR AS WIND VIBRATIONS ARE CONCERNED, THE MOTIONS AND STRESSES DUE TO THE ABOVE WIND INPUTS WILL NOT ADVERSELY EFFECT THE STACK.

29

WIND VIBRATION ANALYSIS (CONT'D.)

OVALING VIBRATIONS DUE TO KARMAN VORTEX SHEDDING

$f = 41{,}000 \frac{t}{D^2}$

t = PLATE THICKNESS, IN IN.

D = STACK DIAMETER, IN FT.

$t = 3/8$ IN. $D = 3.417$ FT.

$f = 41{,}000 \times 3/8 \times \frac{1}{3.417^2}$
$= 1{,}320$ CYCLES PER MIN.
$= 22$ CYCLES PER SEC.

$V = \frac{fD}{2S} = 11.65 f =$ 128 MPH < 110 MPH
THEREFORE NEGLECT STACK OVALING IN RESONANCE WITH THE KARMAN VORTEX SHEDDING.

END

NEDO-21326

APPENDIX B-4

EARTHQUAKE AND WIND ANALYSIS
UNDERGROUND STRUCTURES

MARCH, 1969

Morris Operation
General Electric Company
Morris, Illinois

REPORT ON THE EARTHQUAKE ANALYSES OF THE SAND FILTER, VAULTS, TANKS AND DUCTS FOR THE MIDWEST FUEL RECOVERY PLANT

This report presents the results of dynamic earthquake analyses of the sand filter, vaults, tanks and ducts for the Midwest Fuel Recovery Plant at Morris, Illinois. The sand filter is supported on rock and is surrounded by an earth berm. Vaults, tanks and ducts are embedded in sound rock. All are of reinforced concrete construction.

ANALYTICAL CRITERIA

The analyses are based on the following data (see Reference 1):

1. Design Earthquake

   North - South component of the 1940 El Centro earthquake normalized to 0.10g.

2. Damping

   Liquid sloshing 0.005

ANALYTICAL METHOD

Sand Filter

The filter structure, earth berm and the filter material are individually considered as lumped masses supported by weightless elastic columns fixed at the foundation. Elastic properties of the columns are determined by considering shear deformation only.

Vaults and Tanks

Hydro-dynamic effects were a main consideration in the analysis of the vaults and tanks. Since the mathematically-precise procedure is very complex, a simplified approach based on References 3, 4, 5, 6 and 7 is used. When a tank containing fluid of weigh: W is accelerated in a horizontal direction, a certain portion of the fluid behaves similarly to a solid mass in rigid contact with the wall. This mass exerts a maximum horizontal force directly proportional to the maximum acceleration of the tank bottom. The acceleration also causes another portion of the fluid to respond as though it were a solid oscillating mass flexibly connected to the walls. The maximum amplitude of the mass relative to the walls determines both the maximum vertical displacement of the water surface (slosh height) and the horizontal force exerted on the walls. Figure A provides dynamic constants used in determining the period and magnitude of sloshing. In this figure, alpha is the ratio of twice the height to the average width of the tank.

DISCUSSION OF RESULTS

Sand Filter

The results of the filter structure, earth berm and filter material are tabulated on page 13. The filter material and berm have about the same response; the filter structure is more flexible than the other two elements, and a uniform lateral pressure of 30 pounds per square foot is required for the seismic design of the walls.

Vaults and Tanks

Since the vaults and tanks are embedded in sound rock, the lateral soil pressures on these elements are assumed negligible, and only the slosh height of the fluid was calculated. Following are the tabulated results:

ELEMENT	SLOSH HEIGHT (FT)	
	Design Earthquake	Shutdown Earthquake
1. Cladding Vault	0.4	0.8
2. Low Activity Waste Vault	0.4	0.8
3. Chemical Vault	0.51	1.02
4. Fuel Unloading Pit	0.4	0.8
5. Fuel Transfer Pit	0.2	0.4
6. Waste Storage Basin	0.53	1.06
7. Fuel Storage Basin	0.56	1.12

Ducts

Air distribution tunnel, asphalt vault tunnel, waste-disposal cart tunnel and pipe tunnel are embedded in sound rock and the design earthquake will not have significant effect in their design.

MIDWEST FUEL RECOVERY PLANT

SAND FILTER, VAULTS, TANKS AND DUCTS

REFERENCES

1. General Electric Company

   Design and Analysis Report for the Midwest Fuel Recovery Plant, including Amendments 1 and 3.

2. The Fluor Corporation, Ltd.

   Main Process Building Structural Drawings

   SK5 - 3915 through SK5 - 3918
   SK5 - 4554
   Sk5 - 4772
   4204 - 5 - 1116A & B
   4204 - 5 - 1117A & B
   4204 - 5 - 1118A & B
   4204 - 5 - 1100A
   4204 - 5 - 1101A
   4204 - 5 - 0526A

   Design Criteria for Underground Vaults, DS - 27, 29, 30, 31.

