From: "Hamer, Mike" <mhamer@entergy.com>
To: "Jonathan Rowley" <JGR@nrc.gov>
Date: Thu, Sep 7, 2006 12:02 PM
Subject: VHS reports

Attached to this e-mail are additional reports for Seismic design consideration and inspection report on VY dam.

Mail Envelope Properties (4500426F.7ED : 1 : 2029)

Subject:	VHS reports
Creation Date	Thu, Sep 7, 2006 12:01 PM
From:	"Hamer, Mike" <mhamer@entergy.com>
Created By:	mhamer@entergy.com

Recipients
nrc.gov
TWGWPO03.HQGWDO01
JGR (Jonathan Rowley)

Post Office	**Route**
TWGWPO03.HQGWDO01	nrc.gov

Files	**Size**	**Date & Time**
MESSAGE	119	Thursday, September 7, 2006 12:01 PM
TEXT.htm	1636	
seismic design (vy).pdf	446501	
Safety Insp (Vernon Project).PDF		2578147
Mime.822	1	

Options

Expiration Date:	None
Priority:	Standard
ReplyRequested:	No
Return Notification:	None
Concealed Subject:	No
Security:	Standard

Junk Mail Handling Evaluation Results
Message is eligible for Junk Mail handling
This message was not classified as Junk Mail

Junk Mail settings when this message was delivered
Junk Mail handling disabled by User
Junk Mail handling disabled by Administrator
Junk List is not enabled
Junk Mail using personal address books is not enabled
Block List is not enabled

VERMONT YANKEE NUCLEAR POWER CORPORATION

SEISMIC DESIGN CONSIDERATIONS FOR THE VERNON DAM AND THE VERMONT YANKEE REACTOR BUILDING

AMENDMENT NO. 6

MAY 24, 1967

INTRODUCTION

This document contains seismic design information requested by the Commission Staff. The material presented herein consists of:

I Stability analysis of the Vernon Dam.

II An analysis of the extent of cracking of the reactor building which might result from the postulated earthquake at the reactor site.

I STABILITY ANALYSIS OF THE VERNON DAM

A) SUMMARY

1) PURPOSE AND SCOPE

The purpose of this analysis was to determine the in-place stability of the Vernon Dam located on the Connecticut River at Vernon, Vermont. This analysis was performed in conjunction with the Vermont Yankee Nuclear Power Project which will be constructed on the west shore of the Connecticut River approximately 2500 feet north of the dam.

Particular attention was given to dynamic stability of the dam when affected by the maximum hypothetical earthquake for this site.

The scope of this analysis included the following:

a) Stability analysis and design review of the typical dam cross-section or block with a base elevation of 171.13 (USGS).

b) Stability analysis and design review of the dam block crossing the original river channel, which has a varying cross-section and a minimum base elevation of 141± (USGS).

Preliminary copies of the detailed stability computations have been submitted informally to Professor Newmark. Minor modifications to the analysis of the section of the dam which bridges the old river channel resulted in a slight change in the static and dynamic overturning factors. The modifications were as follows:

a) Use of the full base width for uplift on the East portion of the deep dam section.

b) The addition of the equivalent concrete pier load on the upstream face of the deep dam section (East and West).

c) Utilization of full tailwater pressure for the East portion of the deep dam section.

2) RESULTS

The results of this analysis, assuming 100 percent uplift are as follows:

	Typical Dam Section (base el 171.13)		Dam Block Section in Deep Original River Channel	
	Static	Dynamic	Static	Dynamic
Overturning Factor	1.32	1.01	1.44	1.14
Shear Friction Factor	12.44	8.48	9.50	6.56
Toe Compression (no base Tension)	5.19 k/ft^2	67.29 k/ft^2	8.20 k/ft^2	22.3 k/ft^2

3) CONCLUSIONS

It is our belief that the Vernon Dam is a Class I structure in that it can withstand, without gross failure, the maximum hypothetical earthquake selected for this site.

The analysis indicates that the critical block section is that with a base elevation of 171.13 (USGS) and that the deeper block section filling the original river channel, will act as a plug with a substantially higher safety factor.

All block sections investigated remain stable under all of the loading conditions.

B) DISCUSSION

1) DESCRIPTION

Vernon Dam is located on the Connecticut River at Vernon, Vermont. It is a gravity type dam of concrete construction with a crest length of 956 feet and a reservoir with a gross capacity of approximately 40,000 acre-feet. It is primarily used for hydro-electric generation and has an installed capacity of approximately 25 mw. Construction of the dam was completed in 1909.

Normal head-water level for the dam is at the top of the flashboards at elevation 220.13 (USGS). Minimum tailwater level is at elevation 178.13 (USGS). During flood conditions, the dam is submerged and stability increases.

2) DEVELOPMENT OF COMPUTATIONS

a) General Conditions and Assumptions

The details of the sections of the dam which were analyzed were ob-obtained from the New England Power Company which owns and operates the Vernon Hydroelectric facility. For the analysis, headwater was taken at normal water level at the top of the flashboards (El 220.13 USGS) and tail-water, as determined from the tailrace rating curve, was taken at a mini-mum (no flow) elevation (El 178.13 USGS).

Uplift was investigated using a gradient extending from tailwater to headwater. It is believed that the actual effect of the uplift would be best approximated by use of a 2/3 factor on the base area. However, as an additional factor of safety, uplift was calculated and used in the compu-tations as affecting 100 percent of the base area.

Earthquake factors of 0.14g horizontal and 0.093g vertical, acting simul-taneously, were used in the dynamic analysis in each case. These factors are based on the maximum earthquake ever expected at this site. Earthquake for-ces were considered to affect the headwater above El 171.13 and the dam it-self. Dynamic headwater force was computed using "Bureau of Reclamation Engineering Monograph No. 19" with a pressure coefficient (c) equal to 0.735 and assuming a parabolic curve of increased water pressure. Because of the transitory nature of the earthquake forces, the uplift and water pressure between the dam and the backfill material were not affected. Also, earth-quake effect was not added to the tailwater as it would improve stability.

b) <u>Specific Conditions and Assumptions</u>

1) Typical Dam Section (Base El 171.13) - (See Sketch 1)

The static headwater force against the dam was computed between normal water level (El 220.13) and base elevation (El 171.13). Tail-water calculations were computed between minimum tailwater (El 178.13) and base elevation (El 171.13).

The Shear Friction Factor was determined using "Bureau of Reclamation Engineering Monograph No. 19" and assuming a coefficient of internal friction (tan Ø) of 0.7 and a cohesion (c) of .15 k/in.^{2} which is half the minimum shearing strength normally used by the Bureau of Reclamation. Compression tests of rock cores from the Vernon Nuclear site have shown a minimum strength of 15,700 psi. The rock at the dam is geologically similar to these cores and the shear values assumed are therefore extremely conservative.

2) *Deep Dam Section*

Since the dam cross-section varies through this deeper portion, it was decided to analyze a section at each end of the block which crosses this zone and to average the results to obtain the actual overturning safety factor for the block in question. The length of this block is approximately 55 feet. The original river channel crosses the dam axis on a skew in this area and therefore provides an excellent key. The passive resistance of the material adjacent to the dam (as shown in the attached sketches) was neglected. Including the effect of this material would improve the safety factor.

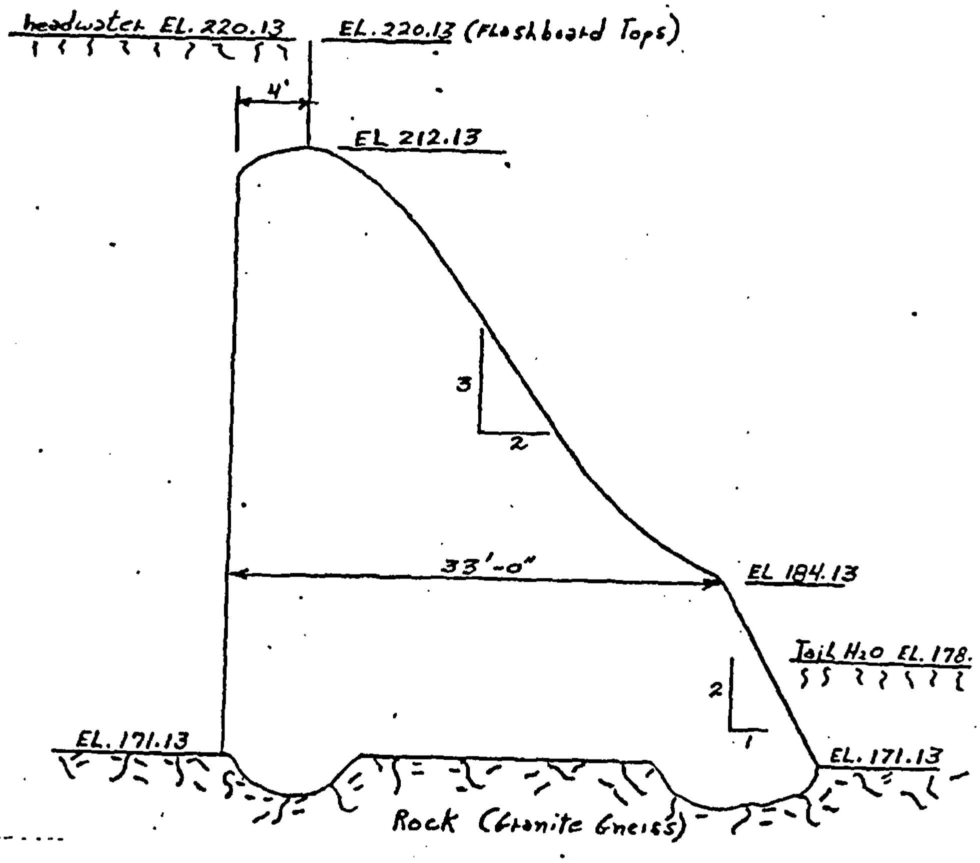
a) East-End Section - (See Sketch 2)

The static headwater force against the dam was computed between normal water level (El 220.13) and El 153.13 which is the minimum elevation at the heel of the dam. Tailwater forces against the dam were computed between minimum tailwater (El 178.13) and El 146.13. This same depth of 32 feet was used in determining uplift.

b) West-End Section - (See Sketch 3)

