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STN-50-529 Palo Verde Nuclear Station, Unit 2, Arizona Publi 05000529
STN-50-530 Palo Verde Nuclear Station, Unit 3, Arizona Publi 05000530
AUTH.NAME AUTHOR AFFILIATION
VAN BRUNT,E.E. Arizona Public Service Co.
RECIP.NAME RECIPIENT AFFILIATION
KNIGHTON,G. Licensing Branch 3

SUBJECT: Forwards response to G Knighton 840830 request for documentation re water leakage from temporary lines.Item 5 stated incorrectly.Util 840530 note from K Jones not K Schecter of Bechtel.

DISTRIBUTION CODE: B001D COPIES RECEIVED:LTR 1 ENCL 1 SIZE: 56
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	IE FILE	1	1	IE/DEPER/EPB 36	3	3
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	NRR/DE/EHEB	1	1	NRR/DE/EQB 13	2	2
	NRR/DE/GB 28	2	2	NRR/DE/MEB 18	1	1
	NRR/DE/MTEB 17	1	1	NRR/DE/SAB 24	1	1
	NRR/DE/SGEB 25	1	1	NRR/DHFS/HFEB40	1	1
	NRR/DHFS/LQB 32	1	1	NRR/DHFS/PSRB	1	1
	NRR/DL/SSPB	1	0	NRR/DSI/AEB 26	1	1
	NRR/DSI/ASB	1	1	NRR/DSI/CPB 10	1	1
	NRR/DSI/CSB 09	1	1	NRR/DSI/ICSB 16	1	1
	NRR/DSI/METB 12	1	1	NRR/DSI/PSB 19	1	1
	NRR/DSI/RAB 22	1	1	NRR/DSI/RSB 23	1	1
	REG FILE 04	1	1	RGN5	3	3
	RM/DDAMI/MIB	1	0			
EXTERNAL:	ACRS 41	6	6	BNL(AMDTS ONLY)	1	1
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[illegible]

Arizona Public Service Company

ANPP-30652 EEVBJr/TFQ/NEM
September 26, 1984

Director of Nuclear Reactor Regulation
Attention: Mr. George Knighton, Chief
Licensing Branch No. 3
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Subject: Palo Verde Nuclear Generating Station (PVNGS)
Units 1, 2, and 3
Documents Relating to Water Leakage
from Temporary Lines
Docket Nos. STN 50-528/529/530
File: 84-056-026; G.1.01.10

Reference: Letter from G. Knighton (NRC) to E. E. Van Brunt, Jr. (APS) dated August 30, 1984: Request for Documents Relating to Water Leakage from Temporary Lines-Palo Verde.

Dear Mr. Knighton:

Attached is the requested information regarding "Documents Relating to Water Leakage from Temporary Lines."

Item number 5 is state incorrectly. It should read:
May 30 note (1 page) from Kent (Jones), of APS, not Ken (Schecter), Bechtel, to Steve (Shepherd), Bechtel, which transmits review comments (handwritten in margins) on Responses to NRC Requests for Information (45 pages of draft edition).

If you have any questions, please contact me.

Very truly yours,

E E Van Brunt/BSK

E. E. Van Brunt, Jr.
APS Vice President
Nuclear Production
ANPP Project Director

EEVBJr/NEM/mb
Attachments

cc: E. A. Licitra (w/a)
A. C. Gehr (w/a)

B001
1/1

bcc: D. B. Karner
W. F. Quinn
T. F. Quan
K. E. Jones
LCTS Coordinator
S. H. Shepherd
J. R. Bynum
D. B. Fasnacht
D. R. Canady
J. M. Allen

ANPP-30652

STATE OF ARIZONA)
) ss.
COUNTY OF MARICOPA)

I, A. Donald B. Karner, represent that I am Assistant Vice President of Arizona Public Service Company, that the foregoing document has been signed by me for Edwin E. Van Brunt, Jr., Vice President, Nuclear Production, on behalf of Arizona Public Service Company with full authority so to do, that I have read such document and know its contents, and that to the best of my knowledge and belief, the statements made therein are true.

Donald B. Karner

Sworn to before me this_____day of___________, 1984

Notary Public

My Commission Expires:

____

212-05.. rev. 3/80

'83 OCT 14 P3:31

NS DOC CENTER

(attachment to Item 1)

ARIZONA PUBLIC SERVICE COMPANY

COMPANY CORRESPONDENCE

ANPP-30652

DATE: October 13, 1983

TO: A. Carter Rogers
Sta. # 3003

FROM: Martha McKinley
Sta. # 1380
Ext. # 7024 Martha

SUBJECT: Mesa Tribune Inquiry

Max Jennings, editor of The Mesa Tribune, is asking questions about information provided to him by someone who has or now works at Palo Verde.

The event in question is the breaking of a water line under Unit #1. His source claims that nothing has been done about some apparent damage or erosion caused by excess water under the unit. According to Max's source, the weight of the reactor broke the water line under the building.

Max has been given some documentation including a report number (DER) 82-166, field boring logs U2-5-1 and U2-5-4 performed 2/22/82 through 3/1/82, photographs of water bubbling up from under Unit #1, Ertec test results on seive analysis, dry density, moisture content, void ratio, and percent saturation calculations.

Apparently, this documentation says something should have been done as a result of the water under Unit #1. Max admits he knows nothing about this sort of thing and would like an explanation.

Max has been told the lines under Unit #2 were plugged before a leak occured and the design of these lines under Unit #3 was changed before anything could happen. He is asking if this is true. He also wants to know what the Ertec tests revealed, any conclutions and any actions taken. He wants to know if the soi was recompacted, too.

Max would like to have a response hopefully the middle of next week. Any information you can undercover will be appreciated. Please call if you have questions or if I can clarify this with Max.

MM/LM

xc: Ed Van Brunt

212-03C
rev. 3/80

CC: [illegible] (attachment to item 2)

Martha — VE — we are in a solid position on this one in all respects. I do suggest that any and all interviews on it be done by VE — He is totally knowledgeable and has all background to be consistent —

ARIZONA PUBLIC SERVICE COMPANY

COMPANY CORRESPONDENCE

DATE: October 25, 1983

OCT 2[illegible]

TO: Dan Green
Sta. # 1386

DGM
10/27

FROM: Martha McKinley
Sta. # 1380
Ext. # 7024

Martha

SUBJECT: Mesa Tribune Inquiry

I met with Max Jennings, executive editor of The Mesa Tribune, last week when I delivered background material on the broken water lines at Palo Verde. Max was given copies of the final Deficiency Evaluation Report to the NRC and he has been loaned copies of Ertec and Bechtel studies done on this matter. The APS/NRC records on this are filed under DER #81-35 and #81-55.

Max would like to meet with Ed Van Brunt to discuss this matter further. Because both Max and Ed have been out of town recently, such a meeting hasn't been arranged to date.

Here's the information I obtained from Max last week. A former Ertec employee, concerned that soil has eroded from under the containment building of Unit #1, began stealing Ertec field logs in 1982. The logs stolen were those relating to DER 81-35. The employee didn't voice his concerns to anyone at Ertec, Bechtel, APS or the NRC. He left Ertec and went to work for another local firm.

The former employee decided to come forward with his concerns and stolen documents now after hearing about Palo Verde pump problems. Apparently, he thinks the soil erosion (from what he claims is Unit #1 containment building) has resulted in settlement which has created problems for the pumps and related components.

Max Jennings has copies of the stolen Ertec field reports. He has hired Lowry Associates of Sacramento to evaluate the documents. The documents are hand-written reports of Palo Verde soil analyses done in early 1982. Each document has a column for "remarks." Many of the remarks indicate soil has been disturbed. Lowry's review agrees that the stolen documents say that soil in fact has been disturbed and that soil density is below specifications.

Max Jennings has done some preliminary investigation into his source. Basically, the source came to Max with letters of

reference from Ertec. He did not leave the company under poor terms. He did, in fact, work for Ertec when he said he did.

The source doesn't claim Ertec has done any wrong. Rather, Bechtel has manipulated the data Ertec collected and dismissed any need to replace and compact eroded soil. The source says the problem is a correctable one although it will take time and money to correct.

When Max has a chance to meet with Ed Van Brunt, he'd like to discuss the following questions:

What does the Ertec report show?

How was any problem corrected?

How did water get into the containment pump if it came from water lines under the auxiliary and control buildings?

The stolen Ertec reports (and the Lowry review) indicate soil density is below specs.

Meanwhile, Max was to verify that the source was claiming erosion from under the containment building, not the auxiliary building as the APS report to the NRC states.

Both Max and Ed are due back in town Thursday morning. I will pursue making arrangements for a face-to-face meeting with them.

MMcK:eb

Attachments

(attachment 6)
(Item 3

Item 4:

Item 4 pertains to the issue of the nature of the hydraulic pathways which the leaks took prior to their discovery and the shutting off of the water in the lines. This issue is somewhat important because it has implications to the size of the area which may have been affected by the leaks. The question takes issue with the theory put forth in the DER that the leaks were confined primarily to narrow pathways along interfaces between the pipes and the soil and the walls and the soil prior to emerging from the seismic gap cover plate and from the sides of the slab in the dead spaces at both units. The question proposes an alternate hypothesis of leak movement which says that the water more or less uniformly spread through the soil in an ever incresing bulb centered about the leak until the backfill was all saturated and the water was exerting uplift on structures. Only at this point did the water emerge at the seimic gap. The question implies that this postulation is more logical than the one put forth in the DER and that the theory of the leak being confined to narrow pathways is not supported by the report or even justified for that matter.

After consultation with Walter Ferris on this issue we plan to provide a comprehensive answer to the question which will provide a logical technical basis for our conclusion that the leaks most likely were confined to narrow pathways, and that the alternate hypothesis of a spreading pressure bulb of water exerting uplift forces on structures is not as technically well-founded. The answer will discuss the fact that the water emerged from the pipe under significant pressure (70-100 psi) and would have logically immediately sought the path of least resistance which is known from experience to be along interfaces between the soil and the pipes and walls.