3. TID - 7024, Nuclear Reactors and Earthquakes, United States Atomic Energy Commission.

4. Housner, G. W., Dynamic Pressures on Accelerated Fluid Containers, Bull. Seismological Society of America, Vol. 24, 1934

5. Hoskins, L. M., and Jacobsen, L. S., Water Pressure in a Tank Caused by a Simulated Earthquake, Bull. Seismological Society of America, Vol. 24, 1934

6. Jacobsen, L. S., and Ayre, R. S., Hydro-dynamic Experiments of Rigid Cylindrical Tanks Subjected to Transient Motions, Bull. Seismological Society of America, Vol. 41. 1951

7. Jacobsen, L. S., Impulsive Hydro-dynamics of Fluid Inside a Cylindrical Tank and of a Fluid Surrounding a Cylindrical Pier, Bull. Seismological Society of America, Vol. 39, 1949

MIDWEST FUEL RECOVERY PLANT

SAND FILTER, VAULTS, TANKS AND DUCTS

LIST OF FIGURES AND RESULTS

Figure 1
Vault and Basin Location Plan ---------- 7

Calculations and Results

Sand Filter Structure ---------- 8-14
Cladding Vault ---------- 15-16
Chemical Vault ---------- 17-18
Low Activity Waste Vault ---------- 19
Waste Storage Basin ---------- 20-21
Fuel Storage Basin ---------- 22-23
Fuel Unloading Pit ---------- 24-25
Fuel Transfer Pit ---------- 26-27

Figure A
Hydro-dynamic Constants for Rectangular and Cylindrical Tanks ---------- 28

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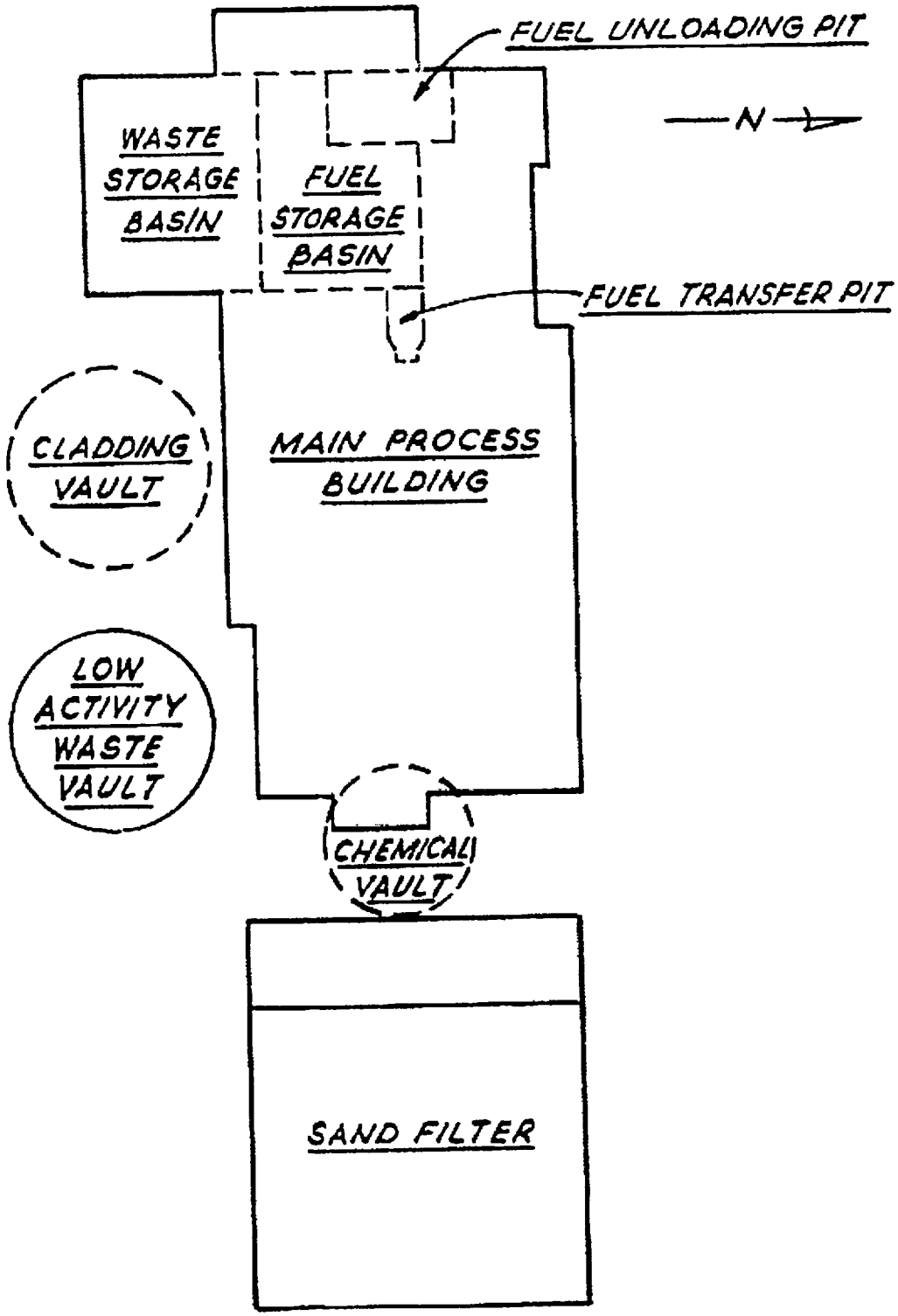