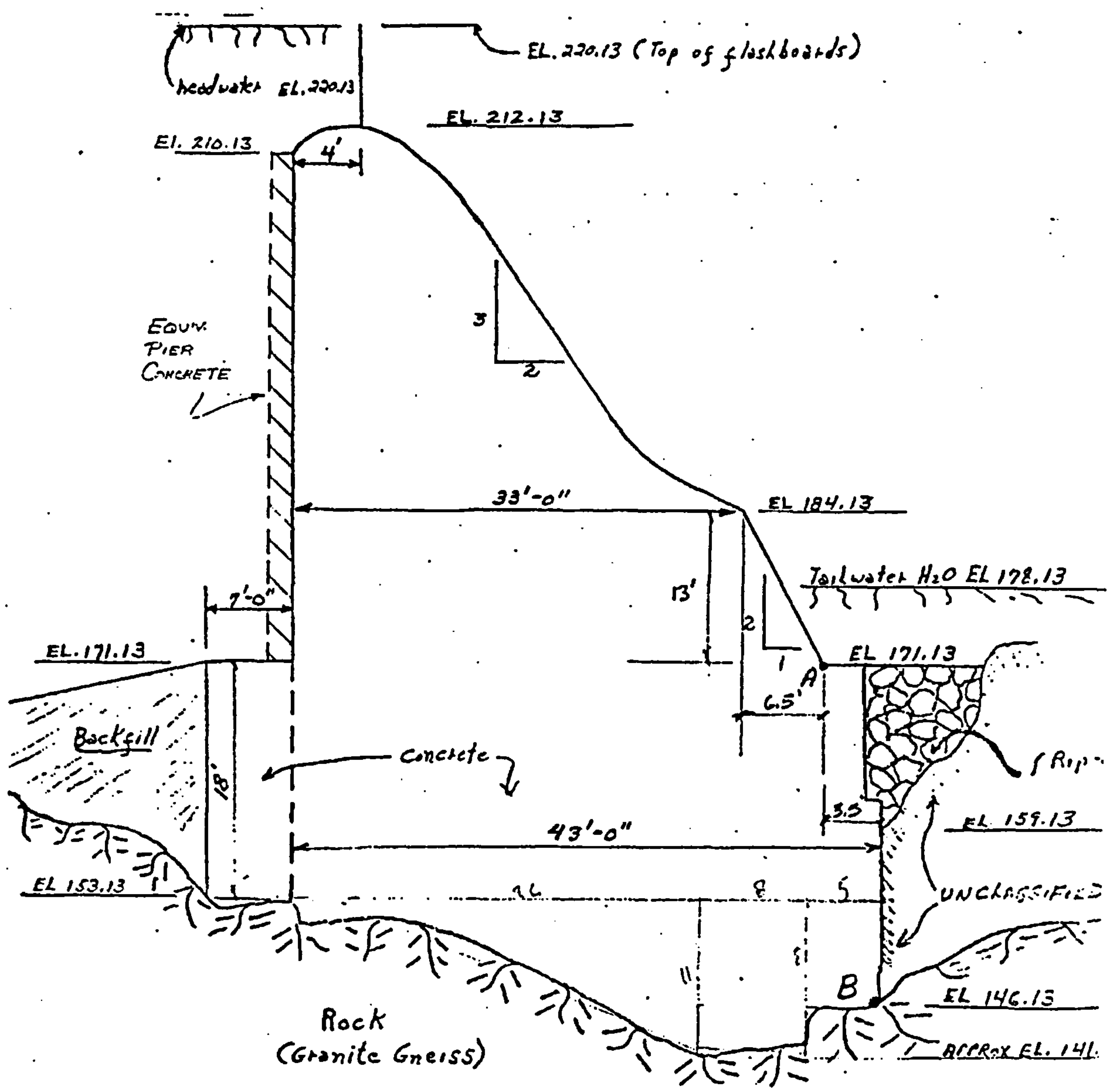
The static headwater force against the dam was computed between normal water level (El 220.13) and El 149.13 which is the minimum elevation at the heel of the dam. Tailwater forces against the dam were computed between minimum tailwater (El 178.13) and El 166.13. Uplift was computed using this 12 foot height of tailwater.

I.6


headwater EL. 220.13
EL. 220.13 (Flashboard Tops)
4'
EL 212.13
3
2
33'-0"
EL 184.13
Tail H2O EL. 178.
2
1
EL. 171.13
EL. 171.13
Rock (Granite Gneiss)

SKETCH 1

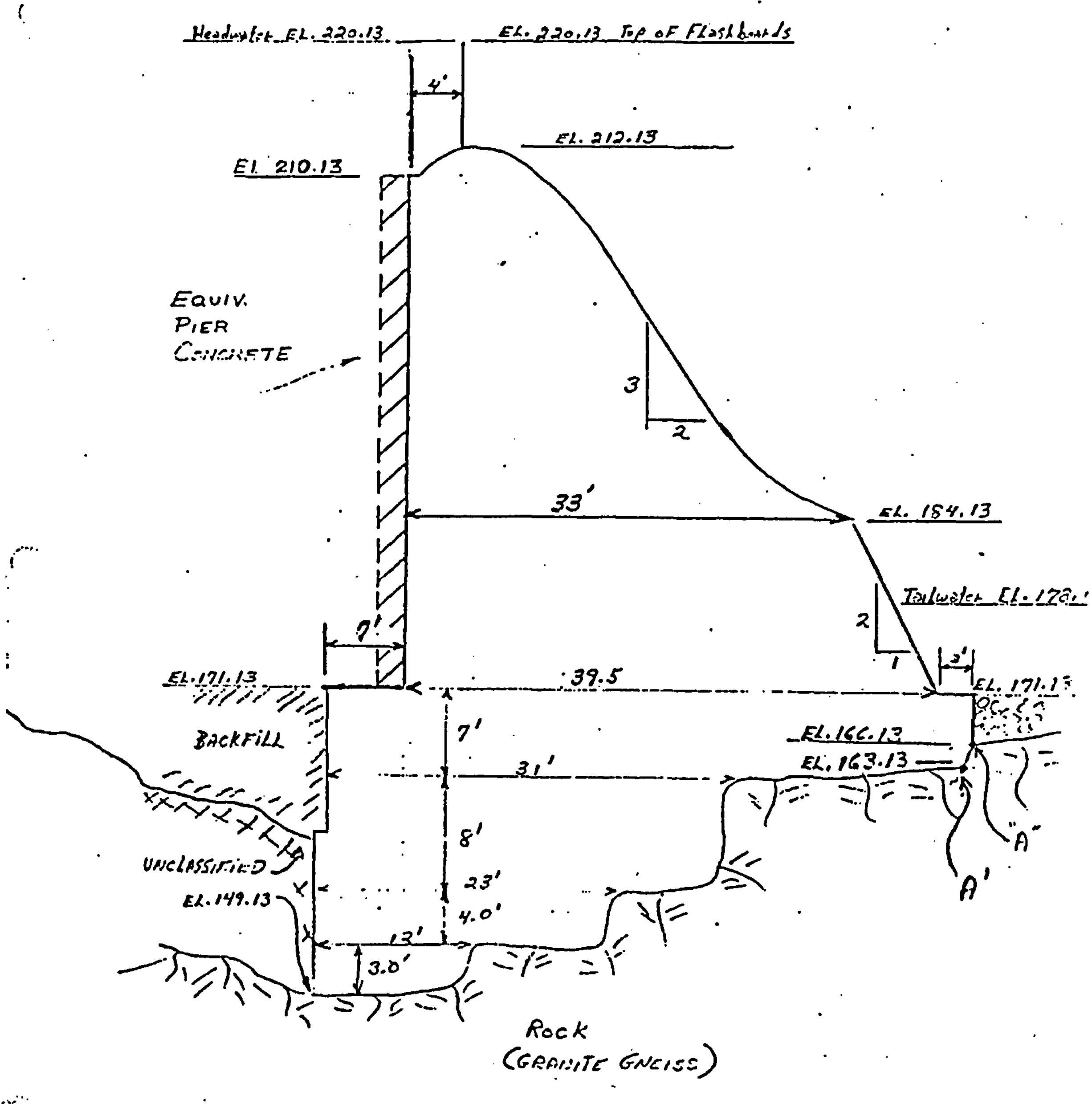
I.7



(NTS)

SKETCH 2

I.8


Headwater EL. 220.13
EL. 220.13 Top of Flashboards
4'
EL. 212.13
El. 210.13
EQUIV. PIER CONCRETE
3
2
33'
EL. 184.13
Tailwater EL. 178.1
2
1
7'
3'
EL. 171.13
39.5
EL. 171.13
BACKFILL
7'
EL. 166.13
EL. 163.13
31'
8'
"A"
UNCLASSIFIED
A'
EL. 149.13
23'
4.0'
13'
3.0'
Rock
(GRANITE GNEISS)

SKETCH 3

II REACTOR BUILDING CRACK ANALYSIS UNDER MAXIMUM POSTULATED EARTHQUAKE

The design earthquake at the Vermont Yankee site has a ground acceleration of 0.07g. An analysis has been performed to determine the extent of cracking of the concrete walls of the reactor building which would result from a seismic disturbance with a ground acceleration twice that of the design earthquake or 0.14g.

With a ground acceleration of 0.14g, it is expected that diagonal cracks will develop in the lower exterior concrete walls of the reactor building. The estimated total length of these cracks is 1300 linear feet. It is assumed that 50% of the cracks would close after the earthquake transient. It has also been conservatively assumed that the average cracks width of those cracks which remain open is 20 mills. With 650 linear feet of cracks with a 20 mill opening and a negative pressure of 0.25 inches of water within the reactor building, the inflow leakage rate would be 50 cfm.

Since the standby gas treatment system has a capacity of 1500 cfm, the estimated inflow leakage rate is not considered significant.

The leakage computations used in this analysis were based on AEC research and development report NAA-SR-10100, "CONVENTIONAL BUILDING FOR REACTOR CONTAINMENT" and the techniques employed are the same as those applied to previous analysis of BWR reactor buildings.

VYNPS

8.0-1
8.1-1

STATEMENT

8.0 The following information is required to assess the adequacy of the site and building design criteria.

QUESTION

8.1 Please state the sea level elevation of the station service water intake. If this is higher than the Connecticut River low-flow elevation at this point without the Vernon Dam, please provide the justification.

ANSWER

The station service water intake has a deck El. 237.0 ft MSL (Mean Sea Level - USGS datum) and the bottom of the intake is at El. 190.0 ft MSL. The elevation of the Connecticut River at this point is controlled by the Vernon Dam located approximately 2500 feet downstream from the site.

By coordinating the operation of the Vernon Hydroelectric Station, adjacent to the dam, with the upstream hydroelectric stations, the Vernon pond water level is normally maintained between El. 218 ft MSL and El. 220 ft MSL. The crest of the dam is at El. 212 ft MSL while the flashboard is El. 220 ft MSL. Flashboard replacement requires that the pond level be temporarily reduced to El. 211.5 ft MSL; however, this operation is performed very infrequently, usually once a year. In one instance, the pond level has been lowered to El. 210 ft MSL for gate repairs. Even at this level the Vermont Yankee service water pump suction would remain submerged.

With all units in operation at the Vernon Hydroelectric Station, under maximum head conditions and with no discharge from the upstream Bellows Falls Hydroelectric Station, the Vernon Pond drawdown rate would be 1.3 ft/hr. Administrative controls exercised by the New England Power Company in the operation of the Vernon Hydroelectric Station, however, limit the drawdown rate to 0.3 ft/hr. If this drawdown rate were sustained for as long as 60 hours (assuming the pond to be at El. 218 ft MSL initially and with no inflow) the station service water pump suction would still be submerged. Operating in this mode for such a long period of time is not expected.