The reason for this is that compaction of the backfill is normally not as good around pipes and up against walls as it is away from these obstructions. Further, for the water to move through the body of the soil it would be fighting the rather low permeability of the soil. This backfill had a high percentage of fines content (on the order of 20% or so) and a correspondingly low permeability. It is true that given sufficient time the leaks under very low pressure would have effectively saturated all of the backfill in the area as the question suggests and there is no doubt that this process was under way when the leaks were discovered. Because of the high pressure, however, coupled with the low permeability of the backfill and the presence of the preferred pathways provided by the weak interfaces between soil and piping and between soil and walls, it is belived that the most logical leak mechanism involved rapid progression of the water along these interfaces leading to its emergence at the seismic gap and erosion of soil at this point. It is believed that this would have happened well in advance of the backfill becoming saturated in its totality and exerting uplift on structures.

As a part of the response to this question we will study both groundwater monitoring data as well as settlement data to see whether there is any indication of a rising water table in the vicinity of Units 1 and 2 during the time periods in question, and to see whether there is any indication of uplift on structures. It is anticipated that no such indications will be found.

As a final point, it is not clear why the postulated issue of uplift due to hydraulic pressure is technically relevant to the concern for loss of support for the Auxiliary Building which is the basis for the questions. The subject of uplift does not contribute to an understanding of the effects of soil loss in our view. accordingly, we do not plan to pursue this subject further than it technically merits.

(Attachment to Item 4)

INSTALLATION SPECIFICATION

FOR

EXCAVATION AND BACKFILL

FOR THE

ARIZONA PUBLIC SERVICE COMPANY

PALO VERDE NUCLEAR GENERATING STATION

UNITS 1, 2, AND 3

QUALITY CLASS Q

RECEIVED

SEP 10 1979

CONSTRUCTION
PVNGS

SPECIFICATION NUMBER 13-CM-300

JOB NUMBER 10407
BECHTEL POWER CORPORATION
NORWALK, CALIFORNIA

REV. NO.	DATE	REVISIONS
9	8-27-79	Revised Sections 10.2.7, 10.3.1, and 10.3.6.
		Incorporated SCNs 1585 (MOD), 1692, 1694(MOD), 1700, and 1782.
8	2-20-79	Revised Section 6.7; Incorporated SCNs 1257, 1489 (MOD), and
		1499 in Section 10
7	7-27-78	General Revision. Sections 2.0, 5.0, and 10.0.
6	4-27-78	Incorporated SCNs 874, 882, 903, and 918 Into Sections 10.1, 10.4,
		and 10.5
5	9-14-77	Incorporated SCN-483 in Section 4.0;
		Incorporated SCN-360 in Sections 6.4.4, 10.3.3.1 and 10.3.3.3;
		Incorporated SCN-418 in Section 10.3.2.4 and SCN-467 in Section 6.1.3
4	6-11-77	Revised Section 10
3	4-12-77	Incorporated SCN-191 in Section 6.4.5 and SCN-227 in
		Section 10.3.3.3; Revised Sections 10.3.3.1 and 10.3.3.2
2	1-26-77	Incorporated SCN's - 144, 145, 150, 161, 164
1	10-12-76	Incorporated SCN 24 and SCN 58; Revised Backfill Material Gradations
0	5-21-76	Issued for Construction

PF-875 (10407) 1/77

CONTENTS

9.2.2 Placement

9.2.2.1 Filter Material

Filter material shall be placed immediately following completion of the embankment. The subgrade upon which the filter material is to be placed shall not vary from the planned slope by more than 1 foot measured at right angles to the slope. The placing methods shall be such as to prevent segregation of the material.

9.2.2.2 Riprap

9.2.2.2.1 A footing trench shall be excavated along the toe of the slope. Rocks shall be so placed as to provide a minimum of voids and the larger rocks shall be placed in the toe course and on the outside surface of the slope protection. The rock may be placed by dumping and may be spread in layers by bulldozers or other suitable equipment. Local surface irregularities of the slope protection shall not vary from the planned slope by more than a tolerance of plus 0.50 foot except that the extreme of such tolerance shall not be continuous over an area greater than 200 square feet.

9.2.2.2.2 At the completion of the work, the footing trench shall be filled with excavated material and compaction will not be required.

10.0 BACKFILL

10.1 General

10.1.1 This section includes the material, moisture control, placement, compaction and testing requirements for the various classes of backfill.

10.1.2 Depending on the functional and engineering requirements, the backfill will be classified as follows:

a. Structural Backfill, Class 1 (also called Category 1
7 Structural Backfill)

   This backfill is that fill placed under and around Category 1 structures including Category 1 pipelines and as otherwise shown on the drawings. This material is further defined in 10.2.2.

b. Structural Backfill, Class 2

   This backfill is that fill placed under and around non-Category 1 structures and as otherwise shown on the drawings. This material is further defined in 10.2.3.

c. Embankment Fill

   This backfill is any fill required to raise the existing grade to the required finished grade elevation outside the limits of the power block structures. Further definition is given in 10.2.4.

d. Backfill for Underground Utilities

This backfill is any fill placed under or around buried utilities other than Category 1 pipelines. This definition includes concrete encased electrical ductbanks in utility corridors. Further definition is given in 10.2.5.

e. Category 1 Ductbank Material

This material shall consist of sand as required to encase the electrical ducts. Further definition is given in 10.2.6.

10.2 Material Requirements

10.2.1 General

10.2.1.1 Backfill materials may be obtained from power block excavations or from other approved borrow areas. Designation and approval of a borrow area does not mean that all materials within that area are suitable for backfill. Only suitable material from approved borrow sources shall be placed in the backfill. Material containing brush, roots, peat, sod or other organic, perishable or deleterious matter, snow, ice or frozen soil, shall not be placed in the backfill. If unsuitable material is placed in any part of the backfill, all such material shall be removed and replaced with suitable material.

10.2.1.2 Materials suitable for structural backfill may be separately stockpiled for later use. If stockpiling is done prior to backfilling, gradation testing in accordance with the requirements of 10.4.7 should be performed on the material prior to placing in the stockpile in order to determine its suitability for structural backfill. All materials from stockpiles which have not been prequalified by gradation testing and from other potential borrow sources shall be tested to establish their suitability for backfill in accordance with the requirements of 10.4.7. Only those materials meeting the specified quality requirements shall be used as backfill. 7

10.2.1.3 Materials shall be moisture conditioned as far as practicable in the stockpile areas by sprinkling, aerating, harrowing, discing, draining or other approved means in order to obtain uniform moisture distribution such that the specified density may be obtained. Sprinkling shall be by sprinkler trucks equipped with pressure spray bars and valves to give a uniform and even application of water to the dry areas and a positive control of the rate of water application at all times.

10.2.1.4 Any section of backfill containing material which is too wet or too dry shall not be compacted until the moisture content meets the limits necessary to achieve the specified density or the material shall be removed and replaced with material having a moisture content within acceptable limits.

10.2.1.5 Should, in the opinion of the Field Engineer, any portion of the surface of the backfill become so dry or glazed during construction that bond with the succeeding layer to be placed thereon cannot be obtained or should ruts and roadways develop on the backfill, such surfaces shall be scarified to a minimum depth of 6 inches, releveled, moisture conditioned, and recompacted to the specified density just prior to placing of the succeeding layers.

10.2.1.6 Extreme care should be exercised in placing and compacting backfill in the proximity of all structures. Heavy construction equipment shall not pass over any permanent plant structure or pipe until such structures and/or pipes are covered by the applicable minimum depth of fill shown on the drawings or as specified.

10.2.1.7 During backfill compaction heavy construction equipment shall not be used within a distance of 2 feet (plus or minus) from any concrete structures or walls (except electrical ductbank). The compaction of backfill adjacent to concrete structures or walls (except electrical ductbank) within a distance of 2 feet (plus or minus) and in other restricted areas shall be done using hand-operated vibratory compactors and/or power tampers.

7

10.2.1.8 All classes of backfill shall conform to the material and compaction requirements specified below for the respective class of backfill. The backfill shall be placed and compacted to the elevations and limits shown on the drawings.

10.2.2 Structural Backfill, Class 1

In addition to the general material requirements, this class of backfill shall be well graded and dense, and shall consist of sound, durable material from a designated borrow area or required excavation and shall conform to the following gradation limits:

U.S. Sieve Number or Opening Size	Percent Passing by Weight
1-1/2 inch	80 to 100
No. 4	60 to 100
No. 10	50 to 100
No. 40	20 to 90
No. 100	0 to 60
No. 200	0 to 30

The maximum size of the material shall be 4 inches in confined areas where hand tamping is required and no greater than 2/3 the uncompacted lift thickness in other areas. The material shall not contain soil lumps which will not break down and compact satisfactorily when rolled. The backfill gradation shall be such that each zone is free of pockets and layers or streaks of poorly graded material. Clay layers encountered in the excavation shall not be used in structural backfill.

10.2.3 Structural Backfill, Class 2

This class of backfill shall meet the requirements of Structural Backfill, Class 1, except that the maximum limit of fines passing No. 200 U.S. sieve is 40 percent.

10.2.4 Embankment Fill

All onsite soils, provided they conform to the general material requirements specified, will be suitable for use as embankment fill. In addition the embankment materials shall conform to the following: No rock over 2/3 the loose lift thickness will be permitted in the top lift of the embankment. No rock greater than the loose lift thickness will be permitted in other portions of the embankment.

10.2.5 Backfill for Underground Utilities

All onsite soils will be suitable for the backfill for underground utilities provided they conform to the general material requirements specified. Backfill within 6 inches of the underground utilities shall be free from rocks, hard lumps, and clods greater than 3 inches in maximum dimension.

10.2.6 Category 1 Ductbank Material

Electrical duct sand encasement shall conform to the following gradation limits:

Opening Size or U.S. Sieve Number	Percent Passing by Weight
3/8 inch	100
No. 200	0 to 10

10.2.7 Cementitious Aggregate Concrete

Cementitious aggregate concrete backfill may be used in lieu of embankment fill. This may also be used in lieu of structural backfill Class 2 in non-load bearing applications or backfill for non-Category 1 pipes and ducts for underground utilities with the approval of the Engineer. This shall be used

in such a way as [illegible] pport the entire structural [illegible] m uniformly. A structural system shall not be supported partly on CAC backfill and partly on embankment fill. The mix to be used for CAC backfill, shall be as given on Concrete Placement Drawing 13-C-00A-061.

5

A minimum 12-inch cover of embankment fill shall be provided over CAC backfill. The covering fill shall be placed at least 8 hours after placing CAC backfill. Curing of CAC backfill is not required.

