PACIFIC GAS AND ELECTRIC COMPANY

77 Beale Street

San Francisco, California 94106

September 27, 1973

U. S. Atomic Energy Commission
Washington, D. C. 20545

Attention: Mr. Donald J. Skovholt
Assistant Director for
Operating Reactors
Directorate of Licensing

Re: Docket No. 50-133
License No. DPR-7
Humboldt Bay Power Plant

Gentlemen:

In our meeting of September 5, 1973, you requested that we give you a letter confirming the description of the investigative programs that we discussed at that meeting. The following investigative programs are described in this letter:

I. Geology
II. Seismology
III. Liquefaction and Dynamic Response
IV. Seismic Design Review

I. Program for Additional Geologic Studies and Exploration Work

A. Introduction

This section outlines a program of studies and exploration work for the Humboldt Bay Power Plant site, which would be conducted by Dr. G. H. Curtis and Earth Science Associates.

Attached Figure 1 is a map showing the proposed location of the principal geophysical and subsurface exploration activities.

B. Background

In a letter dated October 28, 1971, the AEC discussed the geology and seismicity of the Humboldt Bay Power Plant site, and specifically requested PG&E to make determinations regarding the location and activity of the Little Salmon fault. This was done during the course of an investigation conducted by Dr. G. H. Curtis and D. H. Hamilton of Earth Sciences Associates. The results of the investigation were presented in the report, "Geology of the Southern Humboldt Bay Area and the Humboldt Bay Power Plant Site." This report was forwarded to the AEC and the major conclusions were summarized in a letter dated January 17, 1973, from PG&E to the AEC which formally answered the AEC letter of October 28, 1971. Subsequently, the USGS sent the AEC a draft review by letter dated February 16, 1973, commenting on the geology report. The points raised in that review, which encompassed the general geology and seismicity of the Humboldt Bay site, were then discussed in a meeting held on June 13, 1973, between AEC and PG&E.

The outcome of the June 13 conference was the identification of several specific areas where the AEC proposed that more information be obtained. Our understanding of the two major geological items for obtaining additional data was as follows:

1. Further definition of critical elements of stratigraphy, especially the distribution of and bases for differentiating between the Hookton and Carlotta Formations.

2. Further specific evidence of the precise location, geometry, interrelations, and age relationships of the Little Salmon fault and the possible fault along the Bay Entrance alignment of anomalies.

Other comments indicated interest in other areas as well, particularly the distribution of marine terraces in the area.

C. Proposed Geologic Program

1) Stratigraphy. The question of the characteristics and distribution of the Hookton and Carlotta Formations would be reviewed in consultation with Dr. B. A. Ogle. Questions of stratigraphic and structural interpretation would first be reviewed in the office, and, if necessary, a program of field explorations designed. Such a program could include drilling a continuously sampled boring from an area of recognized Hookton over Carlotta, such as Table Bluff, perhaps supplemented by excavation of a few backhoe test pits in critical areas to provide specific data pertinent to resolution of the question. Special petrologic studies to detect possible differences in provenance, mineralogic studies, or paleomagnetic studies may also be made in order to ascertain whether any bases exist for distinguishing between Hookton and Carlotta rocks by such means. The program would include a field review by Dr. Ogle, and his views and conclusions would be set forth in a written report or statement.

2) Verification of location of and relationships between the Little Salmon and possible Bay Entrance faults. Much of the discussion has focused on the following points:

   a. The position of the Little Salmon fault north of its point of intersection by the Brauner Well on Humboldt Hill.

b. The relationship of the Little Salmon fault to the Hookton Formation.

c. The location and dip of the possible Bay Entrance fault east of Buhne Point, and

d. The relationship of the possible Bay Entrance fault to the Little Salmon fault, and to the Hookton Formation.

The acquisition of additional data would be done on a staged basis, in order to allow review and evaluation of each stage before going on to the next one. Successive stages would represent increasingly detailed exploration work. The approximate location of the proposed exploration work is shown on Figure 1. The staged program can be outlined as follows:

Stage I

a. Perform further analysis of existing gravity and seismic profiling data pertinent to the location and geometry of the Little Salmon fault and the Bay Entrance anomalies.

b. Make two seismic refraction survey traverses designed to locate the Little Salmon fault at the north end of Humboldt Hill and under the Elk River valley.

c. Make one or two detailed gravity traverses and possibly one seismic refraction traverse to attempt to trace the Bay Entrance anomaly landward and target it more precisely for subsurface exploration.

Stage II

Borings would be made at several locations chosen on the basis of available data plus results from the Stage I work, to bracket and verify the location of the Little Salmon fault and any fault along the Bay Entrance anomaly trend. Several of these borings would be extended through the actual fault planes. The borings would be sampled continuously or at selected intervals and electrically logged.