VAULT & BASIN LOCATION PLAN

FIGURE 1

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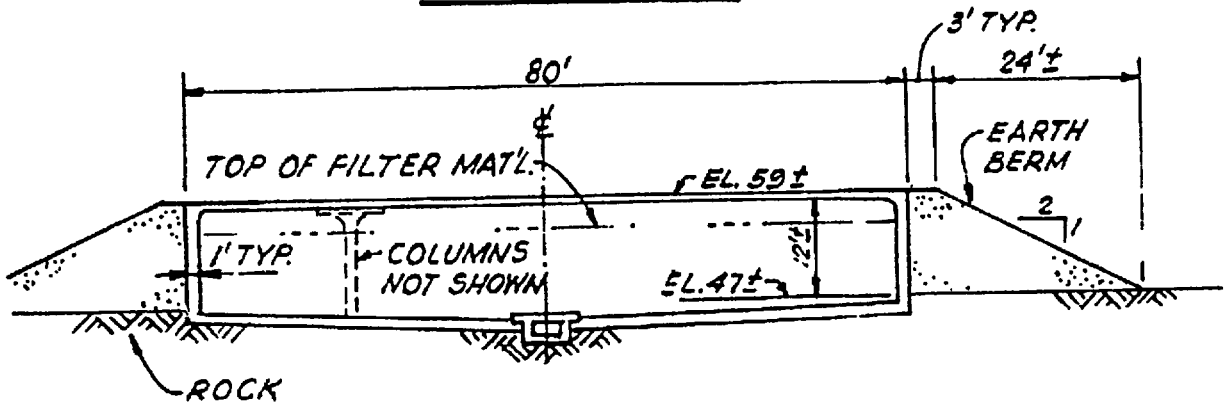
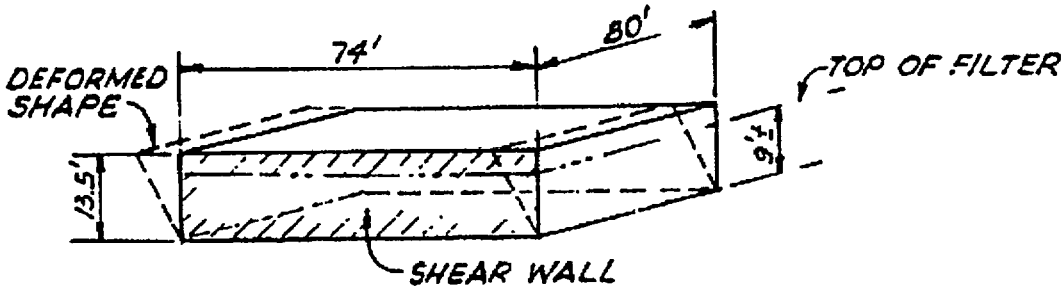
JOB NO 1301-3 JOB MIDWEST FUEL RECOVERY PLANT BY SJK DATE 15 DE
CLIENT FRO SUBJECT SAND FILTER STRUCTURE CHK'D CYC DATE 15 DE

SEISMIC REVIEW

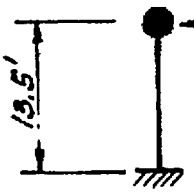


SECTION THROUGH
SAND FILTER STRUCTURE

A. STRUCTURE ALONE



1. MODEL



MASS: TRIB. WT. OF WALLS,
FULL WT. OF ROOF SLAB

STIFFNESS: DUE TO SHEAR DEFORMATIONS
OF 74' LONG SHEAR WALL.

2. MASS

74' WALLS: 13.5 X 74 X 2 X ½ X 1.0 X .15 = 150^K
80' WALLS: 13.5 X 80 X 2 X ½ X 1.0 X .15 = 162
ROOF: 80 X 74 X 1 X .15 = 886
TOTAL WT. = 1,198^K

SHEET N

SF-1

1. MODEL (CONT'D.)

STIFFNESS

$K = \frac{G\bar{A}}{L}$: $\bar{A}$ = SHEAR AREA, $G = .40E$, $L = 13.5'$

$K = \frac{.40 \times 432{,}000\,\bar{A}}{13.5} = 12{,}800\,\bar{A}$

$\bar{A} = 2 \times 1 \times 74 = 148\ FT.^2$

$K = 12{,}800 \times 148 = 1.89 \times 10^6\ K/FT.$

2. PERIOD OF VIBRATION

$T = 2\pi\sqrt{\frac{M}{K}}$

$T = 6.28\sqrt{\frac{1198 \times 10^{-6}}{32.2 \times 1.89}} = 0.03$ SEC. (RIGID!)