10.3 Placement Requirements

10.3.1 General

10.3.1.1 After the completion of footings and walls, and prior to placement of backfill, all forms shall be removed and the excavation cleaned of all trash, debris, and unsuitable material. Before placing backfill material, the soil surface shall be scarified to a depth of at least 6 inches, and compacted to the specified density for the class of backfill being placed.

7 Exception: In restricted areas, such as along electrical ductbanks, in pipe trenches, and as referenced in 10.3.2.1, density tests are allowed to determine if the existing in situ conditions exhibit competent bearing. If conditions do not meet the specified densities, the in situ materials shall be reworked as required to achieve the specified densities.

10.3.1.2 The surface of the backfill shall be maintained in such condition that construction equipment can travel on any part of the backfill. Unless otherwise approved by the Field Engineer the backfill shall be raised uniformly in a horizontal plane. The differential elevation between adjacent zones shall not exceed two feet unless the edge slope of the backfill is one horizontal to one vertical or flatter.

10.3.1.3 No backfilling against structural concrete shall be done until the concrete has attained a strength equal to 70 percent of the design strength. Backfilling against structural concrete such as pipe anchor blocks and mass concrete foundations may be done as soon as the forms are removed, but in no case, sooner than 24 hours after completion of the concrete placement. If the subgrade structural concrete has been waterproofed, the backfilling shall be done so as not to damage the waterproofing.

10.3.1.4 . Nonstructural concrete, lean concrete backfill, electrical ductbank encasement, curbs, gutters, catch basins, and ditch lining may be backfilled as soon as forms are removed, but in no case earlier than 24 hours after completion of placement, except for lean concrete backfill which may be backfilled 5 hours after the concrete has been placed.

10.3.2 Placement of Structural Backfill, Class 1

The general conditions of 10.3.1 shall apply to this class of backfill. This material shall be placed in successive horizontal layers and shall be compacted to not less than 95 percent of the maximum dry density in accordance with ASTM D 1557..

Additionally, test fills will be constructed in accordance with Specification 13-CM-301, Test Fills for Category I Structural Backfills. The purpose of the test fill program is to demonstrate that the type of compaction equipment, number of equipment passes, soil layer thickness, and the moisture control used will result in the required compaction for the structural backfills. Test Fill II Program report data with appendices A, B, and C are available in the jobsite files. Test fills shall not be required for hand-operated equipment. Compaction equipment and procedures meeting the specification requirements, listed below, shall be incorporated in the final construction work for achieving the specified compaction. The soils testing laboratory will perform all tests in accordance with 10.4.

SUMMARY OF TEST FILL PROCEDURES MEETING

STRUCTURAL BACKFILL, CLASS 1 COMPACTION CRITERIA

Criteria No.	Roller Type	Maximum Uncompacted Thickness of Each Lift	Minimum Number of Roller Passes For Each Lift	Operating Speed of Roller
1	Ingersoll-Rand SPF-54* or	4 inch	6	2 mi/h
	Buffalo Bomag BW-210	4 inch	6	2 mi/h
2	Ingersoll-Rand TDF 30* or	6 inch	8	2 mi/h
	RayGo Rascal 420-C	6 inch	4	1 mi/h
3	RayGo Rascal 420-C	9 inch	6	1 mi/h
4	Dynapac CA-25PD	9 inch	6	1 mi/h

*Note: These two pieces of equipment produced marginal results during the test fill studies. Special care should be taken when using this equipment for compaction.

10.3.2.1 Backfill in Restricted Areas

Backfill in restricted areas, for the purposes of this specification, is defined as backfill below elevation 51'-6" around the Auxiliary Building, backfill in the Containment Building between the reactor pit walls and tendon gallery walls, and backfill in areas, as determined by the Field Engineer, too restricted to allow operation of heavy compaction equipment. In areas too restricted to allow operation of the specified roller types, the backfill shall be compacted in lifts with uncompacted thickness no greater than 6 inches using hand-operated or light power compaction equipment.

10.3.2.2 Backfill in Unrestricted Areas

For the purpose of this specification, backfill in unrestricted areas is defined as all backfill not covered in 10.3.2.1. Backfill in these areas shall be compacted using heavy compaction equipment and as covered in 10.3.2.

10.3.3 Placement of Structural Backfill, Class 2

The placement criteria for Structural Backfill, Class 1, as described in 10.3.2 shall be used as guidance for this class of backfill to achieve the specified compaction.

7 10.3.4 Placement of Embankment Fill

Embankment material shall be placed in successive horizontal layers and shall be compacted to not less than 90 percent of maximum dry density as determined by ASTM D 1557, unless otherwise shown on the engineering drawings.

10.3.5 Placement of Backfill for Underground Utilities

10.3.5.1 Bedding requirements are given in 13-CM-335, "Underground Utilities." Backfilling of pipe trenches shall not be completed until the entire length of pipe being backfilled has been inspected, tested if required, and approved by the Field Engineer. Backfill shall be placed in layers of uniform thickness, and compacted by rolling or tamping. The lift thickness of loose backfill material placed shall not exceed 12 inches, and shall be as required by the Field Engineer considering the type of backfill material, compaction equipment and type of pipe. This method of filling and compacting shall be continued until the backfill is 12 inches over the top of the pipe. At no time during backfilling operations shall the differential elevation of the top of backfill on opposite sides of a pipe exceed 1 foot.

10.3.5.2 Each layer of backfill up to a level 1 foot above the top of the pipe shall be compacted to a density not less than 90 percent of the maximum dry density as determined by ASTM D 1557.

10.3.2.1 Backfill in Restricted Areas

Backfill in restricted areas, for the purposes of this specification, is defined as backfill below elevation 51'-6" around the Auxiliary Building, backfill in the Containment Building between the reactor pit walls and tendon gallery walls, and backfill in areas, as determined by the Field Engineer, too restricted to allow operation of heavy compaction equipment. In areas too restricted to allow operation of the specified roller types, the backfill shall be compacted in lifts with uncompacted thickness no greater than 6 inches using hand-operated or light power compaction equipment.

10.3.2.2 Backfill in Unrestricted Areas

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10.3.3 Placement of Structural Backfill, Class 2

The placement criteria for Structural Backfill, Class 1, as described in 10.3.2 shall be used as guidance for this class of backfill to achieve the specified compaction.

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Embankment material shall be placed in successive horizontal layers and shall be compacted to not less than 90 percent of maximum dry density as determined by ASTM D 1557, unless otherwise shown on the engineering drawings.

10.3.5 Placement of Backfill for Underground Utilities

10.3.5.1 Bedding requirements are given in 13-CM-335, "Underground Utilities." Backfilling of pipe trenches shall not be completed until the entire length of pipe being backfilled has been inspected, tested if required, and approved by the Field Engineer. Backfill shall be placed in layers of uniform thickness, and compacted by rolling or tamping. The lift thickness of loose backfill material placed shall not exceed 12 inches, and shall be as required by the Field Engineer considering the type of backfill material, compaction equipment and type of pipe. This method of filling and compacting shall be continued until the backfill is 12 inches over the top of the pipe. At no time during backfilling operations shall the differential elevation of the top of backfill on opposite sides of a pipe exceed 1 foot.

10.3.5.2 Each layer of backfill up to a level 1 foot above the top of the pipe shall be compacted to a density not less than 90 percent of the maximum dry density as determined by ASTM D 1557.

10.3.5.3 Backfill shall be placed to a minimum depth of 3 feet above the top of pipes or structures before power operated heavy hauling or rolling equipment is operated over the pipe or structure, except that equipment not exceeding a total weight of 5,000 pounds is permitted after backfill has been placed and compacted to a minimum depth of 1 foot above the top of pipe.

10.3.5.4 All backfill more than one foot above the top of the pipe, shall be placed in 8 inch loose lifts along the trench, and compacted to not less than 85 percent of maximum dry density as determined by ASTM D 1557. The backfill material shall be brought up above the adjacent rough grade in the form of a mound of sufficient height, to allow for settlement of the trench material.

10.3.5.5 Compaction of trench backfill by ponding and jetting will not be permitted.

10.3.5.6 Pipe trenches shall be backfilled as soon as possible after the pipe has been installed; provided, however, that no backfilling shall be performed until trenching, pipe bedding and installation, and testing, if required, have been approved by the Field Engineer.

10.3.5.7 Backfilling of trenches for temporary construction services in areas other than power block structures or Category I structures, areas not under roads or other permanent plant items, are exempt from the compaction density requirements of this section. These areas may be backfilled as directed by the Field Engineer.

10.3.6 Placement of Category 1 Ductbank Material

Sand shall be placed around the ducts by dumping and shall be wetted and rodded as necessary to prevent bridging. The sand shall be compacted by vibrating in place, to a density not less than 95 percent of the maximum dry density as determined by ASTM D 1557. Sand shall be added as required during compaction to fill depression caused by vibration or as required by the Field Engineer.

Final ductbank configuration shall conform to the ductbank details shown on the electrical drawings. Backfill to grade above ductbank shall conform to applicable material requirements of the area where the ductbank is located, (i. e., structural backfill Class 1 in Category 1 areas or backfill for underground utilities in yard areas).

10.4 Testing

This section includes the testing requirements for Structural Backfill, Class 1 and 2, Embankment Fill, and Backfill for Underground Utilities. The frequency of tests is outlined in 10.4.6.

10.4.1 A qualified soils testing laboratory shall perform all tests on compacted materials to assure compliance with these specifications. The

soils testing laboratory will conduct field density and other tests and the related laboratory compaction testing to determine the relative degree of compaction and other properties. At the direction of the Field Engineer and concurrent with construction, the soils testing laboratory will take representative samples of the material from the borrow areas, stockpiles or areas of backfill. These samples will be tested as required to determine the acceptability of backfill quality and degree of compaction including any control or record tests which may be required.

10.4.2 Representative samples of the backfill shall be tested in the laboratory in accordance with ASTM D 1557 in order to determine the maximum dry density and optimum moisture content applicable to the backfill material being used.

10.4.3 Moisture content determination shall be made in accordance with ASTM D 2216.

10.4.4 During backfilling, the field compaction shall be monitored by performing field density testing in accordance with ASTM D 1556. Compaction equipment must be stopped during the time the sand is being poured into the hole during these field density tests so as to prevent its densification by vibration. All field density tests should be made at a depth of one foot below the surface of the compacted lift.