A report presenting data and conclusions from the Stage I and II work would be prepared at the conclusion of Stage II.

II. Program for Additional Seismologic Studies

This section outlines a program of further review of the general seismology and tectonics of the Humboldt Bay region.

A. Background

At the June 13, 1973, meeting with the AEC Staff, it was indicated that further review of the general seismology and tectonics of the Humboldt Bay region would be required in order either to specifically document, or provide bases for reevaluating, the existing conclusions regarding the Operating Basis Earthquake and Safe Shutdown Earthquake for plant design.

B. Proposed Seismologic Program

Additional studies of the seismicity of the Humboldt Bay region would be directed toward refinement and documentation of knowledge about conditions there, as previously summarized by Professor Byerly, and in addition a program of micro-earthquake recording would be established to specifically look at the Little Salmon and the possible Bay-Entrance faults in order to detect any activity and to obtain data that could be used to delineate the subsurface extension of these faults. It is proposed that this program be conducted by Dr. Stewart Smith.

The Seismologic program would include the following elements:

1. Plotting of all recorded epicenters from the Berkeley and NOAA seismographic networks.

2. Assessment of the probable error of location of epicenters in different parts of the region, at successive states of development of the recording network.

3. Establishment of a network of telemetered seismic stations in the area. Stations would be installed at a spacing of about seven miles and would be telemetered to a central location. Data would be recorded on magnetic tape with a single channel of visual recording for monitoring purposes. It is expected that this program would require at least a year of operation in order to establish the activity of faults in the area and obtain data that could be used to delineate the subsurface extension of these faults.

III. Program for Liquefaction Potential and Dynamic Response Spectra Determination

A. Introduction

At the June 13, 1973 meeting, AEC Staff personnel indicated that they were not in agreement with the Dames and Moore report which concluded that liquefaction would not take place at 0.4g. Accordingly new samples would be taken and additional studies would be made.

The principal purposes of the studies would be to evaluate the liquefaction potential of the soils in the vicinity of the plant site, and also to determine the effects of soil-structure interaction on the dynamic response of the major structures located within the Humboldt Bay Power Plant site. In order to accomplish these studies, the following would be done by Dames and Moore:

1. Comprehensive field investigation to establish the subsurface soil conditions and obtain undisturbed samples of the in-situ soils;

2. Performance of laboratory tests to classify the in-situ soils and to evaluate their static and dynamic strengths; this would include evaluation of the dynamic soil parameters (equivalent modulus and damping),

and the dynamic strength characteristics of the in-situ soils under simulated earthquake loading conditions;

3. Development of horizontal and vertical acceleration time histories to match the prescribed AEC response spectra corresponding to Safe Shutdown Earthquakes and the Operat-Basis Earthquake;

4. Deconvolution of the acceleration time hostories corresponding to the free-field motion to obtain the acceleration time histories at the base of the free field soil profile;

5. Evaluation of the liquefaction potential of the in-situ sandy soils during the Safe Shutdown and the Operating Basis Earthquakes; and,

6. Performance of two-dimensional dynamic response analyses to determine the effects of soil-structure interaction on the response within the containment structure, adjacent structures, and the intake structure.

B. Field Investigation

The purpose of the field investigation portion of the work would be to develop sufficient data on the in-situ soils for the comprehensive seismic studies to be performed on the site. This data would be developed by drilling a series of test borings. At the present time it is planned to drill a total of five borings in the vicinity of Units 1, 2, and 3 and the intake structures to depths ranging from 80 to 200 feet. The location of these borings is shown on Figure 2. Borings 1 and 3 would be drilled to depths of 200 feet while Borings 2, 4, and 5 would be drilled to depths of approximately 80 to 100 feet. The exact depths of the borings, however, would depend on the actual subsurface soil conditions encountered. Undisturbed samples of the in-situ soils would be obtained using the Dames and Moore U-Type and piston tube samplers. Standard Penetration Tests would also be performed to obtain blow count information for estimating the relative density of the in-situ sandy soils. Samples would be taken at the rates of one every five feet for the first 100 feet and one every ten feet thereafter. Standard Penetration Tests would be made within the top 100 feet immediately after taking the Dames and Moore samples.

C. Laboratory Tests

A comprehensive laboratory testing program is planned on samples of the in-situ soils obtained from the field investigation. The laboratory tests to be performed would include: vane shear tests, unconfined compression tests, consolidated-drained triaxial tests to evaluate the static shear strength of the soils, moisture-density tests to evaluate the in-place dry densities and moisture contents, and classification tests such as compaction tests, relative density tests, grain-size analysis and Atterberg Limits.