3. RESPONSE

SINCE STRUCTURE IS RIGID USE

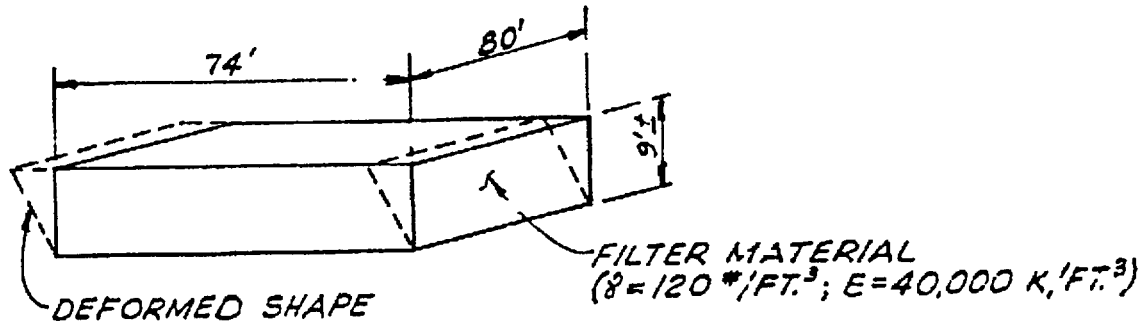
$S_A = 0.10\,g$

$F_{INERTIA} = 0.10 \times 1198 \approx 120^K$

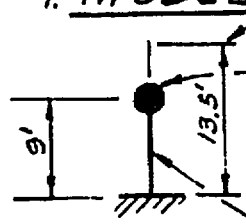
DISPLAC = $\Delta_{STR} = 120/1.89 \times 10^6 = 63.5 \times 10^{-6}$ FT.

SHEET N

B. FILTER MATERIAL ALONE



1. MODEL



MASS: (1) FOR PERIOD USE ⅓ (WT. OF FILTER)
(2) FOR DISPLAC. USE UNIFORM DISTRIB. OVER 9' & EXTEND LINEARLY TO GET EQUIVALENT DISPLAC. OF "a".

STIFFNESS: SHEAR DEFORMATIONS ONLY.

MASS

TOTAL SOIL WT.: 74 X 80 X 9 X .120 = 6380 K

WT. FOR PERIOD CALC.: ⅓ X 6380 = 2127 K

WT. FOR DISPLAC.: 6380/9 = 709 K/FT.

STIFFNESS

$\bar{A} = 5/6 \times 74 \times 80 = 4{,}930 \text{ FT.}^2$

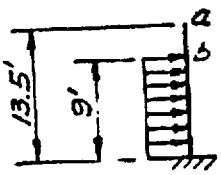
$$K = \frac{.4 \times 40{,}000 \times 4{,}930}{9} = \underline{\underline{8.77 \times 10^6 \text{ K/FT.}}}$$

2. PERIOD OF VIBRATION

$$T = 6.28\sqrt{\frac{2127 \times 10^{-6}}{32.2 \times 8.77}} = \underline{0.02 \text{ SEC. (RIGID!)}}$$

3. RESPONSE

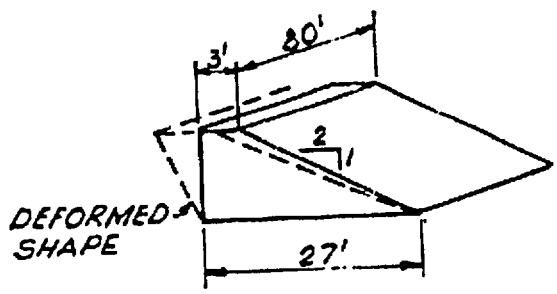
SINCE STRUCTURE IS RIGID, USE $S_A = 0.10g$



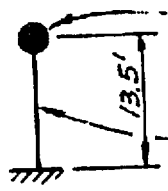
$$\Delta_{ab_{SP}} = \tfrac{1}{2}\left[\frac{.10 \times 709 \times 9 \times 9}{16{,}000 \times 4{,}930}\right] = 36.4 \times 10^{-6}$$

$$\Delta_{a_{SP}} = 13.5/9 \times 36.4 \times 10^{-6} = \underline{54.5 \times 10^{-6} \text{ FT.}}$$

C. BERM ALONE



1. MODEL



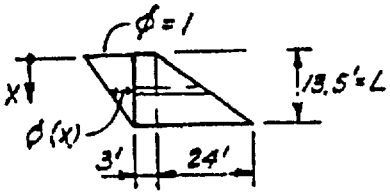
MASS: (1) FOR PERIOD, USE GEN. MASS, M^*
(2) FOR DISPLAC. USE DISTRIB. MASS

STIFFNESS: SHEAR DEFORMATIONS ONLY

SEISMIC REVIEW

MASS

GENERALIZED MASS FOR PERIOD:



$$W^* = \int_0^L W(x)\cdot\phi(x)^2\,dx$$

$$\phi(x) = \left(1 - \frac{X}{L}\right)$$

$$W(x) = .120 \times 80\left[3 + 24\frac{X}{L}\right]$$

$$W^* = 9.6\int_0^L \left[3 + 24\frac{X}{L}\right]\left[1 - \frac{X}{L}\right]^2 dx = 389^K$$

STIFFNESS

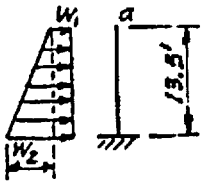
$$\Delta = \frac{1}{K} = \frac{1}{G}\int_0^L \frac{dx}{(200 + 118.3X)} \; ; \quad K = .864 \times 10^6 \; K/FT.$$

2. PERIOD OF VIBRATION

$$T = 6.28\sqrt{\frac{389 \times 10^{-6}}{32.2 \times .825}} = 0.02 \text{ SEC. (RIGID!)}$$

3. RESPONSE

SINCE STRUCTURE IS RIGID, USE $S_A = 0.10g$



$W_1 = 28.8 \; K/FT. \times .10 = 2.88 \; K/FT.$

$W_2 = 230.0 \; K/FT. \times .10 = 23.00 \; K/FT.$

$$\Delta = \frac{1}{G}\int_0^L \frac{V(X)\,dx}{\bar{A}(X)} = \frac{1}{G}\int_0^L \frac{W_1 X\,dx}{(200 + 118.3X)} + \frac{1}{G}\int_0^L \frac{W_2 X^2\,dx}{2L(200 + 118.3X)}$$

$$\Delta = \underline{\underline{48.4 \times 10^{-6} \text{ FT.}}}$$

SHEET NO.