The relative degree of backfill compaction shall be determined by dividing
7 the field dry density by the maximum dry density determined by laboratory testing for the backfill material being tested and the result shall be expressed as a percentage.

10.4.5 The backfill quality shall be determined by performing gradation test in accordance with ASTM D 422 using the wet sieving procedure.

10.4.6 The minimum frequency of testing required to monitor the backfill compaction and quality for the various classes of backfill is given in table 1.

10.4.7 Unless Structural Backfill, Class 1 or 2 is obtained from prequalified stockpiles, gradation testing specified below will be in addition to any other gradation tests that are performed in conjunction with laboratory or field testing.

If material is obtained directly from excavations in potential borrow areas or from stockpiles which have not been prequalified, periodic gradation testing on representative soil samples in accordance with ASTM D 422 using wet sieving procedure is required to ensure that only materials conforming to the specified gradation limits for the structural backfills are used. This testing should be performed prior to hauling the materials to the backfilling area. A minimum of one gradation test in accordance with ASTM D 422 using the wet sieving procedure for every 1000 yd^3 of potential fill material is required to monitor the quality of backfill material.

If nonuniform soil conditions exist in the borrow areas or unqualified stockpiles, more gradation tests will be required to establish the material suitability. The extent of additional testing shall be determined by the Field Engineer who is monitoring the quality of backfill materials and responsible for the excavation work.

10.4.8 All test report results, for both density and gradation, shall be in whole numbers. This is not meant to reduce the accuracy of calculations, which may be carried to any necessary degree of accuracy. It is necessary that any reported value, to be compared with values stated in this specification, be on the same basis.

Table 1

MINIMUM FREQUENCY OF TESTING

	Density Test		
	1	2	3
Class of Backfill	Field Density Test ASTM D 1556	Laboratory Compaction Test ASTM D 1557	Gradation Test, ASTM D 442 (Wet Sieving Procedure)
a. Structural Backfill Class 1 and 2			
1. Unrestricted Areas	One test for every 20,000 ft^2 of fill placed per foot of depth or one test each shift, whichever is the greater number.*	One test for every 5000 yd^3 of fill fill placed and compacted or one test per day, whichever is the greater number.	One test for every field density or laboratory compaction test listed in columns 1 and 2.
2. Restricted Areas	One test for every 200 yd^3 of fill placed or one test each shift, whichever is the greater number.*	Same as above.	Same as above.
b. Embankment Fill	One test for every 2000 yd^3 of fill placed or one test each shift, whichever is the greater number.	One test for every 10,000 yd^3 of same type of fill placed or as required by the Field Engineer.	None.
c. Back fill for Underground Utilities	One test for each 500 feet of utility corridor or on test each shift, whichever is the greater number.	One test for every 5000 yd^3 of same type of fill placed or as required by the Field Engineer.	None.

* Additional density tests can be taken at option of the Field Engineer [illegible]

ATTACHMENT A

QUALITY VERIFICATION DOCUMENT REQUIREMENTS

The following documents are required to be filed under this specification:

Specification Paragraph	Required Document
6.1.1	Excavation Inspection and Verification Reports
10.4	Material and Compaction Tests and Verification Reports

(attachment to Item 5)

Steve,

These are comment on 87-35 from Joe Kanovich and EE [illegible]

If you have any question call me

Later

Ken guess what showed up. Steve

Kent
5/30

RESPONSES TO NRC
REQUESTS FOR INFORMATION

QUESTION 1

Explain the basis for locating temporary water and other utility lines under Category I structures. Provide a drawing showing locations of these lines.

RESPONSE: The deep portion of the Auxiliary Building was one of the first areas to be constructed in the sequence of work owing to its position as the deepest structure in the power block. As the walls of the deep portion rose, it was necessary to provide construction utilities inside the building so that interior work could proceed. These utilities included domestic service water, construction service water, compressed air, oxygen, argon, and MAPP gas. It was also required to provide fire protection to the interior of the structure.

This description needs to be made much clearer

I think we need to make clear that it was not possible to get these services into the lower area of Aux Bldg without going under some class I structure slab.

These mandatory utilities were routed in such a way to avoid, as far as possible, being later overlain by safety-related structures constructed at higher elevations. At Unit 1, the Auxiliary Building wing at elevation 70 feet necessarily overlaid a short segment of the fire protection line and for Unit 2, all of the utilities, because they were required to penetrate the deeper basement wall underneath the wing. At Units 2 and 3, the utilities run under the north edge of the Control and Radwaste Buildings. Six construction drawings (figures 1-1 through 1-6) are submitted with this response to illustrate the temporary utilities which were installed and their routing at all three units. Figures 1-1 and 1-2 show the temporary utilities for Unit 1, figures 1-3 and 1-4 for Unit 2, and figures 1-5 and 1-6 for Unit 3.

why difference for the 3 Units?

This may not be a good argument since we could have come down from above into this area.

After installation of the temporary utilities, they were covered with structural backfill so as not to interfere with construction access in the congested areas around the outside of the Auxiliary Building. Compaction procedures around the pipes are discussed in the response to Question 2.

Does this mean where it enters the Category I structure or when it goes under a category I structure?

QUESTION 2

Describe the compaction procedures for the backfill and the subgrade that were used around these utilities. Provide photographs of the bearing surface of the backfill before the mudmat or foundation was placed.

I don't see that we responded to this request

RESPONSE: The backfill material placed around the temporary utilities in the vicinity of Category I structures was Category I structural backfill. This backfill material was placed per Section 10.0 of Specification 13-CM-300 (see Attachment A). Paragraph 10.3.1.1 describes the preparation of the subgrade before the backfill is placed. Figures 2-1 through 2-6 illustrate the conditions of the bearing surfaces and compaction procedures.

The following construction sequence was used for installation of the temporary pipelines under the wing of the Auxiliary Building and under the slab in the dead space:

Doesn't this apply wherever it goes under a category I structure?

The deep portion of the Auxiliary Building up to 70-foot elevation was constructed first. Then the Category I structural backfill was placed around the Auxiliary Building walls until the bottom elevation for the temporary pipes was reached. Then the temporary pipes were installed (see figure 2-7). Finally, the Category I structural backfill was placed until the elevation for the mudmat under the Auxiliary Building wing was reached. These temporary pipelines under the wing of the Auxiliary Building and under the slab in the dead space were not installed in trenches cut in the backfill.

Was sand put around some of the pipe to allow for differential settlement?

QUESTION 3

Explain the mechanism by which these leaks might have been caused and the sequence of the events that led to the pipe leaks. Describe the extent of

leakage and show the affected zone(s), using drawings, with an explanation of the basis for your conclusions.

RESPONSE: Possible mechanisms causing the observed pipe leakage include corrosion, settlement, and faulty welding of joints or a combination of these. The site has had a history of corrosion problems and this is considered to be a leading factor in view of the lengthy period of time during which the pipes were buried prior to the leaks developing. Settlement of structures may also have been a contributing factor. As construction progressed around and above the buried pipes and as the load on the backfill increased, settlement of the structures, although small, could have stressed the pipes at joints, elbows, etc., such that they could have developed breaks. This failure mode would be enhanced ~~is all the more likely~~ if corrosion had already locally weakened the pipes. Faulty welding is a possible contributing factor although this mechanism is considered less likely than corrosion and settlement. The pipes were temporary only, and were not installed to the more exacting standards of permanent piping. The most likely scenario is a combination of local corrosion due to the soil environment around the pipes and stressing of the pipes at critical points as a result of loading of the structures in the vicinity of the pipes.

were there any records on welds

weak arguement considering -- they were installed in category I fill and what happened.

The following sections describe the leakage and affected zones in each unit:

Unit 1

Do we need to say briefly what this is? - I think we need a brief description or at least a reference to DER on how we did this investigat

In the Unit 1 fire line there were two leaks, as discussed in the Deficiency Evaluation Report No. 81-35 (Reference 1). Packer test data showed that the leak at or beyond the upper elbow (refer to figures 3-1 and 3-2) was significantly larger than the leak in the side branch. Packer test data is presented in appendix A to reference 1 and is also presented with this response. The relative leak sizes are based on pressure bleed down rates during the packer tests.

Because of the proximity of the larger leak to the seismic gap cover plate where the washed-out soil was found, this leak is believed to be

responsible for all of the observed soil transport. According to NCR CY2909 (appendix D to reference 1) the leak rate was observed to be 10 gal/h when the leak was first discovered at 12:00 a.m. on September 5, 1981. The leak rate had increased to between 200-250 gal/h by September 6, at which time water to all the temporary utility lines was turned off. Shortly thereafter, it was determined that the source of the water was the temporary fire line. reference?

Because the leak is in close proximity to the exit point at the seismic gap cover plate, it is reasonable to assume that most of the water flowed along this path; therefore, 250 gal/h is a reasonably conservative estimate of the maximum leak rate for this leak. For a 4-inch diameter pipe with a service pressure of from 70-100 psi, this suggests that the size of the break was quite small. If substantial additional leakage had followed another path, this should have been observed at additional exit points sometime during the period September 5 through September 6. No such observations were reported.

wouldn't it be useful to state what the maximum leak rate would be from a 4" diam. line with a guillotine break to put this in perspective?

The fact that when the leak was first discovered on September 5 the rate was only 10 gal/h, which rapidly increased to 200-250 gal/h, indicates that the leak probably developed shortly before it was discovered. If the line had been leaking over a longer period of time, more of a steady-state rate of leakage would have been observed. The leak was apparently in the process of developing when it was discovered and shut off. If one conservatively assumes that the leak started a full day prior to being discovered and that the effective leak rate was 250 gal/h over the entire period of September 4 through September 6, then the total volume of leakage would have been on the order of 20,000 gallons. The vast majority of this went into the Control Building sumps through drains in the dead space slab and was pumped out. At one point the Control Building base slab was covered to a depth of 12 inches prior to pumping. ¶ The mechanism for this leak in Unit 1 appears to have been a combination of corrosion and settlement. Corrosion may have weakened a joint to the point that a small leak occurred due to stress concentrations from the surcharge of overlying structures.

Couldn't we estimate the leak rate by calculating the volume of water we found in Bldg.?

leakage and show the affected zone(s), using drawings, with an explanation of the basis for your conclusions.