Strain-controlled cyclic triaxial tests and resonant column tests would be performed on samples of the in-situ soils to evaluate their dynamic soil properties (equivalent modulus of rigidity or Young's modulus and equivalent viscous damping) and their variation with the level of shear strain for use in strain-compatible dynamic analyses. In addition, stress-controlled cyclic triaxial tests would be performed to evaluate the liquefaction potential of the in-situ sandy soils under simulated earthquake loading conditions.

As the liquefaction potential of sandy materials is very dependent on the duration of strong shaking, it is important, in developing acceleration time histories for the Safe Shutdown and Operating Basis Earthquakes, to take this factor into consideration. It is proposed that for the Safe Shutdown Earthquake, an acceleration time history with a total duration of 30 seconds be used with the time between the first and last acceleration peak exceeding 0.05g being on the order of 24 to 27 seconds. For the Operating Basis Earthquake, an acceleration time history with a total duration of 24 seconds would be used.

D. Acceleration Time Histories for Safe Shutdown Earthquake and Operating Basis Earthquake

It is planned to develop acceleration time histories for the Safe Shutdown Earthquake for peak horizontal accelerations of 0.4g and higher, and for the Operating Basis Earthquake for a peak horizontal acceleration of 0.25g. For each earthquake, both horizontal and vertical acceleration time histories would be developed to satisfy, as closely as possible, the AEC response spectra criteria for various values of structural damping ratio. The procedure which would be used would consist of modifying artificially generated earthquakes so that their response spectra closely matches the prescribed AEC spectra. Dames and Moore has developed analytical procedures using Fast Fourier Transform techniques for doing this.

D. Development of Acceleration Time Histories at Base of Analytical Models

It is planned to develop the design response spectra and the corresponding acceleration time histories by taking the free-field ground motions at a depth corresponding to the competent foundation soils present at the site. Based on previous investigations performed by Dames and Moore at the site, the average depth to the competent soils at the Humboldt Bay Plant site, from a bearing capacity point-of-view, is approximately 20 feet below the existing ground surface. This upper 20 feet consists mainly of weak to moderately stiff, compressible clayey soils.

Assuming the free-field ground motions to be at a depth of 20 feet below the ground surface, the acceleration time histories at the base of the analytical models used in our studies would have to be evaluated for use in the dynamic response analyses. For the liquefaction studies this would be accomplished using a deconvolution procedure developed at the University of California at Berkeley by Schnabel, Lysmer and Seed (1972). This procedure uses a one-dimensional wave propagation model of the free-field soil profile. Once the acceleration time histories at the base of the free-field soil profile have been developed, the liquefaction potential of the in-situ soils can be evaluated. For the soil-structure interaction analyses wave propagation or finite element procedures would be used to deconvolute both the horizontal and vertical acceleration time histories.

F. Liquefaction Potential of the In-Situ Soils

One-dimensional, strain-compatible wave propagation analyses would be performed to evaluate the liquefaction potential of the in-situ soils when subjected to the horizontal input base motions corresponding to the Safe Shutdown Earthquake and the Operating Basis Earthquake. The liquefaction potential of a horizontal soil deposit is very dependent on the time history of horizontal shear stress induced at various depths within the soil deposit by the earthquake excitation. Previous studies have shown that these stresses are not affected by the vertical component of the input motion. Thus, it would not be necessary to include the vertical component of the acceleration time histories in the liquefaction studies. The dynamic soil properties obtained from the laboratory tests would be used in these analyses.

The liquefaction potential of the in-situ soils would be evaluated using two different procedures. The first procedure is similar to that proposed by Seed and others (1969), and consists of the following steps:

1. Representation of the time history of induced shear stress at different levels within the soil profile by an equivalent uniform cyclic shear stress acting for N number of cycles.

2. Evaluation of the cyclic shear stresses required to develop failure by liquefaction in N number of cycles for laboratory soil samples consolidated at the same pressures as those existing at various depths within the soil profile.

3. Comparison of theequivalent uniform cyclic shear stresses induced during the earthquake with the laboratory cyclic shear stresses required to develop failure by liquefaction in N number of cycles in order to evaluate the liquefaction potential of the in-situ sandy soils.