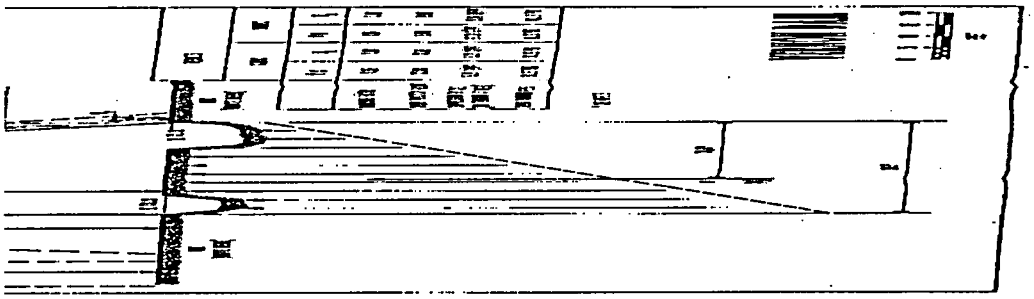
The Vernon Hydroelectric Station's standard operating practices will result in pond levels that ensure submergence of the Vermont Yankee service water pump suction. In addition, the Vermont Yankee Nuclear Power Station operators will be able to communicate directly with the Vernon Hydroelectric Station operators and thereby reduce the likelihood of inadvertent reductions in pond level below prescribed limits.

In order to provide submergence for the service water pumps it is necessary to assure a minimum river level of approximately El. 200.0 ft MSL. This minimum level can be assured from an operations standpoint. In addition, to be consistent with the design criteria for the structures and equipment required for a safe shutdown of the Vermont Yankee Nuclear Power Station, the Vernon Dam should be analyzed for the maximum 0.14g earthquake. Ultimate stability is all that is required under this maximum earthquake, the same as is required for the Class 1 structures and equipment at the Vermont Yankee Nuclear Power Station.

The Vernon Dam is a gravity, concrete overflow type dam which was constructed in 1907. This dam is approximately 600 feet long and about 41 feet high except for a narrow section which bridges the old river channel. This deeper section is limited in extent, approximately 80 feet long, and reaches a maximum depth of some 65 feet. A powerhouse approximately 320 feet long is constructed directly to the west of the overflow section of the dam. The dam and powerhouse are founded on compact rock which is believed to be a granite gneiss, as determined from the borings taken at the Vermont Yankee site, approximately 2500 feet from the dam.

In the analysis of the dam for the maximum earthquake the headwater was taken at El. 220 ft MSL and a minimum tailwater El. 178 ft MSL was used. Uplift on the base of the dam was assumed as varying from full headwater at the head of the dam to full tailwater at the toe of the dam acting on 2/3 of the base area. Calculations were also made to determine the stability with uplift acting on 100% of the base area. The attached sketch, entitled Vernon Dam, shows the dam and loadings used for the analysis. Results of the analysis are also shown in Figure 8.1-1. The overturn factor is equal to the summation of the horizontal earthquake and headwater moment plus the uplift moment divided into the summation of the dead weight moment of the dam, the tailwater moment and the moment due to the water on the dam, all moments taken about the toe of the dam. The shear friction factor is equal to the summation of the horizontal forces divided into the sum of an allowable shear stress of 150 psi times the base area plus the summation of the vertical forces times a coefficient of friction equal to 0.70. The compression on the toe of the dam was computed assuming no tension at the heel. If tension at the heel were to exist it would reduce the computed values.

The results show the dam to be stable under the maximum earthquake loading.



QUESTION

8.2 In Appendix H, dealing with the seismic design criteria, it is recommended that the earthquake spectra corresponding to the N69°W component of the 1952 Taft earthquake normalized to 0.07 g be used for design. Please justify the selection of this particular earthquake spectrum as being characteristic of this site.

ANSWER

There are a number of past earthquake records which are available for use in design. However, only three of these earthquake records are normally used in the analysis and design of structures for earthquake motions. These three earthquakes are as follows:

1) 1957 Golden Gate Park earthquake, S80°E component.
2) 1940 El Centro earthquake north-south component.
3) 1952 Taft earthquake N69°W component.

The Golden Gate earthquake was recorded on competent rock, the El Centro earthquake on deep alluvium, and the Taft earthquake on shallower and firmer alluvium.

The Golden Gate earthquake recorded on rock would appear to be better suited geologically to the Vernon site. However this spectra peaks very sharply at periods of 0.1 to 0.25 seconds and drops off very rapidly for longer periods. By comparison, the Taft earthquake peaks at about 0.2 to 0.5 seconds which is within the range of the estimated period of the reactor building. The Taft earthquake is, therefore, a more severe test for the structure and was selected as the design basis earthquake.

The El Centro earthquake is not typical from a geologic standpoint and in addition is not as severe a spectra for periods under 0.5 seconds.

VYNPS

8.3-1

QUESTION

8.3 A table of damping values is presented on Page XII-2-7, and it is noted therein that reinforced concrete structures are to be designed for 5 percent of critical damping. For this particular plant, which is founded on rock, justification for 5 percent damping is necessary, especially for the design earthquake situation in which the entire structure is assumed to remain elastic. Higher levels of damping are permissible when cracking occurs and, in general, are a function of the stress and deformation level resulting from the loading. Are the same damping values to be employed for both the design earthquake and maximum earthquake conditions?

ANSWER

The subject building is a massive reinforced concrete construction, 144 feet by 144 feet in plan up to elevation 343 feet where the width decreases to about 110 feet. The outside concrete walls and concrete floors comprise the secondary containment (the primary containment is effected by the steel containment vessel). The building period is approximately 0.3 seconds.

The damping value assigned for this massive concrete structure was five percent of criteria for both the design and maximum earthquakes. Using the assigned damping value, shear stresses in the resisting walls of the building are estimated to be approximately 80 pounds per square inch. These values are without consideration of wall reinforcement.

With regard to the assignment of a realistic damping value, it is well known that damping values for concrete structures will vary with stress level, but it must be pointed out that the stress level in the subject reactor building is by no means low when compared to stress levels used in actual damping tests of structures. Furthermore, the subject building has many cross walls, and is filled with equipment, water, fuel elements, etc. - a condition which is far from that represented by a base frame building having low damping characteristics. To our knowledge, no authority has assigned a damping value of less than 5 percent critical damping to a building such as the proposed Vermont Yankee reactor building. Nuclear Reactor and Earthquakes (TID 4500) of the United States Atomic Energy Commission, assigns a damping value of 7 percent for such concrete structures. Of the many earthquake design criteria for nuclear power plants written by other persons knowledgeable in earthquake engineering, none has assigned a value of less than 7.5 percent for similar concrete structures. One very similar to the building in question was assigned 10 percent. In addition, interaction with the foundation material must be considered in the building analysis. This interaction will increase effective damping.

Many recent testing programs have attempted to measure damping in buildings. None of these tests have been performed using stress levels as great as those estimated for the Vermont Yankee reactor building under design earthquake conditions. Two of these programs, however, are of particular interest. Professors Bouwkamp and Clough of the University of California are studying dynamic properties of buildings. Their work is not yet complete but for the concrete building tested, their paper in the 1965 Proceedings of the Structural Engineers Association of California states with reference to proper damping values to be used in the dynamic analysis of concrete structures: ". . . . although the experimental data have not been evaluated completely a damping of at least 5 percent can be safely assumed". They also point out that under larger deflections the damping will increase substantially. The deflections obtained in the test structure were considerably less than estimated for the subject reactor building under the design earthquake loading.

In his thesis entitled Dynamic Response of Multi-story Buildings, (California Institute of Technology), Dr. N. Norby Nielsen has determined certain damping values for a five-story concrete building. Longitudinally, the building is laterally supported by frames; in the transverse direction by shear walls. For the transverse direction a statistical study made of the basic data within the Nielsen report indicates a probable damping of 6.6 percent of stresses corresponding to those in the subject reactor building. While the extrapolation of such data may be large, our statistical studies indicates that the basic data conform to straight line representation - the correlation function is approximately 0.053 - and that there is a 95 percent probability that the damping corresponding to a stress of 80 psi will be equal to 5.7 percent $\pm$ 1.4%. (Refer to Figure 8.3-1.)

In other words, the damping will vary between 7.1 percent and 4.3 percent with a 95 percent probability. While it is true that this study extrapolates information well beyond the range of the basic data, the results confirm the apparent consensus of opinion that realistic damping values for concrete structures is in the range of 5 to 10 percent of critical for the stress levels estimated for the Vermont Yankee reactor building under design earthquake conditions. While damping values in excess of 5 percent might be expected under the maximum earthquake condition it was decided to use 5 percent damping for both earthquakes.

If the concrete structure in question was needed for primary containment or a prestressed thin shell structure, a much lower damping value would be in order. The subject building is not of this type, and the assigned damping value of five percent appears to be realistic. In our research we have not found any test information indicating that lower damping values should be used for a building such as the subject building

Comparison of Vermont Yankee Spectrum with 1993 LLNL Median Uniform Hazard Spectra