RESPONSE: Possible mechanisms causing the observed pipe leakage include corrosion, settlement, and faulty welding of joints or a combination of these. The site has had a history of corrosion problems and this is considered to be a leading factor in view of the lengthy period of time during which the pipes were buried prior to the leaks developing. Settlement of structures may also have been a contributing factor. As construction progressed around and above the buried pipes and as the load on the backfill increased, settlement of the structures, although small, could have stressed the pipes at joints, elbows, etc., such that they could have developed breaks. This failure mode would be enhanced ~~is all the more likely~~ if corrosion had already locally weakened the pipes. Faulty welding is a possible contributing factor although this mechanism is considered less likely than corrosion and settlement. The pipes were temporary only, and were not installed to the more exacting standards of permanent piping. The most likely scenario is a combination of local corrosion due to the soil environment around the pipes and stressing of the pipes at critical points as a result of loading of the structures in the vicinity of the pipes.

were there any records on welds

weak arguement considering they were installed in category I fill and what happened.

‡ The following sections describe the leakage and affected zones in each unit:

Unit 1

Do we need to say briefly what this is? - I think we need a brief description or at least a reference to DER on how we did this investigation

In the Unit 1 fire line there were two leaks, as discussed in the Deficiency Evaluation Report No. 81-35 (Reference 1). Packer test data showed that the leak at or beyond the upper elbow (refer to figures 3-1 and 3-2) was significantly larger than the leak in the side branch. Packer test data is presented in appendix A to reference 1 and is also presented with this response. The relative leak sizes are based on pressure bleed down rates during the packer tests.

Because of the proximity of the larger leak to the seismic gap cover plate where the washed-out soil was found, this leak is believed to be

The second and smaller leak in the side branch of the fire line beneath the dead space was very small based on pressure data. The packer tests indicated pressure drops of approximately 6-7 psi in one minute using air (refer to test data submitted with this response). This would correspond to very small openings in the pipe. This type of leak suggests corrosion as a primary mechanism and the pipe could have been leaking at a slow rate for a longer period of time without being detected. It is not believed that this leak contributed to the soil eroded in the vicinity of the seismic gap cover plate. The soil samples obtained from beneath the dead space (refer to the Ertec report - attachment A to reference 1) showed that the general area of this leak had been saturated. No estimate of the total volume of leakage from this source is possible.

(described in ______

The locations of the leaks at Unit 1 are shown on figures 3-1 and 3-2. The affected zones include an area of soil washout in the vicinity of the seismic gap cover plate related to the leak at or beyond the upper elbow, and a general area of transient saturation with localized soft zones beneath the dead space (refer to Ertec report - attachment A to reference 1) and probably extending slightly underneath the wing of the Auxiliary Building.

The size of the zone affected by soil washout from the seismic gap can be estimated by working backwards from the reported 2 to 3 cubic yards of muck removed from the dead space slab to a corresponding void size in the compacted backfill behind the cover plate. The size of the void would be substantially smaller than the volume of muck due to the loose state and high water content of the muck as compared with compacted backfill. Density test data given in the Ertec report (attachment A to reference 1) indicate that the average relative compaction in the area was 98% of the maximum dry density per ASTM D-1557. The corresponding void ratio was approximately 0.35. If one assumes the material was redeposited in the dead space at 75% relative compaction, the corresponding void ratio would be approximately 0.76. By comparing the void ratios one may calculate that 3 cubic yards of muck would translate

into 2.3 cubic yards of compacted backfill removed. Two cubic yards of muck would translate into 1.5 cubic yards of compacted backfill removed. The total volume of void space created by the soil washout was therefore on the order of 1.5 to 2.3 cubic yards. It should be noted that other estimates of the muck were much less than 2 to 3 cubic yards. For example, one estimate is based on the removed muck reportedly filling one and one-half 55 gallon drums. This would be less than 1 cubic yard, total. As the nonconformance report cites 2 to 3 cubic yards, the DER uses 2 to 3 cubic yards; however, it is believed to be very conservative.

Don't we need a reference for this?

The zone affected by the smaller leak in the side branch has been conservatively bounded for analytical purposes in the absence of direct data. As explained in the response to Question 9, a zone beneath the Auxiliary Building wing measuring 20 by 35 feet in plan view was assumed to experience a total loss of soil support for purposes of reanalyzing the structural integrity of the Auxiliary Building. Although this zone may have experienced a transient saturation condition it certainly has not experienced a total loss of soil support; therefore, the analysis has accounted for the absolute worst-case condition.

The extent of area subjected to saturation by the leaks is not of concern because without other disturbing factors, such as soil erosion for example, the properties of the backfill in the saturated condition are more than adequate to support the structures under both static and dynamic loading. This has been established by extensive laboratory testing, the results of which are presented in the PSAR. This is discussed further in the response to Question 9c.

Unit 2

At Unit 2, the two-inch diameter domestic service water (DS) line, the construction air (CA) line, and the fire protection line developed leaks. As discussed in reference 1, the most significant leak from the standpoint of soil transport was that in the DS line. The location of this leak was determined by packer testing to be a welded joint at the

"tee" fitting just outside of the Auxiliary Building basement wall penetration (refer to figures 3-3 and 3-4). This leak would hold no air pressure whatsoever in the packer tests (see attached test data). Because of its location just outside the Auxiliary Building basement wall, this leak could be visibly observed from inside the building by looking into the pipe from its cut-off end just inside the wall. The leak was observed to be a knife-edge break transverse to the pipe in the lower portion of the welded joint. The width of the break could not be measured directly but was sufficient to allow coarse sand particles to enter the pipe during backflow following a packer test.

The mechanism for this break could have been faulty welding at the joint, settlement, corrosion, or a combination of these factors. Because the break was at a weld, the quality of the weld is called into question; however, settlement of the Auxiliary Building and possible weakening of the weld by corrosion are likely to have contributed to the break.

It is interesting to note that in this case a partial break of a 2-inch line produced immediate pressure drops to 0 psi in the packer tests. This serves to bound the size of other leaks which tested at a slower rate of pressure drop.

It isn't clear to me what area this is. Is this in the seismic gap area

The rate of water leakage for the DS line is not as well documented as for the Unit 1 fire line. When discovered, the leak had flooded the slab in the dead space to a depth of 13 feet with the level rising. After shut-off, the water was observed to be turbulent. After shutting off the construction air line, the turbulence ceased. No further information is available; however, it seems likely that the volume of water involved in this leak is similar to that involved in the Unit 1 fire line leak. The reason that the water level rose in the dead space at Unit 2 was because the drains in the dead space slab were plugged. Had this not been the case, water would have flowed to the Control Building sumps as at Unit 1.

The second water leak at Unit 2 was in the side branch of the fire line as discussed in reference 1. Packer testing of this leak yielded a bleed down rate of approximately 6 psi over a 2-minute period. This leak rate is insignificant as compared with that in the DS line and it is believed that this leak did not contribute to any soil erosion. The mechanism which caused this leak is most likely corrosion.

significant

The CA line was not packer tested because it was not felt that an air leak would have contributed to soil erosion.

The quantity of muck removed from the dead space at Unit 2 was estimated to be approximately 3 cubic yards. This would relate to an in-place void volume similar to that of Unit 1, or about 2.3 cubic yards.

How do we know this?

Unit 3

The leak at Unit 3 was different from those in Units 1 and 2 in that no soil was washed out. Further, the different layout of the lines (refer to figures 1-5 and 1-6) precluded packer testing because of the presence of elbows in the pipe at all access points. Therefore, detailed information on leak mechanisms at this unit is not available. It is likely that the mechanism or mechanisms involved were similar to those at Units 1 and 2. Figure 3-5 shows the likely location of this leak.

I think this is a very weak description. What line leaked in Unit 3?

Also indicate an area of seepage. This doesn't seem to square with 1st section of this [illegible]

QUESTION 4

In Section 5.4 of Reference 1, you have suggested that, in all probability, the leaks caused quite narrow paths within the backfill along pipes and walls before finding an exit. You did not provide any logical basis for this hypothesis. Explain why it is not more probable that the pipe leaks caused saturation and uplift on almost all the backfill before seepage exited in the seismic gap. Results of your limited excess grout take during grouting of the pipelines tend to support the idea that erosion of soil may not be concentrated in the area of the pipe leaks. I don't think we answered this part of question

RESPONSE: In Section 5.4 of the report a probable scenario of leak development was presented which involves preferred pathways of leakage beneath the Auxiliary Building which were most likely quite narrow. The logical basis for this scenario is provided by the following:

- The leaking lines were pressurized.
- The backfill is tightly compacted, and has relatively low permeability.
- The interfaces between pipes and soil, and walls and soil, are zones of weakness which would have been very rapidly exploited by water under pressure seeking an exit path.
- Once an exit path had been established most of the leakage would have followed this path in preference to entering the surrounding tight, low permeability backfill.

This does not preclude the possibility of saturation of the backfill in the general area of the leaks; in fact, soil samples obtained beneath the dead spaces between the Auxiliary and Control Buildings at both Units 1 and 2 indicate generally saturated conditions in these areas. The extent of saturated areas is not of concern as discussed in the

responses to Questions 3 and 9c because test data has shown that the undisturbed backfill has more than adequate strength and compressibility characteristics in the saturated condition to support structures under static and dynamic loading.

At Unit 1 the more significant of the two leaks was located at or beyond the upper elbow of the fire line as discussed in the response to Question 3. As shown on figures 3-1 and 3-2, this is only a few feet away from the seismic gap from which the soil was washed out. Because the pipe-soil and wall-soil interfaces adejacent to the seismic gap are narrow zones of weakness to be exploited by water flowing under pressure, it is most likely that the soil disturbance was concentrated along these narrow pathways rather than spread through the general mass of the backfill.

Uplift of the Auxiliary or Control Buildings could not have occurred as a result of pipe leaks for the following reasons:

- There is no evidence of significant uplift in the settlement monitoring records of either building.
  Does this imply that there was an indication of some "minor" uplift in settlement data?