The choice of the significant number of cycles and the average shear stress for use in this procedure depends on the judgement of the individual performing the analysis. In order to overcome this, a second procedure developed by Donovan (1971), utilizing a stochastic approach to liquefaction analyses, would also be used. It is essentially a discrete cumulative damage approach and involves evaluation of the cumulative damage produced by each stress cycle of the shear stress time history. Modifications have been made to Donovan's original presentation in that the actual computed shear stress time histories induced by the earthquake are used to evaluate the liquefaction potential of the soil instead of using statistical earthquake parameters. By summation of the cumulative damage produced by all the stress cycles corresponding to each shear stress time history, the liquefaction potential at various depths within the soil profile could be evaluated. The validity of this technique has been confirmed by evaluation of liquefaction during past earthquakes.

G. Dynamic Analyses to Evaluate Soil-Structure Interaction Effects

The following three cross sections have been selected for analyses:

1. A cross section through the containment and refueling building of Unit No. 3 and the adjacent auxiliary bay and turbine building;

2. A cross section perpendicular to the first cross section including Units 1, 2, and 3; and,

3. A cross section through the intake structures of Units 1, 2, and 3.

In the soil-structure interaction models which would be developed for these cross sections, the structures adjacent to the containment and refueling building and the intake structures would only be modeled approximately since the main reason for modeling these structures is to determine the effect of their mass and stiffness on the dynamic response at their foundation level and in the vicinity of the structures. The dynamic response of the soil-structure interaction model for each of the above cross sections would be obtained using either a two-dimensional, strain-compatible, finite element technique or a two-dimensional, strain-compatible, wave propagation analysis. These procedures allow different moduli and damping to be used in various portion of the models. The two-dimensional wave propagation computer program has only recently been developed at the University of California at Berkeley, and is well-suited for analyses of stiff systems, since it does not filter out any of the high frequencies of the earthquake motion. In addition, it makes it possible to deconvolute both the vertical and horizontal components of an acceleration time history, simultaneously, to the base of the free-field soil model.

IV. Program for Seismic Design Review

A. Introduction

This section outlines a proposed program for reviewing the seismic design of the structures, systems, and components important to nuclear safety. It is proposed that this program be done by the Bechtel Power Corporation, the original architect-engineer for the plant. This program does not include the reactor vessel, reactor core and reactor vessel internals which have been reviewed by General Electric Co. The results of the General Electric Co. analysis would be submitted in the future.

In the event that portions of structures, systems, or equipment exceed allowable code values, prudent engineering judgement and a more sophisticated analysis might be used to assure overall acceptability.

An Operating Basis Earthquake equal to 0.25g would be used in the analysis.

Safety related structures, systems, and equipment would be evaluated to establish what Safe Shutdown Earthquake the plant is capable of withstanding.

B. Structures

1) General

For the normal and accident conditions, the structural design would be checked in accordance with the codes and regulations listed in Table I and with loading combinations shown in Tables II and III.

Concrete structures would be analyzed using ultimate strength method described in ACI 318-71. "Yield Line Theory" may be used, however, the combined loads would be limited to 90% of the calculated failure capacity. Effects of excessive deflections and cracking would be considered in verifying no loss of function of any safety-related system.

Structural steel would be analyzed in accordance with the AISC Manual of Steel Construction, Parts 1 and 2.

2) Soil-Structure Interaction and Structural Models

The following models would be used in conjunction with the finite-element, Dames and Moore soil-structure interaction investigation:

a. Appropriate structural mathematical models would be constructed for structures to be included in the finite element, soil structure interaction investigations. These would include:

1. Unit 3 Refueling Building
2. Unit 3 Turbine Building
3. Unit 3 Auxiliary Bay
4. Intake Structure for Units 1, 2, and 3
5. Unit 2 Structure
6. Unit 1 Structure

All of the models would be two or three dimensional, as necessary, and would be used for later analyses in the following directions:

1. North-South
2. East-West
3. Vertical

b. Fixed-base analyses would be used to obtain the natural frequencies and mode shapes of all of the above structures.

c. Simple mathematical finite element models or lump mass models of all of the above structures would be used to match as closely as possible the natural frequencies and, mode shapes of b. above. These would be used by Dames and Moore in its analyses.

d. All of the above analyses would be conducted using the latest available techniques, in accordance with Bechtel document BC-TOP-4 and other applicable criteria. The computer codes are based on linear, elastic theory for both static and dynamic analyses.

TABLE I

Governing Codes and Regulations

1. Uniform Building Code (UBC) 1970 Edition

2. American Institute of Steel Construction (AISC)
"Manual of Steel Construction" 7th Edition, 1970
"Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" adopted 2/12/69.

AISC "Specification for Structural Joints Using ASTM A325 or A490 Bolts".

3. American Iron and Steel Institute (AISI)

"Specification for the Design of Cold-Formed Steel Structural Members" - 1968.

4. American Concrete Institute (ACI)

"Building Code Requirements for Reinforced Concrete" - (ACI 318-71).