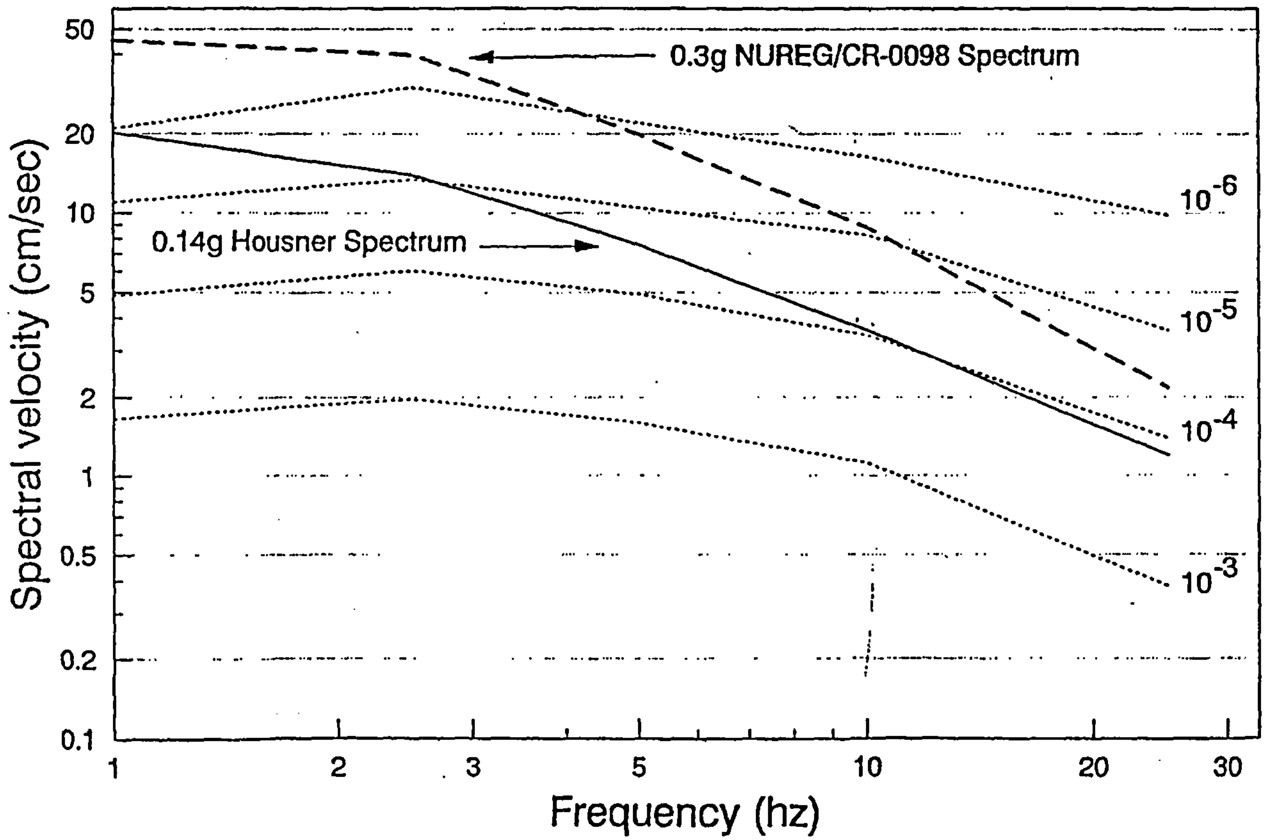


Figure 6

FIFTH QUINQUENNIAL SAFETY INSPECTION
VERNON PROJECT

FERC Project No. 1904

October 30 1987

Prepared for

New England Power Company

by

G E I
1021 Main Street
Winchester, MA 01890
(617) 721-4000

Project No. 87123

TABLE OF CONTENTS

LIST OF FIGURES

APPENDICES

I. SUMMARY

The Vernon Project is located on the Connecticut River in the Towns of Vernon, Vermont, and Hinsdale, New Hampshire. The licensed project consists of a 600-foot-long spillway and a powerhouse (Fig. 1). The left abutment is a long natural soil ridge called Vernon Neck. The project was constructed between 1907 and 1910. A powerhouse addition was constructed between 1918 and 1921.

Previous Federal Energy Regulatory Commission (FERC) quinquennial safety inspections for this project performed in accordance with FERC Order No. 315 were dated November 1967, November 1972 and November 1977. The 1982 quinquennial inspection was conducted in accordance with FERC Order 122.

Findings of the fifth FERC Safety Inspection of the project required at five year intervals are presented. The inspection was performed in accordance with Part 12 of FERC Order No. 122 effective March 1, 1981 and FERC letter dated May 15, 1987, Appendix H. There have been no federal, state or independent consultant reports relating to safety of project structures since the last quinquennial safety inspection report.

The project structures are founded on hard massive gneiss. There are no adversely oriented bedding planes or joints observed at the site and there are no known active faults in the area of the project.

Project instrumentation consists of an extensive powerhouse crack monitoring program. There is no indication of changes or trends other than seasonal (thermal) cyclic variations in the crack dimensions. In our opinion, this program can be terminated; however, the gages should be maintained and read after major floods, felt earthquakes and/or the next quinquennial safety inspection.

A survey of four cross sections of Vernon Neck is conducted at five year intervals to detect upstream/downstream changes in cross section. No changes indicating significant reduction in cross section have been detected to date. This program should continue at five year intervals or after major floods.

In general, the project structures are in good condition and well maintained. The powerhouse superstructure was in good condition and all mechanical equipment is well maintained and serviceable. The project spillway structure and powerhouse intake have been extensively modified since the last inspection to improve spillway crest control, obtain access to Vernon Neck and to improve trash rack cleaning procedures.

The project spillway can pass up to 51 percent of the Probable Maximum Flood (PMF) at zero freeboard. The flood of record is 185,000 cfs or 32 percent of the PMF in March 1936. The estimated PMF is 567,100 cfs. At PMF, significant damage to project structures would result due to overtopping flows.

Stability analyses show the project powerhouse structure to be stable for all loading conditions including normal operating reservoir, ice loading, zero freeboard (References 2 and 4). The studies in References 2 and 4 were extended in this inspection report to include 0.10g earthquake loading, and analysis of the modified spillway structures. It is concluded that all structures are stable for the loading conditions investigated. At PMF, the spillway structures become submerged weirs and the powerhouse will be heavily damaged.

Based on the information available from prior inspection reports and the observations made during this inspection, there are no recommendations for emergency remedial action or additional monitoring programs at this project.

It is recommended that rock scour downstream of the deep tainter gate be evaluated and stabilized. It is also recommended that the heavy tree and brush growth on the Vernon Neck be selectively cleared.

II. DESCRIPTION OF PROJECT

A. General

The Vernon Project was constructed by the Connecticut River Power Company and is presently owned and operated by the New England Power Company (NEP). Construction began in 1907 and was completed in 1910. The power plant was put into commercial operation on December 1, 1909. In 1910, the final three of the eight original generating units were placed in operation. An addition to the generating station and the installation of two additional generating units was commenced in 1918 and completed in 1921. These units were put into commercial operation on March 12, 1921.

Effective date of FERC License is June 1, 1979. The date of expiration of the license is April 30, 2018.

The project is located on the Connecticut River in the towns of Vernon, Vermont, and Hinsdale, New Hampshire (Fig. 1). The project structures include a gravity concrete spillway section equipped with flashboards, radial gates, and sluice gates, and a non-overflow section comprised of a trash sluice and the head works and powerhouse.

B. Project Data

The following project data are taken from References 1 through 4. The gross drainage area above the project is approximately 6266 square miles. The reservoir extends upstream for approximately 30 miles above the project and has a surface area of 2550 acres at El. 220.13 NGVD (126.0). For reference, elevations are given as NGVD with equivalent project datum in parentheses. Project datum is 94.13 feet above NGVD (126.0 project Datum = 220.13 NGVD).

Other statistics are as follows:

Normal Maximum Reservoir Elevation		220.13 feet (126.00)
Normal Operating Reservoir Elevation		218.00 feet (123.87)
Normal Tailwater Elevation		184.80 feet (90.67)
Usable Storage (8 ft. drawdown)		18,300 ac. ft.
Spillway - Length - clear		542.50 feet
Crest Elevation	- 10 x 50 gates (4)	212.13 feet (118.00)
	- 10 x 10 panels (10), flashboards 3 (bays)	212.13 feet (118.00)
	- 20 x 50 gates (2)	202.13 feet (108.00)
Discharge Capacity	- W.S. El. 220.13	83,200 cfs
	- W.S. El. 228.13	127,600 cfs

C. Powerhouse

The project powerhouse contains ten generating units consisting of eight units rated at 2000 kw and two units rated at 4200 kw. The installed capacity is 24,400 kw. The powerhouse has an integral intake structure with intake gates, trash rack and trash rake. An upstream trash boom protects the structures against floating debris, Figs. 2, 3, 4 and 5.

D. Trash Sluice

A trash sluice abuts the east (left) side of the powerhouse and is controlled by a motor-driven drop gate, Fig. 6.

E. Spillway

The project spillway is 600 feet long. During the period September 1985 through November 1986, the spillway was modified to install operable gates and an access bridge. The modified spillway consists of the following from right (west) to left (east):

Type	Number	Height (ft)	Width (ft)
Tainter Gate	4	10.0	50.0
Hydraulic Steel Flashboard Panels	10	10.0	10.0
Pin Flashboards	2	8.0	50.0
Pin Flashboards	1	8.0	42.5
Tainter Gate	2	20.0	50.0
Sluice Gates	8	9.0	7.0

F. Vernon Neck

The Vernon Project is located on a bend of the Connecticut River. Vernon Neck is a natural soil ridge that extends approximately 1/2 mile to the left (east) of the project spillway and is considered part of the water retaining structures for the project, Fig. 1 and Appendix B.

G. Standard Operational Procedures

The Vernon Project is operated as a run-of-river hydroelectric project. Flows in excess of generation requirements are released by operation of the project spillway crest control structures.

III. CONSTRUCTION HISTORY

The Vernon Project was constructed between 1907 and 1910 with commercial operation beginning on December 1, 1909. Two additional generation units were installed between 1918 and 1921 with commercial operation on March 12, 1921.

With the exception of regular maintenance, no significant changes were made to the project until the addition of a fish ladder between May 1979 and May 1981.

In September 1985, a major construction program was initiated to add crest gates and hydraulic steel panels to the project spillway, to construct an access bridge across the spillway, and to improve the powerhouse intake structure by addition of an access bridge and hydraulic trash rake. These facilities were effectively completed November 1986.

IV. GEOLOGY

The Vernon Dam is located on the Connecticut River between the states of Vermont and New Hampshire. The bedrock of this region consists of folded sediments and metasediments of the Silurian and Devonian Periods. Older sediments and metasediments are exposed along intermittent stretches of the Connecticut River. In general, the older sediments and metasediments are of the Ordovician Period. Metavolcanic rocks of Ordovician Age and igneous intrusive rocks of the Paleozoic Era also outcrop along the Connecticut River.

West of the Connecticut River, in Central Vermont, the bedrock is generally older. It consists of folded and faulted sediments, metasediments and igneous rocks of the lower Paleozoic Cambrian and Ordovician Periods.

In the vicinity of Vernon Dam, the bedrock consists of the lower Devonian intrusive igneous gneiss of the Oliverian plutonic series. This rock type forms the geologic structure referred to as the Vernon Dome. The bedrock is hard and massive with no adversely oriented weak planes or joints evident, Photos 7, 8 and 9.

There are no known active faults near the project site.

V. MONITORING PROGRAMS

A. Powerhouse

Numerous cracks in the project powerhouse are monitored for activity by use of trammel points and paper tapes. New Avongard Calibrated Crack Monitoring gages were installed in 1980. The Avongard Gage is a direct reading biaxial graphic movement monitoring device. The two sections of the device are mounted on opposite sides of a crack and the crosshair on one element is aligned with the grid on the other element. Changes in position over time can be interpreted to the nearest 0.1 millimeter. To date, no significant changes or trends are discernible in the trammel points, paper tapes or Avongard gages other than seasonal (thermal) cyclic variations, Appendix B.

All crack monitoring devices were read following the 1982 New Brunswick and the Laconia, NH, earthquakes. No detectable changes in crack widths were observed due to these earthquakes.

It was observed in Reference 4 that some of the powerhouse cracks in the downstream right corner were likely associated with the March 1936 flood which reached a headwater elevation of 231.4 (137.3), or about 5 feet above the generator deck.

B. Vernon Neck

At periodic intervals, NEP conducts cross-section surveys at four locations on Vernon Neck. The most recent surveys were conducted in July 15-17, 1987, Appendix B. When superimposed on surveys taken since 1924, no significant changes could be detected in the main cross section of the neck. Some continuing minor changes at the downstream toe caused by seasonal river erosion and deposition during flood flows is considered insignificant since the toe is protected by riprap. The next survey of Vernon Neck should be conducted as part of the next quinquennial safety inspection or following a major flood ($Q \geq 150,000$ cfs).

C. Adequacy

The current program of instrumentation and monitoring of project structures is adequate, and no new or supplemental programs are required. The original data are on file at the project office.