- Pore pressures in excess of about 40 psi would be needed to cause uplift of the Auxiliary and/or Control Buildings. By comparison only about one tenth of this pressure would be sufficient to uplift the slab in the dead space and even lower pressures would be sufficient to displace the rodo foam filler between this slab and the two category I structures. Therefore, well before the pore pressures could ever reach magnitudes sufficient to cause uplift in the heavy Category I structures the slab and foam filler in the dead space between the two structures would be displaced setting up a localized, potentially erosive seepage condition in the dead space area. ~~There is physical evidence in Unit 2 that the foam filler was indeed displaced in a local area.~~ There is no evidence that the slab was uplifted in either Unit 1 or

2, and there is no physical evidence that the foam filler was pushed up at Unit 1 or Unit 2.

what about Unit 3?

- Pore pressures greater than 40 psi would have to be applied over large portions of the basemats of structures, including areas far removed from the leaks, for the backfill to be effectively unloaded. Those high pore pressures over large areas would not develop because the backfill is in contact with local coarse sandy zones in the native soil around the perimeter of the power block excavation which would tend to drain water away from the power block and relieve buildup of pore pressures. This is illustrated by the as-built geologic maps of the power block excavations presented in Appendix 2D to the FSAR.

QUESTION 5

In Table 1 of Ref. 2, you indicate that there are only two locations at which a leak is known to exist in the Fire Protection Pipeline. In Table 2 of the same reference, you have shown that leakage was observed at many more locations of this pipeline. Explain this discrepancy. Also, a review of Table 2 indicates that you observed leaks in CA, CSW, O_2, MP and AR lines as well. Explain how you considered the effect of leakage in these lines in your final safety evaluation of the backfill condition. Also explain the mechanism which caused the leaks in these lines.

RESPONSE: Table 1 of Reference 2 lists the two leaks in the Unit 1 fire protection line identified by the various tests performed. As shown in this table, one leak was located at or beyond the upper elbow of the inclined section of pipe, and the second leak was located in the side branch. Table 2 of Reference 2 lists five tests performed to locate the leaks. As shown in the table, the first test was of the side branch and identified this leak. The second and third tests were of the inclined section and involved two different test techniques which identified the leak at or beyond the upper elbow. The fourth and fifth tests were of the entire system and indicated leakage through these same leaks. The

I gather these are all the same leak. We should say so as we go.

five different tests listed in Table 2 are not intended to imply the existence of five different leaks in the fire protection line, hence there is no discrepancy between the two tables. This should be reworded to make clear all we have is different data about one leak

Did we test all lines for leakage before modification? If so we should say so.

Table 2 of Reference 2 lists observed leakage during testing of the CA, CSW, O_2, MP, and AR lines. This testing was carried out to check these lines for proper venting immediately prior to injecting grout to permanently seal them. The testing was not for the purpose of investigating pre-existing conditions. At the time of the tests these lines had been out of service for at least 11 months (the length of time since discovery of leakage from the fire protection line and shutoff of all temporary utilities). Prior to the testing, these lines had been extensively modified preparatory to grouting. Modifications included cutting lines and installing vent valves. The observed leakage listed in Table 2 for the CA, CSW, O_2, MP, and AR lines verifies adequate venting for grouting purposes and is not relevant to a postulated leakage condition during the service life of these lines. For example, Table 3 of reference 2 indicates that grout vented from "the open end" of the CA pipe in the NW yard area. This open end resulted from excavation and cutting of the abandened pipe in preparation for gravity. what?

With the exception of the CSW line, all of the other lines mentioned above carried air or gases rather than water. Any pre-existing leaks during the service life of these lines would not be expected to transport soil. No leakage of water from the Unit 1 CSW line was ever detected. Did we test line?

For these reasons postulated leakage from the CA, CSW, O_2, MP, and AR lines was not considered in the final safety evaluation of the backfill condition.

If we could say we had no leaks then we don't need to mention "postulated" leakage.

QUESTION 6

Provide a listing of the locations and elevations of the safety-related buried pipes and utilities essential for plant operation (that are not abandoned) that are within 100 ft radius of the known and potential location

of pipe breaks and/or leaks. Give dates of installation of these pipelines and utilities and explain your reasons to believe that these utilities will remain safe and functional despite the nearby pipe leaks, consequent backfill erosion, and potential removal of soil from this area and beneath these facilities.

RESPONSE: There are no safety-related buried pipes within the 100-foot radius of the know or potential location of the pipe leaks. There are two buried safety-related electrical duct banks that penetrate the south wall of the Control building, nearly 100 feet from the pipe leaks. The electrical duct banks, along with their elevations and installation dates, are shown on figure 6-1.

Does this mean more or less than 100 ft?

These electrical duct banks are rather distant from potential leaks and lie way beyond any point from which the leaking water exited. There is no evidence of any significant soil removal or backfill erosion near these utilities.

Even if soil removal is postulated from this area, the cables in the electrical duct bank would remain safe and functional since electrical duct banks are able to span short distances (up to 6 feet) without any support.

Have we had any leakage into duct banks?

QUESTION 7

Your settlement data and plots dated October 3 and 6, 1983 (Ref. 3) are not self-explanatory. Update this information and provide the following additional details.

(a) On large scale drawings (plans and cross-sections) show the location and dimensions of the Auxiliary Building (including deep section), control building, corridor building and the abandoned Fire Protection System pipelines and any other abandoned utility lines. Clearly identify each building area and location of pipe lines and known and

potential locations of breaks. Show the location of settlement monuments on the same drawings.

(b) Tabulate values of maximum measured total settlements at monuments that are located in the Containment Building, Auxiliary Building, Control Building, Corridor Building, Radwaste Building, and Diesel Generator Building (i) at approximately the time of discovery of pipe leakage (say August 1981), (ii) in November 1981 (iii) after grouting (in November 1982) and (iv) based on latest settlement readings.

(c) Tabulate values of the measured maximum differential settlements at the time periods indicated in (b) above, and show comparisons of the measured data with the anticipated differential settlements assumed in the analysis of these structures and their appurtenances. Evaluate the impact of any differences between the measured and anticipated settlements on the design and performance of these structures and appurtenances.

(d) Large scale time (dates) vs. settlement plots of data between the time period of discovery of pipe leakage and the latest readings at monuments located in the Containment Building, Auxiliary Building, Control Building, Corridor Building, Radwaste Building, and Diesel Generator Building. Analyze the data to indicate (i) history of total maximum tilt of eact building (plots of date vs. maximum tilt) and (ii) history of maximum differential settlement between adjacent buildings (plots of date, vs. maximum differential settlements). Discuss the impact of these values on the design and performance of the various seismic Category I structures, systems and components.

RESPONSE:

(a) Six drawings (figures 7-1 through 7-6) are submitted with this response to show the location and dimensions of the Auxiliary Building, Control Building, Corridor Building and the abandoned Fire Protection pipelines and other abandoned utility pipelines.

Figures 7-1 and 7-2 show the plans and cross-sections for Unit 1, figures 7-3 and 7-4 for Unit 2, and figures 7-5 and 7-6 for Unit 3. The settlement monuments are shown on these drawings. See figures 3-1 through 3-5 for the locations of the known and potential pipe leaks.

(b) Tabulated values of settlements monitored at selected settlement marker monuments are presented in Reference 3 Tables 1, 2, and 3, "Settlements Monitored at Powerblock Settlement Markers" for Units 1, 2, and 3, respectively. Included are settlements monitored at all monuments mounted on the following structures at each powerblock:

1. Containment Building
2. Auxiliary Building
3. Control Building
4. Radwaste Building
5. Diesel Generator

The locations of settlement markers are shown in figure (**). No settlement marker monuments or monitoring data are available for the Corridor Building at any of the powerblock units.

Settlement markers are pins or bolts installed on structural members of critical structures to monitor settlements by optical surveying methods. These markers were installed after foundation mats had been poured and, therefore, provide only a partial record of settlements. Typically, a third to a quarter (**) of the weight of a given structure would already be in place before the settlement markers were installed. However, as can be seen by noting the construction milestone dates on the "Partial Settlement Record Plots" (figures ** through **), the settlement markers generally recorded a major portion of the settlements which occurred during the construction and adjacent backfilling of each structure, and

**To be confirmed/supplied

they recorded all of the post-construction settlements of each structure.

The settlements tabulated in tables 7-1, 7-2, and 7-3, "Settlements Monitored at Powerblock Settlement Markers", are the total settlements monitored at each point from the initial reading on the partially completed structure to the date specified. Milestone dates for start of construction, placement of settlement markers, and end of construction are also presented for proper interpretation of the settlement data.

It should be noted that for structures that were constructed before the pipe leaks, including the Auxiliary and Control Buildings in Unit 1, the maximum total settlement after the pipe leak was only 0.3 inches. Structures that were still under construction experienced slightly more settlement because of the additional load that was placed after the pipe leaks. In all cases, however, post construction total settlements were well below the nominal design limit of 1.5 inches.

I don't understand this - Is this due to weight of water in Bldg?

(c) Tabulated values of measured differential settlements at selected time periods between adjacent powerblock structures are presented in tables 7-4, 7-5, and 7-6, "Differential Settlements between Adjacent Settlement Markers" for Units 1, 2, and 3, respectively. The settlement values presented in these tables were scaled directly off the "Partial Differential Settlement Record" plots presented in figures (**) through (**). These plots do not show any anomalous trends that could be attributed to the pipe leaks.

Total post-construction differential settlements between adjacent safety-related structures as shown in tables 7-4, 7-5, and 7-6 are equal to or less than 0.2 inch. This is much less than the 1/2 inch allowable post-construction differential settlement

**To be supplied

Table 7-1

SETTLEMENTS MONITORED AT SETTLEMENT MARKERS - UNIT 1

Structure	Settlement Marker Point	Milestone Dates (month-year): Startup Construction	Milestone Dates (month-year): Placement Settlement Markers	Milestone Dates (month-year): End Construction	Settlement inches (1): Pipe Leak Discovered (9-81)	Settlement inches (1): 3 Months After Leak (12-81)	Settlement inches (1): After Grouting (11-82)	Settlement inches (1): Latest Reading (1-84)
Containment Building	U1-SM-20	9-76	10-77	3-80	1.7	1.8	1.9	2.0
	U1-SM-24	9-76	10-77	3-80	1.6	1.8	1.8	1.9
	U1-SM-25	9-76	10-77	3-80	1.4	1.6	1.6	1.7
Auxiliary Building	U1-SM-21	9-76	10-77	10-79	1.0	1.2	1.2	1.2
	U1-SM-22	9-76	10-77	10-79	1.3	1.4	1.5	1.5
	U1-SM-23	9-76	10-77	10-79	1.9	2.0	2.1	2.2
Control Building	U1-SM-34	8-77	1-78	3-79	1.4	1.4	1.5	1.5
	U1-SM-35	8-77	1-78	3-79	1.7	1.7	1.8	1.8
	U1-SM-36	8-77	1-78	3-79	1.7	1.8	1.9	2.0
	U1-SM-37	8-77	1-78	3-79	1.3	1.5	1.6	1.6
Radwaste Building	U1-SM-30	4-78	4-78	12-80	0.6	0.6	0.7	0.7
	U3-SM-31	4-78	4-78	12-80	1.3	1.4	1.5	1.6
	U3-SM-32	4-78	4-78	12-80	1.7	1.8	1.9	2.0
	U3-SM-33	4-78	4-78	12-80	0.8	0.8	0.8	0.9
Diesel Generator Building	U1-SM-50	3-78	6-79	6-79	0.2	0.3	0.3	0.4
	U1-SM-51	3-78	6-79	6-79	0.2	0.2	0.3	0.3
	U1-SM-52	3-78	6-79	6-79	0.1	0.2	0.3	0.4

(1) Total settlements measured from the first settlement marker reading to the date specified.