5. American Welding Society (AWS)

AWS Structural welding code AWS D 1.1-72

6. Atomic Engergy Commission (AEC)

Publication TID 7024 "Nuclear Reactors and Earthquakes"

7. American Petroleum Institute (API)

   1. API Standard 620 "Recommended Rules for Design and Construction of Large Welded Low-Pressure Storage Tanks"Fourth Edition, February 1970.

   2. API Standard 650 "Welded Steel Tanks for Oil Storage" June 1970.

8. USA Standards Institute

"Specification for Aluminum Alloy Tanks" (B96.1 - 1967).

9. American Society of Mechanical Engineers (ASME)

"Boiler and Pressure Vessel Code": 1971 including addenda.

Section III - Nuclear Power Plant Components

10. American Society of Civil Engineers (ASCE)

Paper No. 3269 "Wind Forces on Structures".

TABLE II

Load Combinations for Safety Related Structures
During Normal Operation

Design Load Combinations Concrete Structures (1)	Stress Limits
a. $U = 1.4\ D + 1.7L + 1.1\ T_o + 1.1\ H_o$ b. $U = 0.75\ (1.4D + 1.7L + 1.7W) + 1.1\ T_o + 1.1\ H_o$ c. $U = 0.75\ (1.4D + 1.7L + 1.9E) + 1.1\ T_o + 1.1\ H_o$ d. $U = 0.9D + 1.3W + 1.1\ T_o + 1.1\ H_o$ e. $U = 0.9D + 1.4E + 1.1\ T_o\ 1.1\ H_o$	NOT APPLICABLE
Design Load Combinations Steel Structures	
f. $D + L + T_o + H_o$	F_s
g. $D + L_o + T_o + H_o + E$	$1.33\ F_s$
h. $D + L_o + T_o + H_o + W$	$1.33\ F_s$

(1) The appropriate equations shall be checked for H and F in accordance with Section 9.3 of ACI 318-71.

TABLE III

Load Combinations for Safety Related Structures During Accident Conditions

Design Load Combinations Concrete Structures (1)
a. $U = D + L + T_A + H_A + R + 1.5\ P$ b. $U = D + L_o + T_A + H_A + R + 1.25\ P + 1.25\ E$ c. $U = D + L_o + T_A + H_A + R + P + E'$ d. $U = D + L + T_o + H_o + E'$ e. $U = D + L + T_o + H_o + 1.9E$ (See Note 2)
Design Load Combinations Steel Structures (either method may be used)
A. Elastic Working Stress Method: f. $1.6S = D + L + T_A + H_A + R + P$ g. $1.6S = D + L + T_A + H_A + R + P + E$ h. $1.6S = D + L + T_A + H_A + R + P + E'$ B. Plastic Design Method: i. $.9Y = D + L + T_A + H_A + R + 1.5\ P$ j. $.9Y = D + L + T_A + H_A + R + 1.25P + 1.25E$ k. $.9Y = D + L + T_A + H_A + R + P + E'$

(1) The appropriate equations shall be checked for H and F in accordance with Section 9.3 of ACI 318-71 except that load factors shall be 1.0.

(2) For structural elements whose primary function is to resist earthquake forces, such as struts and braces.

NOTATIONS

D = Dead load of structure and equipment plus any other permanent loads contributing stress.

E = Operating Basis Earthquake

E' = Safe Shutdown Earthquake

F = Lateral pressures from liquids resulting from normal operating or accident conditions.

F_s = Allowable stress for structural or reinforcing steel.

f_s = Calculated stress in structural or reinforcing steel.

F_y = Yield Strength for steel.

H = Lateral earth pressure.

H_A = Force on structure due to thermal expansion of pipes under accident conditions.

H_o = Force on structure due to thermal expansion of pipes under operating conditions.

L = Minimum floor design live load.

L_o = Live loads expected to be present when the plant is operating.

P = Vapor pressure load resulting from rupture of process piping.

R = Jet forces or reactions resulting from rupture of process piping (includes missile, jet impingement and equivalent static loading to structure.)

S = Required section strength for structural steel, based on allowable stresses as defined in Part 1 of AISC.

T_A = Thermal loads due to temperature gradient through wall under accident conditions.

T_o = Thermal loads due to temperature gradient through wall under operating conditions.

U = Required strength to resist design loads as defined in ACI 318-71.

W = Wind load.

Y = Required section strength for structural steel, based on plastic design as defined in Part 2 of AISC.

ϕ = Capacity reduction factor (defined in ACI 318-71 Section 9.2).