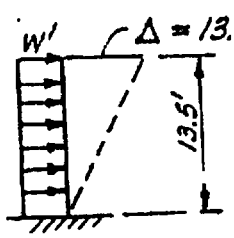
SUMMARY OF RESPONSE			
ITEM	PERIOD (SEC.)	ACCEL. (G's)	DISPLAC. (FT.)
FILTER STRUCTURE	.03	.10	63.5×10^{-6}
FILTER MATERIAL	.02	.10	54.5×10^{-6}
BERM	.02	.10	48.4×10^{-6}

CONCLUSIONS:

1. THE THREE ELEMENTS HAVE ABOUT THE SAME RESPONSE, EXCEPT THE FILTER STRUCTURE ITSELF IS SOMEWHAT MORE FLEXIBLE THAN EITHER THE FILTER MATERIAL OR THE BERM.

2. AS THE FILTER STRUCTURE MOVES, IT IS RESTRAINED EQUALLY BY THE FILTER MATERIAL AND THE BERM, THUS PRODUCING A LOADING ON THE STRUCTURE WALLS. THE ACTUAL SHAPE OF THIS LOAD PATTERN IS NOT KNOWN, BUT FOR DESIGN PURPOSES AN EQUIVALENT UNIFORM LOADING PATTERN WILL BE ASSUMED SUCH THAT IT RESULTS IN A FILTER STRUCTURE DISPLACEMENT OF $(63.5-48.4) \times 10^{-6} = 15.1 \times 10^{-6}$ FT.

DETERMINATION OF WALL LOADING



$$\Delta = \frac{1}{2}\frac{(W'L)L}{G\bar{A}}$$

$\bar{A} = 148\ FT.^2$, $L = 13.5'$, $\Delta = 15.1 \times 10^{-6}\ FT.$

$G = .4 \times 432{,}000 = 172{,}800\ K/FT.^2$

$$W' = \frac{2\Delta G\bar{A}}{L^2} = \frac{2(15.1 \times 10^{-6})(172{,}800)(148)}{(13.5)^2}$$

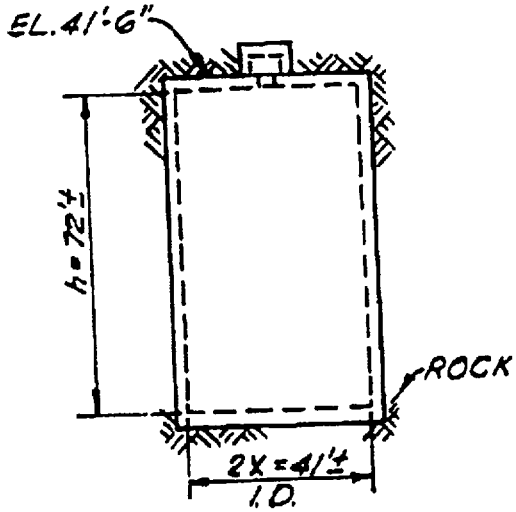
$W' = 4.23\ K/FT.$ TOTAL

$W'' = 4.23/2 = 2.12\ K/FT.$ EACH WALL

$W = 2.12/80 = 26.5\ \#/FT.^2$

FOR DESIGN PURPOSES (SEISMIC) USE A UNIFORM LATERAL WALL PRESSURE OF 30 PSF ACTING EITHER INWARD OR OUTWARD ON WALL.

SEISMIC REVIEW



ELEVATION

A. LATERAL SOIL PRESSURES

SINCE CLADDING VAULT IS EMBEDDED IN SOUND ROCK, IT IS ASSUMED THAT THE VAULT WILL NOT HAVE TO RESIST ANY INERTIA FORCES FROM THE ROCK MASS ITSELF. THEREFORE, THE LATERAL PRESSURES ON THE VAULT DUE TO EARTHQUAKE ARE ASSUMED NEGLIGIBLE.

B. MAGNITUDE OF SLOSHING

1. PERIOD OF SLOSHING

ASPECT RATIO $= h/x = 72'/20.5 = 3.51$

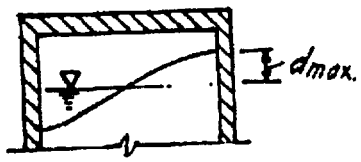
$C_1(h/x) = C_1(3.51) = 1.84$

$C_2(h/x) = C_2(3.51) = 1.53$

$C_3(h/x) = C_3(3.51) = 0.410$

$$T = 2\pi\sqrt{\frac{X}{gC_1}} = 6.28\sqrt{\frac{20.5}{32.2(1.84)}} = 3.7 \text{ SEC.}$$

2. MAGNITUDE OF SLOSHING



$S_A(T = 3.7) = 0.03g$
(DAMPING = 0.5%)

$$d_{max.} = \frac{C_2 C_3 S_A X}{1 - C_2 S_A} = \frac{(1.53)(.410)(.03)(20.5)}{1-(1.53)(.03)} = .$$