It is our opinion the program of crack monitoring in the project powerhouse may be terminated. The gages should be maintained so they can be read following major floods, felt earthquakes, and/or at the next quinquennial safety inspection.

VI. FIELD INSPECTION

A. General

The project structures were inspected on May 21, 1987, by Messrs. Alton P. Davis, Jr., and Marvin Davidson of Geotechnical Engineers Inc. accompanied by Messrs. Denton E. Nichols of New England Power Service Company (NEPSCo), and Hugh W. Sullivan, Charles M. Harrington, and George Webster of New England Power Company (NEP). The water surface elevations at the time of the inspection were approximately as follows:

Headwater Elevation	218.2 (125.1)
Tailwater Elevation	183.2 (90.1)

An inspection checklist is included as Appendix C and inspection photographs are included as Appendix D.

In general, the various project features contain many detailed points of interest and significance relating to their current condition, such as cracks, seepage and spalling. In previous inspection reports (References 1 through 4), these conditions have been discussed in detail, and to avoid repetition, only changes, or previously unreported conditions, will be high-lighted in the following subsections.

B. Powerhouse

The powerhouse superstructure (Figures 3 through 6 and Photo No. 1) is in generally good condition with numerous cracks in the lower brickwork. The elevation 226.88 (132.75) generator floor has numerous random cracks (Photo No. 2). All accessible areas of the substructure were inspected and found to be in generally good condition.

The unit wheel pits were in generally good condition. The wheel pits appear unchanged from conditions noted in the 1982 Inspection Report (Reference 4).

The elevation 189.13 (95.0) walkway over the draft tubes was inspected. To the left of Unit No. 1 draft tube, the downstream pier nose is eroded (Photo No. 10). On the wall downstream of Unit Nos. 7 and 8, the concrete is heavily spalled with pattern cracking. These conditions have not changed significantly since noted in the 1967 inspection report (References 1 through 4). Seepage discharge from the exciter unit area observed in 1982 (Reference 4) has been corrected.

The right abutment upstream training wall and earth backfill are in good condition.

The powerhouse intake structure has been modified since the last inspection to include installation of an access bridge and a new trash rake (Photo No. 1). This is a major improvement for maintenance of the trash racks.

C. <u>Trash Sluice</u>

The elevation 226.38 (132.25) deck and piers are in good condition. The ogee is generally in good condition with light to moderate surface spalling. The concrete face under the stairway to the draft tube access walkway is spalled with heavy pattern cracking and is drummy. This condition is generally unchanged from the 1982 inspection (Reference 4). The trash sluice drop gate was undergoing repair at the time of the inspection.

D. <u>Spillway</u>

The spillway inspection tunnel and sluice gate operator gallery were inspected. The two easterly gates (9 and 10) have been plugged. NEP mobilized in June 1982 and rehabilitated the remaining 8 sluices including installation of new gate seals. All eight sluice gates were overhauled and completed in 1983.

At the west (right) end of the gallery is a room with storage tanks containing hydraulic fluid for operation of the sluice gates. The west wall of the room has a heavy calcite buildup. The flow coming from rock outcrops in the access stairwell from the powerhouse, noted in the 1982 inspection report (Reference 4), has been effectively sealed by a program of chemical grouting conducted by NEP and the flow is significantly diminished. Along the downstream crown of the gallery, a lift line makes slight seepage with heavy calcite buildup. There is a 1/8-inch wide more-or-less continuous crack along the downstream gallery wall. Much of the crack is calcified. These conditions remain as reported since 1967 (References 1 through 4).

Between September 1985 and November 1986, NEPCo installed a major modification to the project spillway. Prior to this time, the crest was controlled by 8-foot-high pin supported timber flashboards. At the time of the inspection, maintenance was being conducted on several of the hydraulic flashboard operators (Photo No. 6). In Photo No. 6, the pin supported flashboards are installed as a cofferdam and the steel flashboards supported by struts to the access bridge during the maintenance period.

NEP reports that all gates were operated during the 1987 flood season. Gates were operated during the 1986 flood season, as reported in Appendix F.

Some plucking of rock and/or dental concrete downstream of the high head tainter gates was observed (Photo No. 9). This should be investigated and additional dental concrete or a concrete apron should be installed to protect the rock in this area.

An emergency gasoline driven generator provides power for the No. 1 and No. 2, 20 x 50 tainter gates.

Standby power for the spillway gates is any one of the ten project generators, which are capable of black start.

E. Fish Ladder

The Vernon fish ladder was placed in service in 1981 and has operated seasonally since that time.

F. Vernon Neck

Vernon Neck is a natural soil ridge of high ground between the reservoir and the downstream river channel and forms the left abutment of the spillway (Fig. 1). The area is inspected regularly by NEP personnel and no significant changes have occurred to date (Photo No. 4). The upstream and downstream slopes are heavily overgrown and need to be selectively cleared.

G. Emergency Action Plan

The Emergency Action Plan (EAP) was posted in the control room and the plant personnel receive an annual EAP training program. The plan was updated in September 1987 and tested in December 1986.

The Vermont Yankee Nuclear Station is sited 1/2-mile upstream of the Vernon powerhouse. The EAP has provision for evacuation in case of a declared radiological emergency condition at the Vermont Yankee Nuclear Station.

H. Miscellaneous

The reservoir shoreline as seen from the powerhouse has no obvious areas of distress or instability. There are no significant changes in the river channel downstream of the dam.

There are no new developments in the reservoir flood zone or the downstream channel since the last inspection report.

There have been no state, federal or independent inspection reports since the 1982 FERC inspection.

VII. SPILLWAY ADEQUACY

The U.S.G.S. stream gaging station at Vernon Dam with a drainage area of 6266 square miles was used to obtain the hydrologic characteristics of the catchment. The following information is reproduced from Reference 3.

A study of flow records which extend back to 1944 was made, and the major non-snowmelt flood events were selected and analyzed for use in deriving the unit hydrograph. The flood of October 1959 was selected to compute the six hour unit hydrograph for the basin at Vernon.

At the Vernon Project, the Probable Maximum Flood (PMF) with a peak inflow of 567,100 cfs would overtop the dam and appurtenant structures in attaining a maximum upstream water surface elevation of approximately 251.0 (156.87) or 39 feet above the spillway crest. A flood of this magnitude would likely destroy the generating capability of the Project and heavily damage the tainter gates of the spillway structures and the powerhouse superstructure. The water would be 25 feet above the generator deck.

Reference 3 concluded that failure of any portion of the impounding structures which might release the total reservoir volume would add less than one percent to the volume produced by the PMF.

The zero freeboard flood, elevation 237, (142.9) with a return period greater than 1,000 years, would reach a peak inflow estimated at 287,000 cfs, Appendix E. The flood of record at the site occurred in March 1936 with a peak inflow of 185,000 cfs, causing an upstream water surface elevation of 231.4 (137.3). Note that all preceding elevations are based on the original spillway configuration which has now been modified.

In Appendix E, a spillway rating curve is presented for the modified spillway. This rating curve assumes that all new tainter gates and hydraulic flashboards will be used as priority gates to delay tripping of the pin supported flash-boards as long as possible. It further assumes that once the head pond reaches elevation 240, the powerhouse superstructure will be effectively lost and discharge will occur through the powerhouse complex as a broad crested weir at approximately elevation 226.

At the PMF, Vernon Neck would have approximately 7 feet of freeboard upstream and up to 10 feet head differential from headwater to tailwater in the downstream river channel. The minimum width of Vernon Neck at elevation 251 is approximately 100 feet.

FIGURES

VIII. STRUCTURAL STABILITY

A. Visual Observation

As noted in Section VI, the project structures are in good condition with no significant deterioration or structural distress observed.

B. Analysis

Stability analyses of all project water retaining structures were presented in the 1982 Inspection Report (Reference 4). Since that report, major modifications to the spillway structures have been conducted to include addition of tainter gates, hydraulic flashboards and an access bridge. Further, the design earthquake coefficient for the project area has increased from 0.05g to 0.10g.

The powerhouse stability analysis in the 1982 Inspection Report has been revised to incorporate the new earthquake coefficient and the results are summarized in Appendix G.

Stability analyses of the modified spillway structures have been conducted as part of this inspection and the results are presented in Appendix G. It is concluded that all project structures meet FERC criteria for stability.

Based on the studies presented in this section for the modified spillway, the previous recommendation to place back-fill concrete in the eroded toe areas downstream of the sluice gates is no longer considered critical. The area should be monitored after major floods (greater than 150,000 cfs). Remedial repairs need be made only if significant additional erosion is detected.

IX. MAINTENANCE AND OPERATION

Maintenance of the project facilities by the project staff is of a high level. The project is operated for run-of-the-river hydropower. Operational procedures are consistent with this goal. Operational procedures are modified during the salmon spawning season in support of the anadromous fish restoration program for the Connecticut River.

Operation, maintenance and surveillance of the project structures are considered satisfactory.

X. CONCLUSIONS

Based on this inspection, the results of stability analyses summarized herein, the results of monitoring programs and review of prior inspection reports, it is concluded that the project structures do not require any emergency remedial work at this time.

The spillway is adequate to pass approximately 51 percent of the PMF at zero freeboard. The flood of record in March 1936 equaled 32 percent of the PMF. The project spillway modification will add significantly to power generation by better controlling the reservoir water level to maintain maximum head.

The spillway, non-overflow and powerhouse structures are stable for all loading conditions investigated using procedures, formulations and criteria currently accepted by FERC. At PMF, the project structures will experience overtopping flows of up to 18 feet above the top of the spillway piers. Heavy damage to the project structures is likely at PMF.

Project instrumentation consists of numerous crack monitoring gages in the powerhouse. This program has shown no significant movements in crack widths to date except for seasonal (thermal) cycles. There were no detectable changes due to the 1982 New Brunswick or Laconia, New Hampshire, earthquakes. In our opinion, this program may be terminated; however, the gages should be retained to permit reading after high flood flows, felt earthquakes and/or during the next quinquennial safety inspection. No additional instrumentation is required at this time.

The Vernon Neck surveys show no significant changes in the cross-section of the neck. These surveys should continue on a five year basis and after any flood exceeding 150,000 cfs.

Project maintenance is very good. Surveillance and operational procedures are adequate. The Emergency Action Plan was posted in the control room and was updated in September 1987 and tested in December 1986. Plant personnel receive an annual EAP training program. The plan includes a Radiological Response plan for the Vernon Nuclear Plant one-half mile upstream. There are no changes in the downstream channel.

The spillway gates are operable and were used during the April 1987 flood. Standby power is provided by any of the 10 powerhouse generators. The eight sluice gates are operable. An emergency generator provides power for the 20 x 50 feet tainter gates.

The exposed bedrock downstream of the new deep tainter gate section shows evidence of erosion. This should be investigated starting with a survey in 1987 followed by a second survey after the 1988 flood season. Remedial action should be based on evaluation of any changes observed.

The toe erosion previously observed downstream of the sluice gate sections is of less concern now that the spillway has been modified and post-tensioned to the bedrock. This area should be monitored after major floods and at five year intervals to detect any significant changes that might warrant remedial work in the future.