3) Structural Investigations

a. Refined two or three dimensional mathematical models of the existing structures would be used as necessary, for the ensuing detailed structural adequacy investigations. Models would be needed for the following:
   1. Unit 3 Refueling Building
   2. Unit 3 Turbine Building
   3. Unit 3 Auxiliary Bay
   4. Intake Structure for Unit 1, 2, and 3
   5. Unit 2 Structure
   6. Unit 1 Structure
   7. Appendages such as the stack and spend fuel pool for Unit 3.

b. The above structures would be analyzed with linear elastic analysis programs, including lumped mass or finite element, as necessary, in accordance with applicable documents such as BC-TOP-4, for all loading combinations.

c. Components of structures would be analyzed whenever possible or necessary to get a better definition of their behavior.

d. If the above linear analyses indicate potential problems with the structures of the structural components, other methods such as inelastic analysis might be investigated and used.

e. If it becomes necessary, certain structural components or structures would be tested to obtain better damping, mode shape, and frequency data at low levels of dynamic excitation. Computer models would be constructed which match the experimentally determined properties. The forces at extrapolated higher accelerations would then be estimated.

f. Whenever modifications in the structure or structural components appear necessary, such modification would be incorporated first in the linear computer models, and then, if necessary, in the inelastic models.

g. Whenever necessary, the effects in variations of properties (such as moduli of elasticity) on the dynamic behavior of the structures and the structural components would be examined.

h. Modal spectral, and sometimes time-history, analyses would be used for the elastic analyses. Time-history analyses - both modal and step-by-step (or exact) would be used for the non-linear investigations.

C. Piping Systems Investigations

Piping systems would be analyzed in accordance with ASME B&PV Code, Section III, Class 2.

1) Dynamic analysis would be conducted in accordance with BP-TOP-1 or other applicable documents.

2) Inelastic analysis would be done if necessary.

3) Certain piping systems would be tested at low levels to determine natural properties, then results would be extrapolated to higher excitation levels, if necessary. (See B.3.e)

D. Buried Piping Investigations

1) From differential building displacements, as determined in the structural investigations, the deflections and forces in the buried piping systems would be determined. Elastic analyses would be used to qualify systems, as defined by the appropriate criteria, such as BC-TOP-4.

2) Inelastic analyses would be used if appropriate or necessary to qualify systems.

E. Equipment Investigations

1) Available data such as testing reports and analysis reports, on the safety essential equipment would be collected.

2) The above data would be compared to the plant equipment and the plant equipment qualified on the basis of similarity, etc. The ability of the equipment to withstand the postulated earthquake within its functional requirements would be evaluated.

3) The equipment not included above would be modeled and analyzed when appropriate, i.e. when the equipment lends itself to analysis. Standard elastic programs would be used and/or inelastic, non-linear analysis would be used if necessary.

4) Equipment which does not lend itself to analysis would be tested, in-situ, to determine its natural dynamic properties, etc. High level dynamic response would be extrapolated from the low-level response through the use of further computer analysis. (See B.3.e)