SHEET

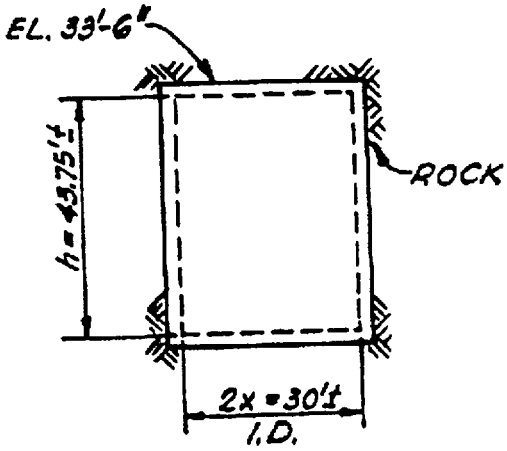
CV-.

3. CONCLUSIONS

a) IF SOME FREEBOARD IS REQUIRED BETWEEN THE FLUID SURFACE AND VAULT ROOF SOFFIT, A MINIMUM OF ABOUT 6" IS NECESSARY FOR THE DESIGN EARTHQUAKE AND 1'-0" FOR THE SHUTDOWN EARTHQUAKE.

b) IF FREEBOARD IS NOT PROVIDED, THE ROOF OF THE VAULT MUST BE DESIGNED TO RESIST AN UPWARD PRESSURE EQUAL TO A HEAD OF 6" FOR THE DESIGN EARTHQUAKE AND 1'-0" FOR THE SHUTDOWN EARTHQUAKE.

SEISMIC REVIEW



ELEVATION

A. LATERAL SOIL PRESSURES

LATERAL SOIL PRESSURES WOULD BE NEGLIGIBLY SMALL FOR REASONS STATED FOR CLADDING VAULT.

B. MAGNITUDE OF SLOSHING

1. PERIOD OF SLOSHING

ASPECT RATIO = $h_1/x = 43.75/15 = 2.92$

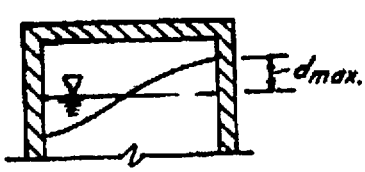
$C_1(h/x) = C_1(2.92) = 1.83$

$C_2(h/x) = C_2(2.92) = 1.53$

$C_3(h_1/x) = C_3(2.92) = 0.410$

$$T = 2\pi\sqrt{\frac{X}{gC_1}} = 6.28\sqrt{\frac{15}{32.2(1.83)}} = 3.2 \text{ SEC.}$$

2. MAGNITUDE OF SLOSHING



$S_A(T = 3.2 \text{ SEC.}) = 0.05g$
(DAMPING = 0.5%)

$$d_{max.} = \frac{C_2 C_3 S_A X}{1 - C_2 S_A} = \frac{(1.53)(.410)(.05)(15)}{1-(1.53)(.05)} = .51 \text{ FT.}$$

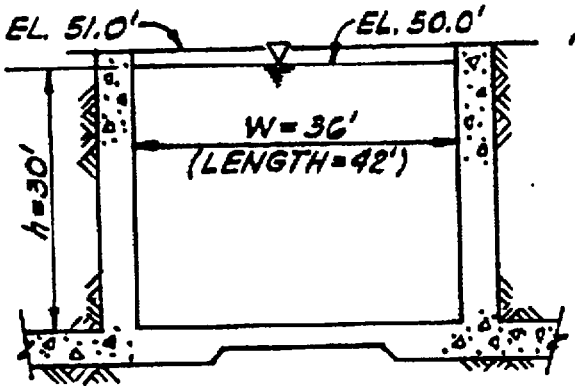
SHEET NO.

CHV-1

3. CONCLUSIONS

FOR DESIGN PURPOSES, THE SAME CONCLUSIONS GENERATED FOR THE CLADDING VAULT APPLY TO THE CHEMICAL VAULT.

A. THE DIMENSIONS, SHAPE, AND EMBEDMENT OF THE LOW ACTIVITY WASTE VAULT IS APPROXIMATELY THE SAME AS THE CLADDING VAULT. ALTHOUGH THE DENSITY OF THE CONTAINED FLUID IN THE LOW ACTIVITY WASTE VAULT IS SOMEWHAT GREATER THAN THE 62.4 POUNDS PER CUBIC FOOT IN THE CLADDING VAULT, THE MAGNITUDE OF SLOSHING AND GENERAL CONCLUSIONS DETERMINED FOR THE CLADDING VAULT APPLY ALSO TO THE LOW ACTIVITY WASTE VAULT.



SECTION

A. LATERAL SOIL PRESSURES

SINCE WASTE STORAGE BASIN IS EMBEDDED IN SOUND ROCK LATERAL PRESSURES MAY BE ASSUMED NEGLIGIBLE.