All recommendations from prior inspection reports for the Vernon Project have been complied with by NEP.

XI. RECOMMENDATIONS

Based on the visual inspection reported herein and review of past inspection reports, we have the following recommendations for the Vernon Project:

- Evaluate the rock and dental concrete erosion downstream of the deep tainter gates and stabilize as required.
- Selectively clear the heavy tree and brush growth on Vernon neck to permit annual inspection of the upstream and downstream slopes.

The evaluation of dental concrete requirements downstream of the new deep tainter gates should begin with a survey in 1987 followed by a second survey after the 1988 flood season. Action should be based on evaluation of any changes observed. The Vernon Neck clearing should be completed in 1988.

XII. CERTIFICATION

The project structures were inspected on May 21, 1987, by Messrs. Alton P. Davis, Jr., and Marvin Davidson of Geotechnical Engineers Inc. accompanied by Mr. Denton E. Nichols of New England Power Service Company; and Messrs. Hugh W. Sullivan, Charles M. Harrington and George Webster of New England Power Company.

This report covers our inspection of the project carried out in accordance with Part 12 of FERC Order No. 122. The project inspection and preparation of this report was done under the direction of the undersigned. The assistance of project staff in conducting the inspection and assembling project data is gratefully acknowledged.

We certify that all work performed in connection with the inspection and investigation of this project and preparation of this report has been done in compliance with Part 12 of FERC Order No. 122 dated March 1, 1981. All conclusions and recommendations in this report have been made independently of the licensee, its employees, and its representatives as required by paragraph 12.37(c)(7) of that order.

THE STATE OF NEW HAMPSHIRE
ALTON P. DAVIS, JR.
CIVIL
No. 5421
LICENSED
PROFESSIONAL ENGINEER

STATE OF VERMONT
ALTON P. DAVIS, JR.
No. 4241
REGISTERED
PROFESSIONAL ENGINEER

Respectfully submitted,

GEOTECHNICAL ENGINEERS INC.

Alton P. Davis Jr., P.E.
Project Manager

Appendix F

Spillway Gate Operation Report

New England Power

New England Power Company
33 West Lebanon Road
P.O. Box 528
Lebanon, New Hampshire 0371

August 25, 1986

Mr. Martin Inwald, Regional Director
Federal Energy Regulatory Commission
New York Regional Office
26 Federal Plaza, Room 2207
New York, N.Y. 10278

RE: ANNUAL OPERATIONAL INSPECTION SPILLWAY GATE TEST AND MINIMUM FLOW REQUIREMENT FOR L.P. No. 1904-VT/NH, VERNON

Dear Mr. Inwald:

Thank you for your letter informing us of Mr. Estenio Rosell's scheduled inspecion on September 16, 1986. Our Mr. Charles M. Harrington, Superintendent of Hydro Maintenance has made arrangements to accompany Mr. Rosell during his insspection.

Records will be available at this location to Mr. Rosell which should adequately demonstrate our compliance with the requirements of our license concerning minimum flows at this location.

Since operation of spillway gates during this period may pose a problem, we have carefully checked our records and find that all spillway gates at this Project were operated within the preceding 12-month period.

At our Vernon Station, the power for a spillway gate operation is supplied by ten individual hydro-electric generating units operating together on any combination thereof. These units were the source of power during the test of the flood gates at this location. Since these ten generators routinely carry load on a daily basis for other purposes, no specific load tests are performed.

We trust this information will preclude any gate operation during Mr. Rosell's inspection; however, if Mr. Rosell should request the operation by performance in his presence, we stand ready to do so.

Very truly yours,

H. W. SULLIVAN, DIRECTOR
HYDRO PRODUCTION

HWS:tl
c/c - G. H. Webster
C. M. Harrington

A New England Electric System company

LIST OF FIGURES

APPENDIX G

STABILITY ANALYSIS

1. Values and Assumptions for Stability Analysis of Concrete Sections

A. Nomenclature:

Effective Length = uncracked portion of base

FH = Summation of Horizontal Forces -.kips

FV = Summation of Vertical Forces - kips (including uplift)

M_r = Summation of Resisting Moments - kip-ft

M_o = Summation of Overturning Moments - kip-ft

$\frac{M_r}{M_o}$ = Factor of Safety Against Overturning

$\frac{FH}{FV}$ = Coefficient of Sliding

B. Unit Weight of Concrete: 150 lbs/cu ft

C. Unit Weight of Water: 62.4 lbs/cu ft

D. Uplift Pressure:

The base pressure was assumed to vary linearly from full headwater pressure at the upstream side to full tailwater pressure at the downstream side taken over 100 percent of the base area for each case analyzed.

Uplift on any portion of the base or section not in compression is assumed to be 100 percent of the headwater pressure for any case with no foundation drainage systems.

Due to the transient or short-term nature of earthquake loading, the uplift is not changed from the pre-earthquake condition due to further propagation of a tensile crack.

E. Lateral Water Pressure:

Headwater pressures were computed using the full heights of water to headwater elevations over the projected height of the structures. Tailwater pressures are taken at full tailwater elevation for non-overflow structures. For overflow structures, tailwater back pressures are based on Figs. 14 through 18, Ven T. Chow Open Channel Hydraulics, 1959.

F. Ice Load: 5 kips per linear foot at normal water level.

G. Earthquake:

Accelerations of 0.10g were applied in a horizontal direction. To obtain the worst case, the resultant force action on the structure due to earthquake is taken in the downstream direction.

The hydrodynamic force was determined using a method presented in Design of Small Dams, USBR, pages 336-337.

H. Resistance to Sliding:

Where the ratio of FH/FV is greater than 0.65, the shearing resistance of the foundation to horizontal movement must be investigated using the Shear Friction Formula.

The factor of safety against sliding is determined by the Shear Friction Formula as:

$$S_{s-f} = \frac{f\ V + c\ A}{H}$$

where: f = coefficient of the angle of internal friction of foundation material (Tan Φ = 0.65)

V = summation of vertical forces

c = unit shearing strength at zero normal load on foundation material (0.192 ksi)

A = area of potential failure plane (area of base in compression)

H = summation of horizontal forces

Typical values of "f" and "c" were taken from "The Sliding Stability of Dams" by Harold Link in Water Power Magazine, March, April and May 1969.

The following factors of safety are generally required for the calculated stress and shear-friction factor of safety within the structure and at the rock-concrete interface, assuming a planar failure surface.

High or Significant Hazard Potential Dams

Usual Loading Combination	3.0
Unusual Loading Combination	2.0
Extreme Loading Combination	1.0

Low Hazard Potential Dams

Usual Loading Combination	2.0
Unusual Loading Combination	1.25
Extreme Loading Combination	1.0

Loading Conditions to be Investigated

a) Usual Loading Combination: Normal Operating Condition
b) Unusual Loading Combination: Flood Discharge Condition
c) Extreme Loading Combination: Normal Operating Condition with earthquake

The applied loads should include the appropriate concrete, water, earth, silt, ice, earthquake, and uplift forces applicable to the loading conditions being investigated.

I. Bearing Pressure:

Maximum bearing stress = 20 tsf on bedrock (278 psi)

J. Factor of Safety Against Overturning:

The minimum factor of safety against overturning is 1.0.

K. Strength of Vertical Connections:

For structures connected to adjacent structures via keyways, the maximum shear strength used across the key = 250 psi.

2. <u>Cases Used in Stability Analysis</u>

CASE I. Normal Operating Water Levels

H.W.L.	=	218.0	(123.9)
T.W.L.	=	184.8	(90.7)

CASE II Normal Operating Water Level with Earthquake

H.W.L.	=	218.0	(123.9)
T.W.L.	=	184.8	(90.7)

CASE III Normal Operating Water Level with Ice

H.W.L.	=	212.1	(118.0)
T.W.L.	=	184.8	(90.7)

CASE IV Normal Flood Conditions (3' over flashboards and prior to flashboard collapse)

H.W.L.	=	223.1	(129.0)
T.W.L.	=	185.1	(91.0)

CASE V Flood of Record Q = 185,000 cfs

H.W.L.	=	231.4	(137.3)
T.W.L.	=	222.9	(128.8)

CASE VI Probable Maximum Flood Q = 567,100 cfs

H.W.L.	=	251.0	(156.9)
T.W.L.	=	247.0	(152.9)

STABILITY SUMMARY

CONDITION	BASE TOT LEN.	BASE CR. LEN	BASE EFF LEN	ΣH (KIPS)	ΣV (KIPS)	ΣH/ΣV	$S_{s\text{-}f}$	RESULTANT FROM DOWNSTREAM	ΣMr (K-FT)	ΣMo (K-FT)	ΣMr/ΣMo	BASE STRESS (psi) UPSTREAM	BASE STRESS (psi) DOWNSTREAM
CASE I	87.5	--	87.5	2657	13853	0.19	25.53	39.0	590,493	50,295	11.74	30.88	60.74
~~CASE II~~	~~87.5~~		~~87.5~~	~~2095~~	~~13853~~	~~0.22~~	~~22.65~~	~~28.25~~	~~590,493~~	~~60,566~~	~~9.75~~	~~28.55~~	~~63.07~~
CASE III	87.5	--	87.5	2777	13853	0.20	24.43	38.52	590,493	56,879	10.38	29.38	62.24
CASE IV	87.5	--	87.5	2763	14494	0.19	24.73	39.71	631,198	55,629	11.35	34.65	61.20
CASE V	87.5	--	87.5	516	9933	0.05	125.77	41.03	421,074	8,591	49.02	27.86	37.85
CASE VI	87.5	--	87.5	96	3828	0.05	607.10	44.12	688,679	608,022	1.13	6.20	5.89

*Tensile crack propagates through full length of base. Loadings, stresses and functions thereof are based on uncracked section.