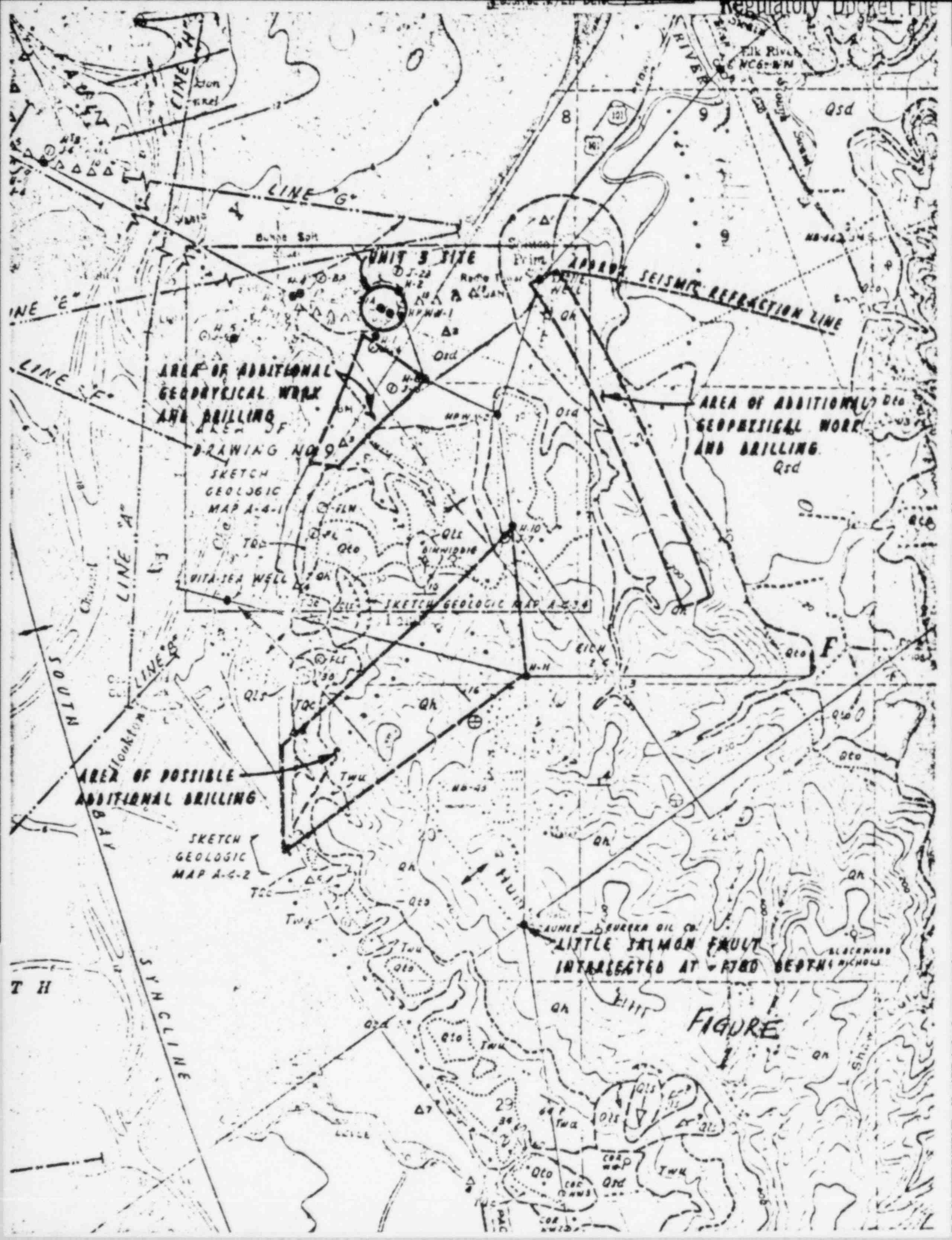
5) If all of the above methods are inadequate, equipment would be tested in-situ to high acceleration levels in order to qualify it.

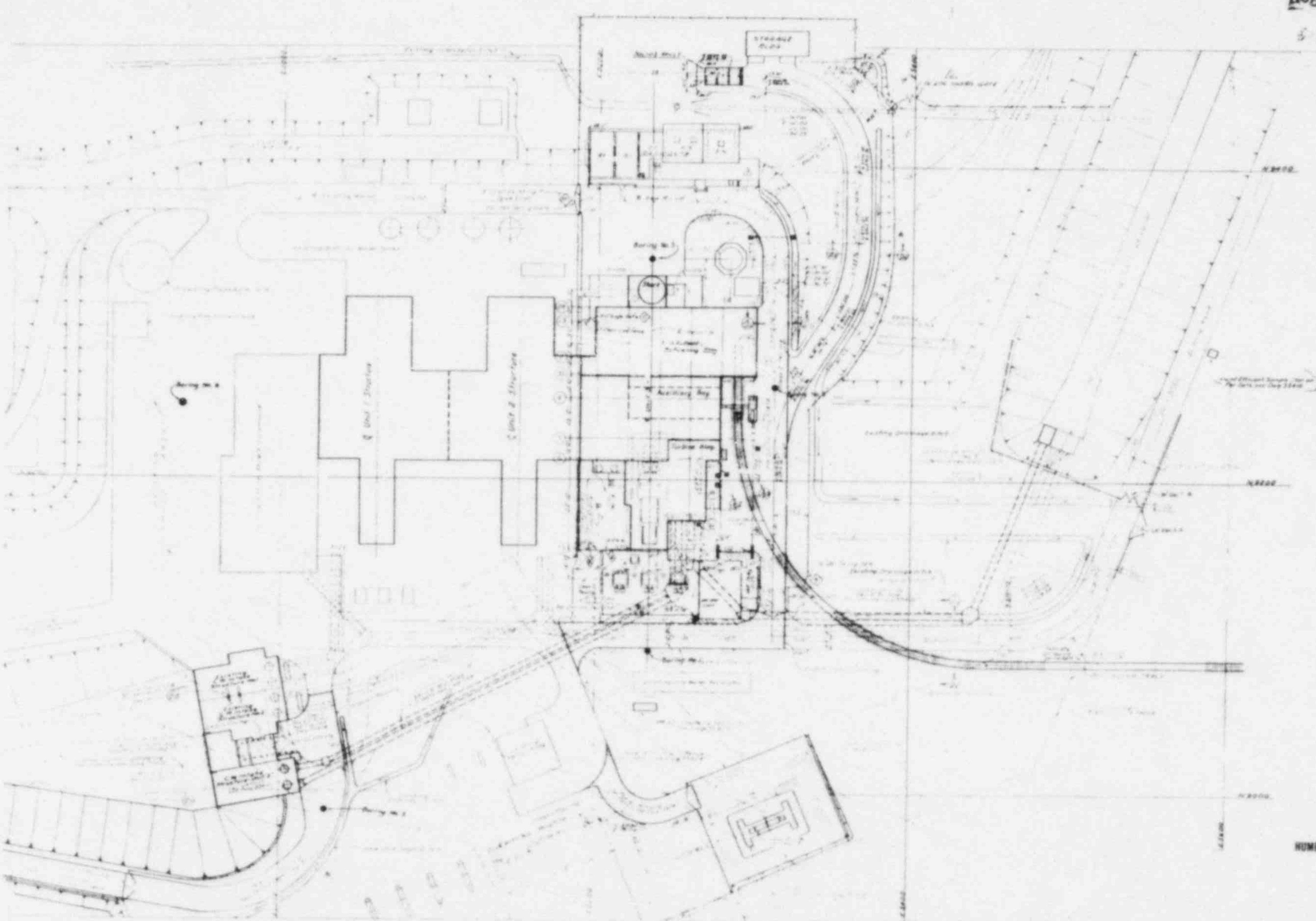
6) Throughout the equipment investigations, it would be necessary to rely on competent engineering judgement to qualify equipment which is often both difficult to analyze or test in-situ.