Table 7-2

SETTLEMENTS MONITORED AT POWERBLOCK SETTLEMENT MARKERS - UNIT 2

Structure	Settlement Marker Point	Milestone Dates (month-year) Startup Construction	Placement Settlement Markers	End Construction	Settlement inches (1) Pipe Leak Discovered (1-80)	3 Months After Leak (4-80)	After Grouting (11-82)	Latest Reading (1-84)
Containment Building	U2-SM-20	6-79	4-80	9-83	1.2	1.5	2.3	2.5
	U2-SM-24	6-79	4-80	9-83	1.5	1.7	2.6	2.6
	U2-SM-25	6-79	4-80	9-83	1.1	1.3	1.9	2.0
Auxiliary Building	U2-SM-21	5-79	9-80	7-83	0.6	0.8	1.1	1.3
	U2-SM-22	5-79	9-80	7-83	0.7	0.8	1.3	1.4
	U2-SM-23	5-79	9-80	7-83	0.9	1.1	1.7	1.8
Control Building	U2-SM-34	7-80	7-81	6-83 (98%)	0.6	0.8	1.1	1.0
	U2-SM-36	7-80	7-81	6-83 (98%)	1.0	1.3	2.3	2.3
	U2-SM-37	7-80	7-81	6-83 (98%)	0.7	1.0	1.6	1.6
Radwaste Building	U2-SM-30	9-81	2-82	6-83	(2)	0.0	0.7	0.7
	U2-SM-31	9-81	2-82	6-83	(2)	0.0	1.7	1.8
	U2-SM-32	9-81	2-82	6-83	(2)	0.0	1.6	1.7
	U2-SM-33	9-81	2-82	6-83	(2)	0.0	0.7	0.7
Diesel Generator Building	U2-SM-50	7-81	6-83	6-83	(2)	0.0	0.8	0.8
	U2-SM-51	7-81	6-83	6-83	(2)	0.0	0.4	0.3
	U2-SM-52	7-81	6-83	6-83	(2)	0.0	0.6	0.6

(1) Total settlements measured from the first settlement marker reading to the date specified.

Need note

See Table 7-3 ?

Table 7-3

SETTLEMENTS MONITORED AT POWERBLOCK SETTLEMENT MARKERS - UNIT 3

Structure	Settlement Marker Point	Milestone Dates (month-year): Startup Construction	Placement Settlement Markers	End Construction	Settlement inches (1): Pipe Leak Discovered (12-81)	3 Months After Leak (3-82)	After Grouting (11-82)	Latest Reading (1-84)
Containment Building	U3-SM-20	6-79	4-80	9-83	1.1	1.3	1.6	2.0
	U3-SM-24	6-79	4-80	9-83	1.0	1.1	1.4	1.7
	U3-SM-25	6-79	4-80	9-83	1.1	1.2	1.2	1.5
Auxiliary Building	U3-SM-21	5-79	9-80	7-83	0.7	0.7	1.0	1.1
	U3-SM-22	5-79	9-80	7-83	1.1	1.2	1.4	1.9
	U3-SM-23	5-79	9-80	7-83	1.2	1.4	1.7	2.0
Control Building	U3-SM-34	7-80	7-81	6-83 (98%)	0.3	0.3	0.7	0.6
	U3-SM-35	7-80	7-81	6-83 (98%)	0.4	0.4	0.7	1.2
	U3-SM-36	7-80	7-81	6-83 (98%)	0.4	0.6	1.1	1.5
	U3-SM-37	7-80	7-81	6-83 (98%)	0.4	0.5	1.0	1.1
Radwaste Building	U3-SM-30	9-81	2-82	5-82	(2)	0.1	0.3	0.4
	U3-SM-31	9-81	2-82	5-82	(2)	0.0	0.5	1.0
	U3-SM-32	9-81	2-82	5-82	(2)	0.0	0.4	0.9
	U3-SM-33	9-81	2-82	5-82	(2)	0.1	0.3	0.5
Diesel Generator Building	U2-SM-50	7-81	6-83	6-83	(2)	(2)	0.2	0.3
	U2-SM-51	7-81	6-83	6-83	(2)	0.0	0.3	0.3
	U2-SM-52	7-81	6-83	6-83	(2)	0.0	0.4	0.6

(1) Total settlements measured from the first settlement marker reading to the date specified.
(2) Settlement ~~market~~ marker point were not yet installed.

Table 7-4

DIFFERENTIAL SETTLEMENTS BETWEEN ADJUSTMENT SETTLEMENT MARKERS - UNIT 1

Powerblock Unit No.	Adjustment Structures: Structure A	Adjustment Structures: Structure B	Points Compared: Strucutre A	Points Compared: Structure B	Total Measured Differential Settlement (in)(1): Pipe Leak Discovered (9-81)	Total Measured Differential Settlement (in)(1): 3 Months After Leak (12-81)	Total Measured Differential Settlement (in)(1): After Grouting (11-82)	Total Measured Differential Settlement (in)(1): Latest Reading (1-84)	Post Construction Settlement To date (in)
1	Control Building	Auxiliary Building	U1-SM-36 U1-SM-37	U1-SM-22 U1-SM-52	0.7 0.5	0.7 0.6	0.7 0.6	0.8 0.7	0.2 0.2
1	Control Building	Diesel Generator Building	U1-SM-34	U1-SM-52	0.0	0.0	0.0	0.1	0.1
1	Radwaste (2) Building	Auxiliary Building	U1-SM-31	U1-SM-22	1.0	1.0	1.0	1.1	0.2
1	Radwaste (2) Building	Control Building	U1-SM-31 U1-SM-32	U1-SM-36 U1-SM-35	0.6 0.9	0.6 0.9	0.7 1.0	0.7 1.0	0.1 0.2

(1) Based on relative movement of adjacent settlement marker monuments starting immediately after both markers were installed.

(2) Not a Category I Safety-Related Structure.

specified as a design criterion. Therefore, any differences between the measured and anticipated post-construction differential settlements are not significant.

(d) Time (dates) vs. settlement plots for 5 structures at each powerblock -- the Containment Auxiliary, Control, Radwaste and Diesel Generator Buildings are presented in figures (**) through (**). Differential settlements between adjacent settlement markers on these structures are presented in figures (**) through (**). Plots of tilt for these structures are presented in figures (**) through (**).

Design criteria specified at PVNGS related to settlements of safety-related powerblock structures are as follows:

- Maximum post-construction total settlement = 1.5 inch
- Maximum post-construction differential settlement = 0.5 inch

Maximum post-construction settlements are much less than the allowable 1.5 inches at all structures, as can be seen in figures (**) through (**). Maximum post-construction differential settlement measured between adjacent safety-related structures studied is 0.2 inches, as noted in tables 7-4, 7-5 and 7-6. This is also less than the allowable 0.5 inches. Therefore, no settlement-related adverse impacts to the seismic Category I structures and components is anticipated. Although no design criteria limits have been specified for tilt, the very small tilt angles measured (less than a maximum of about 0.04 degrees) will not have any adverse effect on Seismic Category I structures.

**To be supplied

Do we have these yet? If not how can we draw a conclusion?

QUESTION 8

Indicate how much settlement of the Auxiliary, Control and Corridor buildings has occurred since the connections between structures and buried

safety-related utilities were made. Evaluate the effect of the past and anticipated future settlement of these structures on safety-related utility connections.

RESPONSE:

There are no buried safety-related utilities between the Auxiliary, Control and Corridor buildings. As stated in Response to Question 6, there are two buried safety-related electrical duct banks that penetrate the south wall of the Control building, nearly 100 feet from the pipe leaks. The electrical duct banks are shown on figure 6-1.

See comment earlier.

The connection of the electrical duct banks to the wall of the Control building utilizes two flexible fittings and is designed for differential settlement and motion due to a seismic event. Therefore, the past and anticipated future settlement of the Control Building will have no detrimental effect on the connection of the electrical duct banks to the building.

How much versus what we expect?

QUESTION 9

In your report on the engineering evaluation of conditions of backfill after pipe leaks (Ref. 1) you have not provided sufficient analysis details and assumptions used in your analysis of the Auxiliary Building. Provide the following information in sufficient detail so as to enable the staff to make an independent evaluation of your analysis.

(a) Your basis for assuming the extent and location of the non-bearing area under the auxiliary building. Discuss the effect of possible backfill voids on the stability of the adjoining control building and corridor building. Explain how you accounted for possible reduced density and reduced strength of the backfill under the structures after the pipe leaks.

(b) The analytical model used to calculate the changed stresses after erosion from pipe leaks, and the various material properties used in the model.

(c) The modified properties of the backfill underneath the foundations after pipe leaks. Compare these properties with the properties of the backfill used in your analysis for conditions existing before the leak. Justify the basis for your assumptions made in the analysis.

(d) The various loads and load combinations used in evaluation the changed stresses.

(e) Details of the numerical results of analysis, and factors of safety, before and after the assumed erosion occurred.

RESPONSE:

Basis and where described?