Powerhouse Unit 5

NEW ENGLAND POWER COMPANY
WESTBOROUGH, MASSACHUSETTS

VERNON PROJECT L.P. 1904

STABILITY SUMMARY

MAIN

DATE NOVEMBER 1982

1270 - 097 - IV

Fig. G1

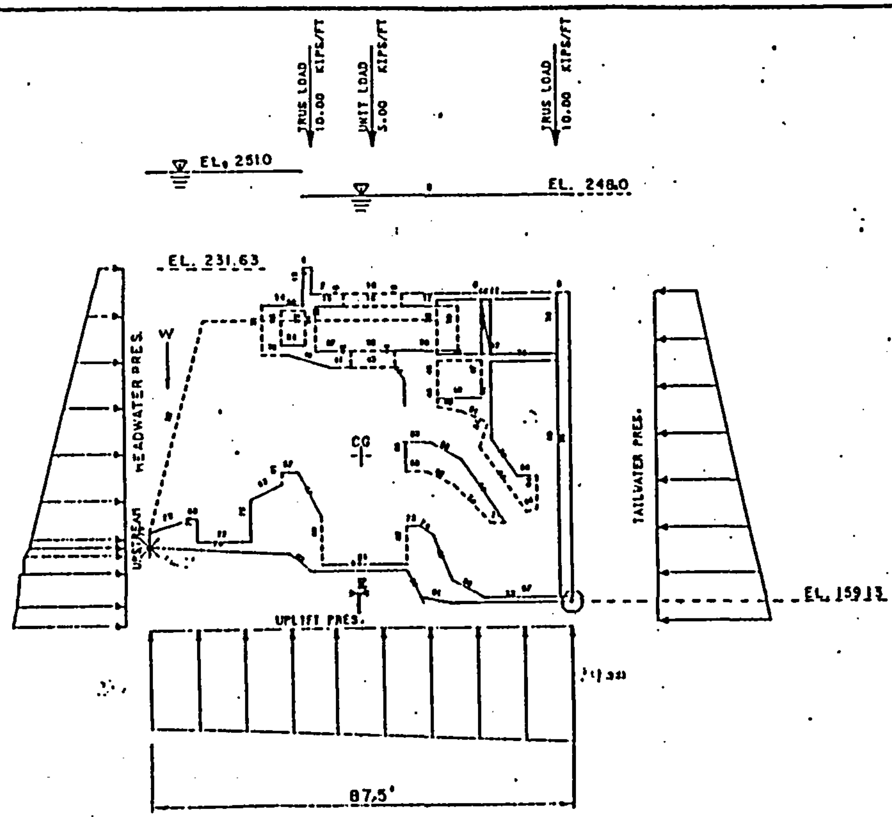
STABILITY SUMMARY

CONDITION	BASE			Σ FH (kips)	Σ FV (kips)	$\frac{\Sigma FH}{\Sigma FV}$	S s-f	RESULTANT FROM DOWNSTREAM	Σ Mr (k-ft)	Σ Mo (k-ft)	$\frac{\Sigma Mr}{\Sigma Mo}$	BASE STRESS (psi)	
	TOT. LENGTH	CR. LEN.*	EFF. LEN.									UPSTREAM	DOWNSTREAM
Case II (CEI)	87.5	--	87.5	3333	13,853	0.24	20.4	37.5	590,493	70,837	8.33	26.4	65.6

*Tensile crack propagates through full length of base. Loadings, stresses and functions thereof are based on uncracked section.

Powerhouse Unit 5

New England Power Company Westborough, MA	Vernon Project	STABILITY SUMMARY
GEOTECHNICAL ENGINEERS INC	Project 87123	Oct. 30, 1987 Fig. C



LOADING CONDITION NO. 6
PMF HEADWATER LEVEL = 251.0

$\Sigma FH = -96.90$
$\Sigma FV = 1828.32$
$\Sigma FH/\Sigma FV = 0.05$
SSF = 607.10
RESULTANT AT X = 43.38
$\Sigma MR = 688678.75$
$\Sigma MO = -608021.75$
$\Sigma MR/\Sigma MO = 1.13$
BASE STRESS (PSI)
6.20 AT X = 0.00
5.89 AT X = 87.50

VERNON STATION POWERHOUSE UNIT 5

0 20 40 60 80
Scale In Feet

Fig. G3

STABILITY SUMMARY

NOVEMBER,

MAIN

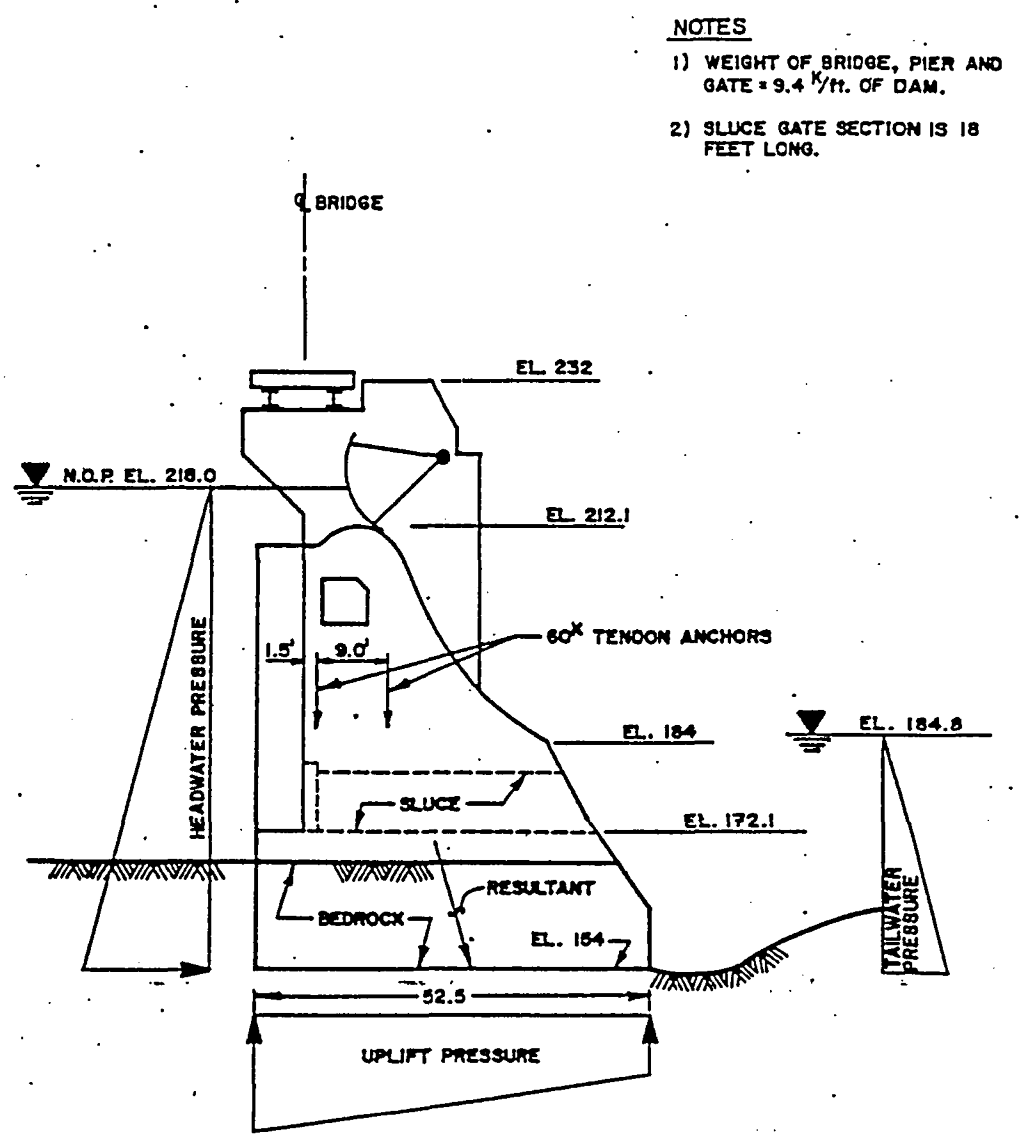
1270 - 097 - 1.

STABILITY SUMMARY

CONDITION	BASE TOT. LENGTH	BASE CR. LEN.	BASE EFF. LEN.	Σ FH (kips)	Σ FV (kips)	$\frac{\Sigma FH}{\Sigma FV}$	S s-f	RESULTANT FROM DOWNSTREAM	Σ Mr (k-ft)	Σ Mo (k-ft)	$\frac{\Sigma Mr}{\Sigma Mo}$	BASE STRESS (psi) UPSTREAM	BASE STRESS (psi) DOWNSTREAM
18-Foot-Wide Section													
Case 1	52.5	--	52.5	1768	2695	0.66	15.8	23.4	194,021	131,000	1.48	13.3	26.2
Case 2	52.5	--	52.5	2365	2695	0.88	11.8	18.9	194,021	143,080	1.36	3.2	36.4
Case 3	52.5	--	52.5	1460	2785	0.52	19.1	25.9	190,124	117,755	1.61	19.6	21.3
Case 4	52.5	--	52.5	2125	2830	0.75	13.2	19.4	203,575	148,640	1.37	4.5	37.0
Case 5	52.5	--	52.5	550	2677	0.20	50.7	32.6	275,882	188,700	1.46	33.9	5.4
Case 6 *	--	--	--	--	--	--	--	--	--	--	--	--	--

* Headwater is 39 feet above spillway crest. Tailwater is 4 feet lower, 35 feet above spillway crest. Spillway is fully submerged during PMF and stable by inspection.

New England Power Company Westborough, MA	Vernon Project	STABILITY SUMMARY SLUICE GATE SECTION
	Project 87123	Sept. 30, 1987 Fig. C6



CASE 1: NORMAL OPERATING POOL

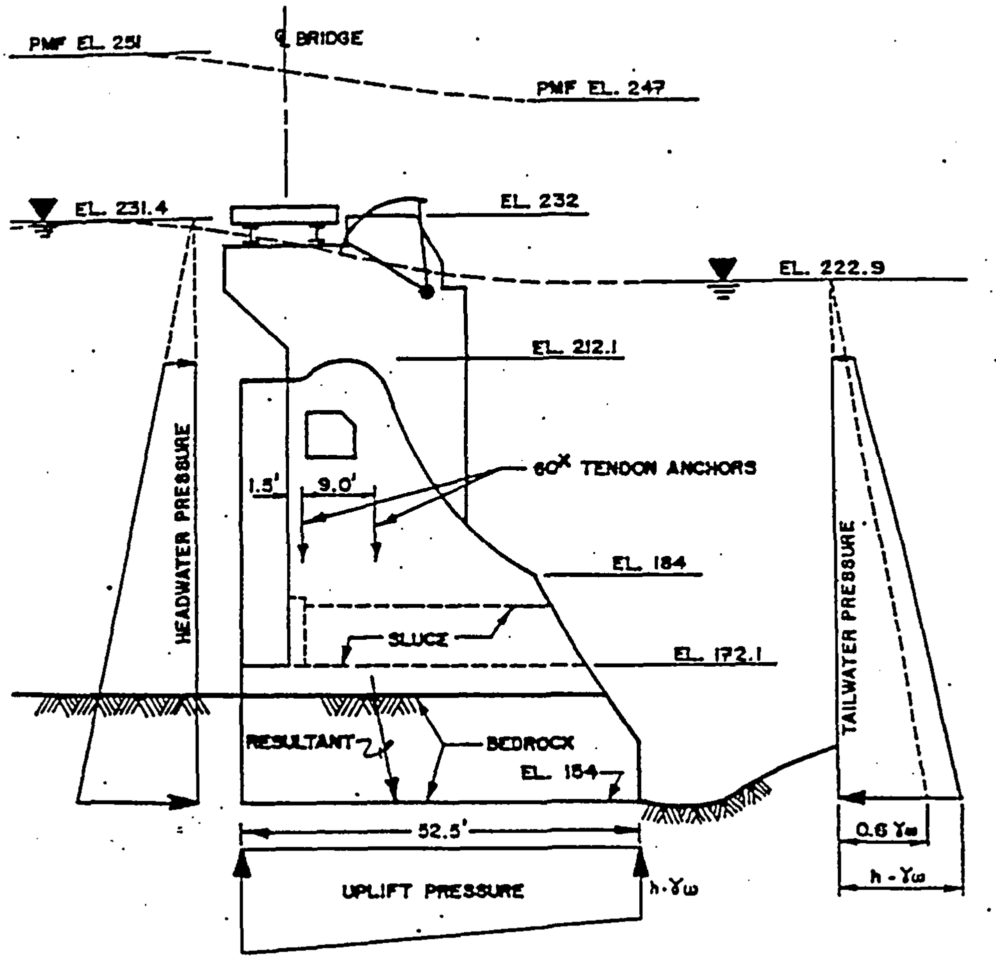
0 20 40

SCALE: 1" = 20'

New England Power Company Westborough, Massachusetts	Vernon Project	LOADING DIAGRAM SLUICE GATE SECTION NORMAL OPERATING POOL
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 87123	October 30, 1987 Fig. G5

NOTES

1) WEIGHT OF BRIDGE, PIER AND GATE = 9.4 K/ft. OF DAM.
2) SLUCE GATE SECTION IS 18 FEET LONG.