B. MAGNITUDE OF SLOSHING

1. PERIOD OF SLOSHING (ASSUME 2X = 39'; AVG. OF LENGTH AND WIDTH)

ASPECT RATIO = $h/x = 30/19.5 = 1.54$

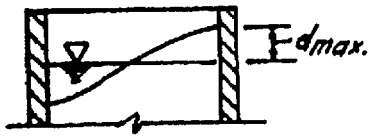
$C_1(h/x) = C_1(1.54) = 1.57$

$C_2(h/x) = C_2(1.54) = 1.57$

$C_3(h/x) = C_3(1.54) = 0.55$

$$T = 2\pi\sqrt{\frac{x}{gC_1}} = 6.28\sqrt{\frac{19.5}{32.2(1.57)}} = 3.9 \text{ SEC.}$$

2. MAGNITUDE OF SLOSHING



$S_A(T = 3.9) = 0.03g$
(DAMPING = 0.5%)

$$d_{max.} = \frac{C_2 C_3 S_A X}{1 - C_2 S_A} = \frac{(1.57)(.55)(.03)(19.5)}{1-(1.57)(.03)} = .53 \text{ FT.}$$

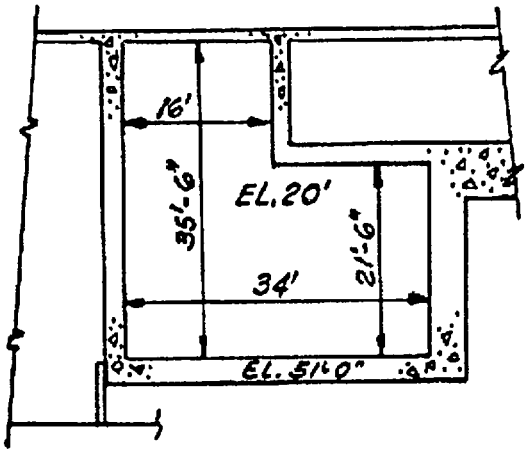
SHEET NO.

3. CONCLUSIONS

a) SINCE THERE IS NO STRUCTURAL TOP TO THE WASTE STORAGE BASIN, SUFFICIENT FREEBOARD WILL BE REQUIRED TO ACCOMODATE THE CALCULATED SLOSHING FOR BOTH THE DESIGN AND SHUTDOWN EARTHQUAKE.

b) FOR THE DESIGN EARTHQUAKE, THE MAXIMUM SLOSHING WILL BE 0.53 FOOT. FOR SHUTDOWN, THE MAXIMUM SLOSHING WILL BE TWICE THIS AMOUNT OR 1.06 FEET. ALTHOUGH ONLY 1.0 FOOT OF FREEBOARD IS PROVIDED, THIS SHOULD BE SUFFICIENT FOR DESIGN PURPOSES.

SEISMIC REVIEW



PLAN

A. LATERAL SOIL PRESSURES

SINCE FUEL STORAGE BASIN IS EMBEDDED IN SOUND ROCK, LATERAL PRESSURES MAY BE ASSUMED NEGLIGIBLE.

B. MAGNITUDE OF SLOSHING

REPRESENT ACTUAL TANK WITH AN EQUIVALENT SQUARE TANK HAVING SAME AREA AND DEPTH.
AREA = 955 SF, 2X = 30.9 FT., h = 30 FT.

1. PERIOD OF SLOSHING

ASPECT RATIO = $h/x = 30/15.45 = 1.94$

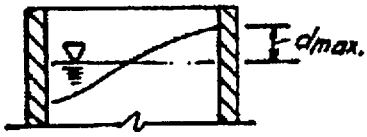
$C_1(h/x) = C_1(1.94) = 1.58$

$C_2(h/x) = C_2(1.94) = 1.58$

$C_3(h/x) = C_3(1.94) = 0.54$

$$T = 2\pi\sqrt{\frac{X}{gC_1}} = 6.28\sqrt{\frac{15.45}{32.2(1.58)}} = 3.5 \text{ SEC.}$$

2. MAGNITUDE OF SLOSHING



$S_A(T = 3.5) = 0.04g$
(DAMPING = 0.5%)

$$d_{max} = \frac{C_2 C_3 S_A X}{1 - C_2 S_A} = \frac{(1.58)(.54)(.04)(15.45)}{1-(1.58)(.04)} = .56 \text{ FT.}$$

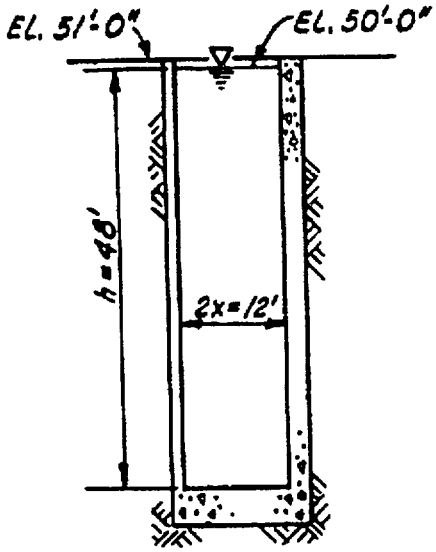
SHEET NO.

3. CONCLUSIONS

a) SINCE THERE IS NO STRUCTURAL TOP TO THE FUEL STORAGE BASIN, SUFFICIENT FREEBOARD SHOULD BE PROVIDED TO ACCOMODATE THE CALCULATED SLOSHING FOR BOTH THE DESIGN AND SHUTDOWN EARTHQUAKE.

b) FOR THE DESIGN EARTHQUAKE, THE MAXIMUM SLOSHING WILL BE 0.56 FOOT. FOR SHUTDOWN, THE MAXIMUM SLOSHING WILL BE ABOUT 1.12 FEET. THIS IS GREATER THAN THE 1.0 FOOT FREEBOARD PROVIDED. HOWEVER, THE EFFECT OF SUCH AN INDICATED OVERFLOW SHOULD BE EVALUATED TO DETERMINE IF ADDITIONAL FREEBOARD WOULD BE REQUIRED.