9a. Figures 9-1 and 9-2 illustrate the following discussion. Figure 9-1 contains the plan and cross section of the Auxiliary Building. Figure 9-2 shows an isometric view of the wing of the Auxiliary Building. The basis for assuming the extent and location of the non-bearing area under the Auxiliary Building was conservatively established. Although the zones of slightly reduced soil densities would still provide considerable bearing capacity, it was assumed for this analysis that total bearing capacity was lost. Also the excess grout that was injected into the soil was not considered, even though it would help regain additional support for the structure.

The area assumed to have lost complete soil support was a 20- by 35-foot strip. The 20-foot width was taken from the outside south edge of the Auxiliary Building. The farthest northward pipe leak was a distance of 6'4" from that south edge under the wing of the Auxiliary Building. The affected area was assumed to extend two to three feet beyond the pipe location. Therefore, for the reanalysis

to be conservative, a value of twice this distance was used, or after rounding up, 20 feet. The 35-foot dimension was taken as the total length of the wing of the Auxiliary Building. The dimensions described above are shown on figure 9-1.

The Control Building is beyond the points where leakage occurred and where the leaking water finally exited. It is difficult to conceive of any soil movement from the direction of this building. Studies of soil densities and settlement data also do not indicate any evidence of reduced soil densities beneath or around this building. Although the Corridor Building is not a Category 1 structure, calculations show that a total collapse of the Corridor Building during a seismic event would not significantly damage any Category 1 structure or equipment.

There is no mechanism to suggest that the soil directly beneath the Control Building has experienced any disturbance. However, it can be postulated that the projection of an angular zone of loading from the north edge of the Control Building would intercept a zone affected by the leaking water and the soil movement. The affect of this zone on the total stability of the Control Building is considered to be negligible since the area of reduced soil density compared to the total area of the Control Building foundation is small.

9b. The reanalysis of the Auxiliary Building with the postulated localized soil disturbance from pipe leaks is similar to the original design. The procedures of the original design and the reanalysis are provided for comparison as follows:

I. ORIGINAL DESIGN

- Dynamic Analysis

The dynamic analysis of the Auxiliary Building was performed using a two-dimensional lumped-mass beam stick model. Seismic forces and in-structure response spectra were computed in accordance with Section 3.7 of the FSAR.

The following outline provides the procedures utilized for the dynamic analysis:

- Developed two-dimensional lumped-mass beam stick model. Calculated wall section properties and loading at each floor level.
- Performed fixed-base modal extraction for natural frequencies and mode shapes.
- Calculated frequency dependent soil springs and damping coefficients using the methods of BC TOP-4A.
- Obtained soil-structure composite modal damping.
- Performed time history analysis.
- Developed instructure response spectra.
- Performed response spectrum analysis to determine shear forces, overturning moment and axial forces.

Stability Analysis

Checked the stability of the Auxiliary Building against overturning, displacement, and sliding resulting from the seismic forces provided by the dynamic analysis.

Basemat Design

The basemat of the Auxiliary Building was designed using conventional methods. Soil pressures at critical points were determined for all design criteria load combinations and seismic directions. The reinforcing steel is designed for the soil pressure profile that produces the highest stress. The basemats at elevations 40 feet and 70 feet were taken to be in the same plane for the soil pressure calculation and the assumption of a linear soil distribution is substantiated with a check on basemat rigidity.

Is this conservative?

Since the postulated localized soil disturbance directly effects the soil pressure distribution on the basemat, an outline of the basemat design procedure used is provided:

- Calculated basemat properties center-of-gravity, area, moment-of-inertia.
- Determined the eccentric overturning moments resulting from the eccentricity of the dead load and live load relative to the basemat center-of-gravity.
- Calculated seismic overturning moments.
- Calculated soil pressures at critical points on the basemat considering applicable load combinations and seismic directions.

the structure. Two shear walls on both the E-W and N-S directions are cantilevered over the non-bearing area (figures 9-1 and 9-2). These walls were conservatively checked as large corbels to show that they can adequately transfer their loads to the bearing-area of the basemat.

9c. In evaluating the geotechnical properties of backfill materials it was conservatively assumed that the backfill would become saturated at some point during the life of the facility. Consequently, laboratory tests on representative backfill materials were conducted under fully saturated conditions. Furthermore, based on these tests the permissible percentage of fines in the backfill was limited to less than 30% in order to minimize any potentially adverse effects of saturation. All test data are included in Appendix 2T of the PSAR.

Shear strength

The design shear strength parameters used for bearing capacity analyses were derived from results of triaxial shear tests conducted under saturated conditions on specimens remolded to about 95% of maximum density (ASTM D-1557). The design parameters selected for design ($\emptyset=36°, c=o$) represent a lower bound fit through the data. Considering that the average "as placed" relative compaction was about 98% and the fact that the tests were conducted under saturated conditions, these parameters would still be conservative even if the backfill became fully saturated after the pipe leaks.

Compressibility

The compressibility of potential backfill materials with various percentages of fines was evaluated by consolidation tests conducted under both "optimum" and saturated moisture conditions. Most specimens were saturated under a surcharge

load of 4000 lb/ft^2 to estimate the potential for hydrocompaction under field conditions. The tests indicated that saturation had no significant effect on the compressibility of backfill soils with less than 30 fines compacted to 95% of maximum density (ASTM D-1557). Saturation did have a significant effect on the compressibility of site soils with more than 40% fines, but such soils were not used in Category I backfill.

Dynamic stiffness and damping

The dynamic properties of backfill soils were evaluated by cyclic triaxial and resonant column tests. The cyclic triaxial tests were conducted under fully saturated conditions while the resonant column tests were conducted under nearly saturated conditions.

was this analysis all done at 0.25g? If so this should be mentioned in that it would increase conservatism over 0.20g for which we are Licensed.

Based on these observations, it is concluded that:

- The geotechnical properties of backfill used in the design calculations would adequately account for the effects of saturation due to pipe leaks.
- Due to the low percentage of fines in backfill soils and relatively high degree of compaction, saturation would not be expected to have a significant impact on the support characteristics of backfill soils at the site.

9d. The original calculation considered all design criteria load combinations. The reanalysis considered the load combination $U = 1.4D + 1.7L + 1.9E$ because it was determined to be the critical combination in the original analysis. Seismic loading in three directions was considered in the reanalysis.

9e. Details of the numerical results are provided in table 9-1. Even though Table 1 of Reference 1 shows that the cantilevered wall

reasons

Future deterioration of support conditions is not expected for the ~~reasons~~ discussed in the response to Question 9c; however, this has already been taken into account in the structural analysis which assumes a total loss of soil support over a conservatively large area. The basis for selection of this area is discussed in the response to Question 9a.

Groundwater effects are not considered significant because the perched groundwater elevation is beneath the backfill elevation and is in a long-term declining trend as reported in the FSAR. The possibility of an earthquake at the site has been factored into the criteria for selection of backfill materials and compaction requirements and has been considered in extensive settlement analyses and liquefaction analyses which are reported in the PSAR and FSAR. The conclusion from this work is that the undisturbed backfill outside the 20- by 35-foot area will continue to provide adequate bearing capacity and compressibility characteristics during and after a seismic event. The nature of the other types of unanticipated disturbances or vibratory activity in the vicinity of the plant postulated in the question is difficult to foresee but the level of such activity should certainly be less than that of a seismic event.

In conclusion, the assumption of total loss of backfill support under a 20- by 35-foot section of the Auxiliary Building wing is considered conservative. Outside of this area, test data show that a transient condition of saturation could not change the backfill properties so as to reduce future bearing capacity or significantly increase compressibility under various vibratory loading conditions up to and including an SSE event. Within the zone of soil disturbance, some filling of voids was achieved by grouting but no credit has been taken for this in the structural analysis. Further grouting of backfill was not undertaken due to the risk of damage to buried installations.

QUESTION 11

In the report dated August 25, 1982 by your grouting consultant John King (Appendix E of Reference 2), it is suggested that "should there be any need to fill remaining voids or consolidate loosened fill material, this can be handled by driving insert pipes into the soil in the gap and pressure grouting through them until the surface starts to heave." Describe detailed plans and procedures that you would use to grout the remaining backfill voids should this action be required to correct foundation deficiencies.

RESPONSE: As discussed in the response to Question 10, additional grouting is not planned. It is concluded that the investigations, analyses, and grouting performed to date have satisfactorily resolved all concerns regarding performance of safety-related structures and installations. The remark by Mr. King quoted in the question was a helpful suggestion should future conditions indicate a need for additional grouting. No such need is foreseen at the present time.

Does King agree with this?

The possibility of grouting in the future is not precluded; however, the development of detailed plans and procedures should await definition of a specific purpose for this grouting. Development of such plans and procedures would need to be based on additional study of soil conditions in the specific area in question. Without such definition and additional study, detailed plans and procedures may not be meaningful.

What would be basis for doing this in the future? Increased settlement or tilt? If this is true shouldn't we indicate criteria here and then monitor so we can disposition this once & for all?

QUESTION 12

Explain how you would implement a grouting program for potential voids in the backfill under Category I structures, without undue risk, while the plant is in operation. Describe possible difficulties and precautions that would be needed, if any, in case the need for this remedial work should arise at a later date.

RESPONSE: As discussed in Question 11, there are no plans to implement an additional grouting program. Should discussions with the Regulatory

see my comment on Question 11. We need to determine now what would require this.

Staff identify a need for further grouting or other remedial activities while the plant is in operation, APS will develop plans and detailed procedures for the work. In addition to the geotechnical review deemed necessary to achieve a quality program, APS would ensure that the work would be reviewed for safety significance in accordance with 10CFR50.59 and the PVNGS Technical Specifications. APS further commits to submit such plans to the Regulatory Staff for their review and approval prior to commencing work.

This really doesn't answer question in my opinion.

REFERENCES

1. Senczyszyn R., et al., "Engineering Evaluation of Conditions of Backfill After Pipe Leaks at the Palo Verde Nuclear Generating Station, Phoenix, Arizona," Deficiency Evaluation Report No. 81-35, Bechtel Power Corporation, LAPD, Norwalk, CA., May 1982.

2. (**)"Grouting for Disposition of Temporary Piping at the Palo Verde Nuclear Generating Station, Units 1, 2 and 3, Phoenix, Arizona," Bechtel Power Corporation, LAPD, 1982.

3. (**)"Settlement Observation Record Tables and Graphs," Bechtel Power Corporation, LAPD, October 3 and 6, 1983.

**Author(s) to be supplied