Very truly yours,

F. T. Searle

Enclosures - 39


UNIT 3 SITE
APPROX SEISMIC REFRACTION LINE
AREA OF ADDITIONAL GEOPHYSICAL WORK AND DRILLING
AREA OF ADDITIONAL GEOPHYSICAL WORK AND DRILLING
AREA OF DRAWING NO 9
SKETCH GEOLOGIC MAP A-4-1
SKETCH GEOLOGIC MAP A-4-3,4
SKETCH GEOLOGIC MAP A-4-2
VITA-SEA WELL
AREA OF POSSIBLE ADDITIONAL DRILLING
LITTLE SALMON FAULT INTERSECTED AT ±1700 DEPTH
LINE "G"
LINE "E"
LINE "F"
LINE "A"
LINE "B"
LINE "H"
SOUTH BAY SYNCLINE
Elk River
RIVER
FIGURE 1



HUMBOLDT BAY POWER PLANT

FIGURE 2

(TEMPORARY FORM)

CONTROL NO: 7301

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CLASS	UNCLASS	PROP INFO	INPUT	NO CYS REC'D	DOCKET NO:
	XXXX			40	50-133

DESCRIPTION:
Ltr re their meeting 9-5-73....furnishing info on I) Geology II) Seismology III) Liquefaction & Dynamic Response IV) Seismic Design Review...

PLANT NAME: Humboldt Bay

ENCLOSURES:

ACKNOWLEDGED

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BOYD	SHAO		SERVICE (L)	DUBE
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DEYOUNG(L)(PWR)	HOUSTON	MULLER	SMITH (L)	INFO
✓SKOVHOLT (L)	NOVAK	DICKER	TEETS (L)	C. MILES
P. COLLINS	ROSS	KNIGHTON	WADE (E)	
	IPPOLITO	YOUNGBLOOD	WILLIAMS (E)	
✓REG OPR	TEDESCO	REGAN	WILSON (L)	
FILE & REGION(3)	LONG	PROJECT LDR		
MORRIS	LAINAS			
STEELE	BENAROYA	HARLESS		
	VOLLMER			

EXTERNAL DISTRIBUTION

[redacted]

✓1 - DTIE(ABERNATHY)	(1)(2)(10)-NATIONAL LAB'S	✓1-PDR-SAN[redacted]
✓1 - NSIC(BUCHANAN)	1-R.Schoonmaker,OC,GT,D-323	1-GERALD LELLOUCHE
1 - ASLB(YORE/SAYRE/ WOODARD/"H" ST.	1-W. PENNINGTON, Rm E-201 GT	BROOKHAVEN NAT. LAB
	1-CONSULTANT'S NEWMARK/BLUME/AGBABIAN	1-AGMED(WALTER KOESTER RM-C-427-GT
✓16 - CYS ACRS [redacted] SENT TO LIC. ASST. 10-1-73 DIGGS	1-GERALD ULRIKSON...ORNL	1-RD..MULLER..F-309 GT