CASE V: FLOOD OF RECORD

0 20' 40'
SCALE: 1" = 20'

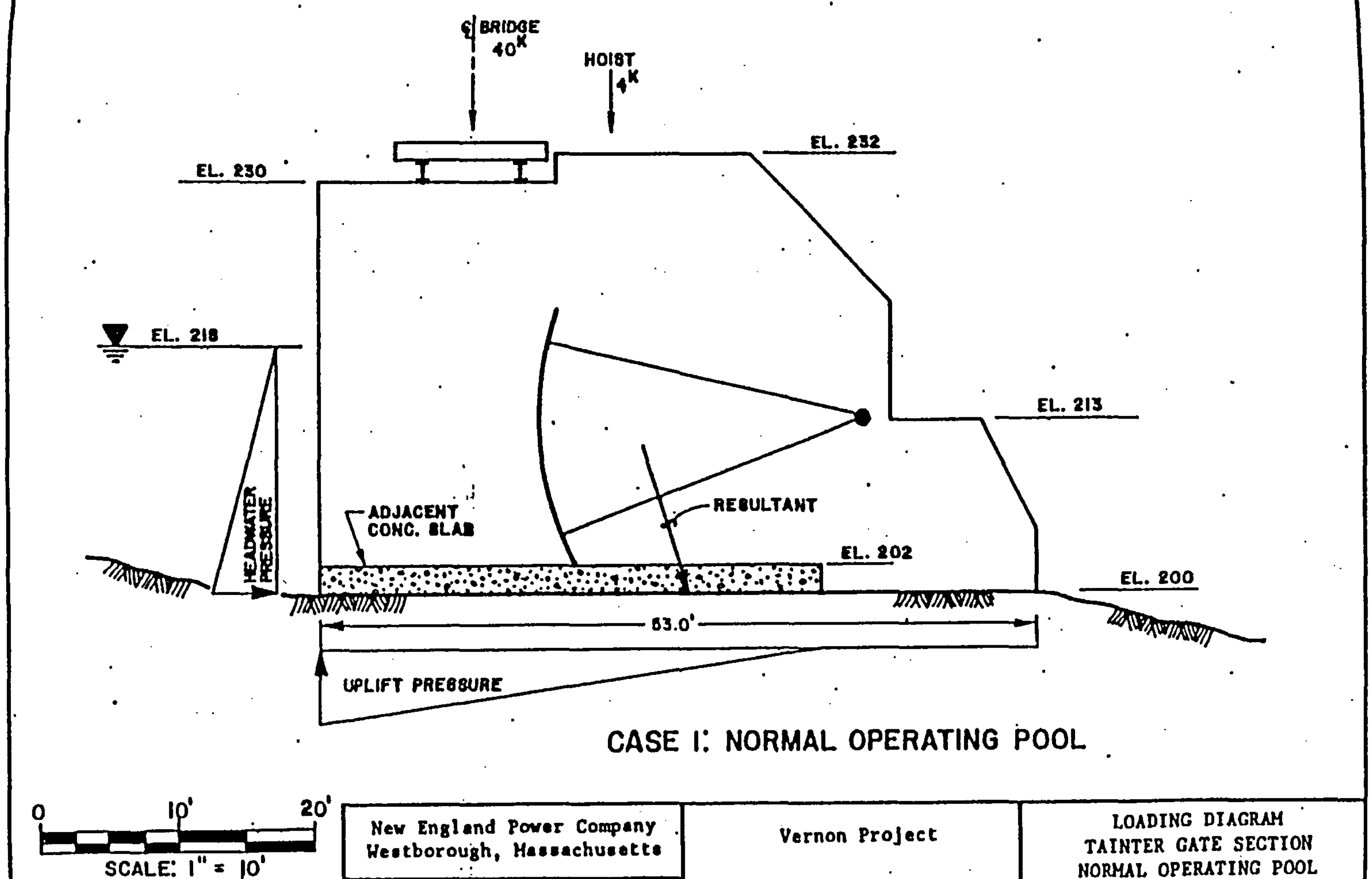
New England Power Company Westborough, Massachusetts	Vernon Project	LOADING DIAGRAM SLUICE GATE SECTION FLOOD OF RECORD
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 87123	October 30, 1987 Fig. G6

STABILITY SUMMARY

CONDITION	BASE			Σ FH (kips)	Σ FV (kips)	$\frac{\Sigma FH}{\Sigma FV}$	S s-f	RESULTANT FROM DOWNSTREAM	Σ Mr (k-ft)	Σ Mo (k-ft)	$\frac{\Sigma Mr}{\Sigma Mo}$	BASE STRESS (psi)	
	TOT. LENGTH	CR. LEN.	EFF. LEN.									UPSTREAM	DOWNSTREAM
55 Foot Unit Block													
Case 1	53.0	--	53.0	550	994	0.55	14.5	25.7	33,021	7,494	4.41	23.6	28.5
Case 2	53.0	--	53.0	598	984	0.61	13.3	25.6	33,021	7,800	4.23	23.1	28.4
Case 3	53.0	--	53.0	523	1029	0.51	15.3	25.2	33,021	7,111	4.64	22.9	31.0
Case 4	53.0	--	53.0	907	955	0.95	8.7	21.6	33,021	12,400	2.66	11.1	38.9
Case 5	53.0	--	53.0	104	1003	0.10	76.7	29.3	33,390	3,964	8.42	34.6	17.9
Case 6 *	53.0	--	53.0	--	--	--	--	--	--	--	--	--	--

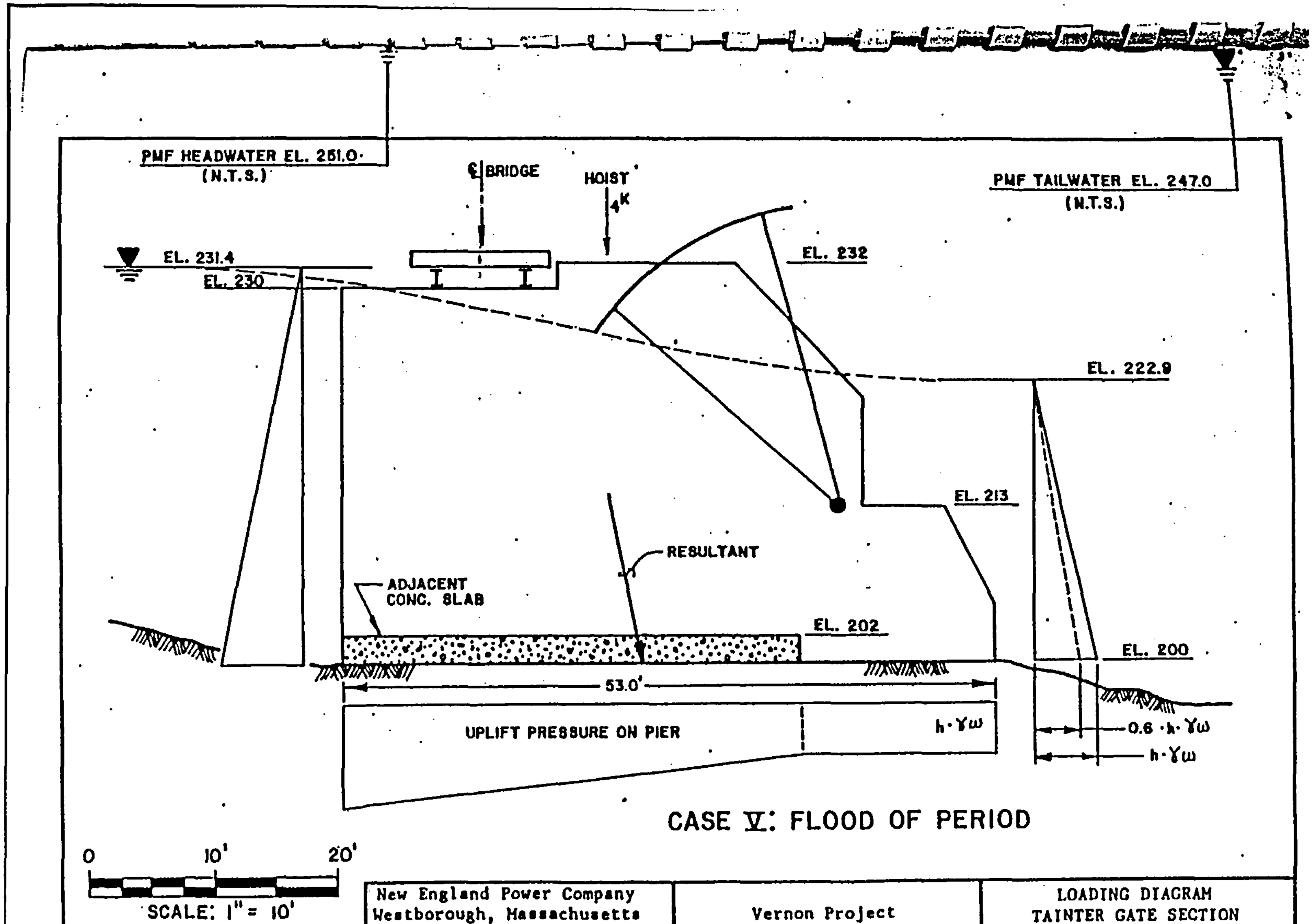
* Headwater is 39 feet above spillway crest. Tailwater is 4 feet lower, 35 feet above spillway crest. Spillway is fully submerged during PMF and stable by inspection.

New England Power Company Westborough, MA	Vernon Project	STABILITY SUMMARY DEEP TAINTER GATE
[illegible]	Project 87123	Sept. 30, 1987 Fig. G7



CASE I: NORMAL OPERATING POOL

New England Power Company Westborough, Massachusetts	Vernon Project	LOADING DIAGRAM TAINTER GATE SECTION NORMAL OPERATING POOL
Φ GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 87123	Octbber 30, 1987 Fig. C8



CASE V: FLOOD OF PERIOD

New England Power Company Westborough, Massachusetts	Vernon Project	LOADING DIAGRAM TAINTER GATE SECTION FLOOD OF RECORD
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 87123	October 30, 1987 Fig. G9