SEISMIC REVIEW



SECTION

A. LATERAL SOIL PRESSURES

SINCE FUEL UNLOADING PIT IS EMBEDDED IN SOUND ROCK, LATERAL PRESSURES MAY BE ASSUMED NEGLIGIBLE.

B. MAGNITUDE OF SLOSHING

1. PERIOD OF SLOSHING

ASPECT RATIO = $h/x = 48/6 = 8.0$

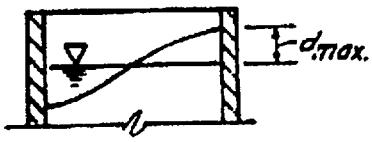
$C_1(h/x) = C_1(8.0) = 1.59$

$C_2(h/x) = C_2(8.0) = 1.59$

$C_3(h/x) = C_3(8.0) = 0.53$

$$T = 2\pi\sqrt{\frac{x}{gC_1}} = 6.28\sqrt{\frac{6}{32.2(1.59)}} = 2.2 \text{ SEC.}$$

2. MAGNITUDE OF SLOSHING



$S_A(T=2.2) = 0.07g$
(DAMPING = 0.5%)

$$d_{max.} = \frac{C_2 C_3 S_A X}{1 - C_2 S_A} = \frac{(1.59)(.53)(.07)(6)}{1-(1.59)(.07)} = 0.40 \text{ FT.}$$

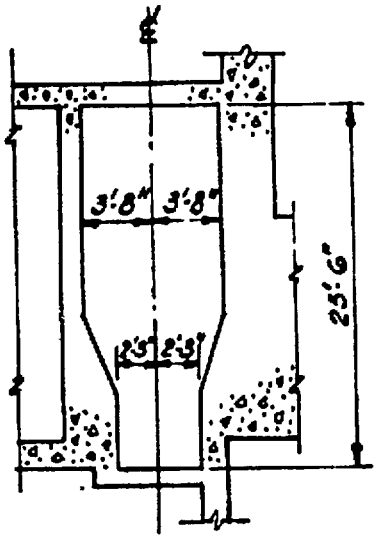
SHEET NO

3. CONCLUSIONS

a) SINCE THERE IS NO STRUCTURAL TOP TO THE FUEL UNLOADING PIT, SUFFICIENT FREEBOARD SHOULD BE PROVIDED TO ACCOMODATE THE CALCULATED SLOSHING FOR BOTH THE DESIGN AND SHUTDOWN EARTHQUAKE.

b) FOR THE DESIGN EARTHQUAKE, THE MAXIMUM SLOSHING WILL BE 0.40 FOOT. FOR SHUTDOWN, THE MAXIMUM SLOSHING WILL BE ABOUT 0.8 FOOT. THIS IS LESS THAN THE 1.0 FOOT FREEBOARD PROVIDED.

SEISMIC REVIEW



PLAN

A. LATERAL SOIL PRESSURES

SINCE FUEL TRANSFER PIT IS EMBEDDED IN SOUND ROCK, LATERAL PRESSURES MAY BE ASSUMED NEGLIGIBLE.

B. MAGNITUDE OF SLOSHING

USE AVERAGE WIDTH FOR 2X

$2X = \frac{1}{2}(7.5 + 4.5) = 6$ FT., $X = 3$ FT.

$h = 48$ FT.

1. PERIOD OF SLOSHING

ASPECT RATIO $= h/x = 48/3 = 16$

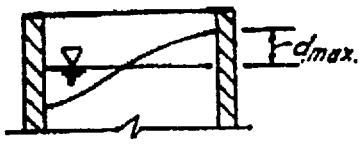
$C_1(h/x) = C_1(16) = 1.59$

$C_2(h/x) = C_2(16) = 1.59$

$C_3(h/x) = C_3(16) = 0.53$

$$T = 2\pi\sqrt{\frac{X}{gC_1}} = 6.28\sqrt{\frac{3}{32.2(1.59)}} = 1.5 \text{ SEC.}$$

2. MAGNITUDE OF SLOSHING



$S_A(T = 1.5) = 0.07g$

(DAMPING = 0.5%)

$$d_{max} = \frac{C_2 C_3 S_A X}{1 - C_2 S_A} = \frac{(1.59)(.53)(.07)(3)}{1-(1.59)(.07)} = 0.20 \text{ FT.}$$

SHEET NO

3. CONCLUSIONS

a) SINCE THERE IS NO STRUCTURAL TOP TO THE FUEL TRANSFER PIT, SUFFICIENT FREEBOARD SHOULD BE PROVIDED TO ACCOMODATE THE CALCULATED SLOSHING FOR BOTH THE DESIGN AND SHUTDOWN EARTHQUAKE.

b) FOR THE DESIGN EARTHQUAKE, THE MAXIMUM SLOSHING WILL BE 0.20 FOOT. FOR SHUTDOWN, THE MAXIMUM SLOSHING WILL BE ABOUT 0.40 FOOT. THIS IS LESS THAN THE 1.0 FOOT FREEBOARD PROVIDED.

END