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WESF NATURAL PHENOMENA HAZARDS SURVEY

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Abstract

A team of engineers conducted a systematic natural hazards phenomena (NPH) survey for the 225-B Waste Encapsulation and Storage Facility (WESF). The survey is an assessment of the existing design documentation to serve as the structural design basis for WESF, and the Interim Safety Basis (ISB). The lateral force resisting systems for the 225-B building structures, and the anchorages for the WESF safety related systems were evaluated. The original seismic and other design analyses were technically reviewed. Engineering judgement assessments were made of the probability of NPH survival, including seismic, for the 225-B structures and WESF safety systems. The method for the survey is based on the experience of the investigating engineers, and documented earthquake experience (expected response) data. The survey uses knowledge on NPH performance and engineering experience to determine the WESF strengths for NPH resistance, and uncover possible weak links. The survey, in general, concludes that the 225-B structures and WESF safety systems are designed and constructed commensurate with the current Hanford Site design criteria.

Janis Bishop 7-1-96
Release Approval Date

Release Stamp

Approved for Public Release

ENGINEERING REPORT

NATURAL PHENOMENA HAZARDS SURVEY FOR THE WASTE ENCAPSULATION AND STORAGE FACILITY

Prepared for

Westinghouse Hanford Company

June 1996

Subcontract WHC-380393

Prepared by

ICF Kaiser Hanford Company
Richland, Washington

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E51162ER

ENGINEERING REPORT

FOR

NATURAL PHENOMENA HAZARDS SURVEY FOR THE WASTE ENCAPSULATION AND STORAGE FACILITY

Prepared by

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Richland, Washington

for

Westinghouse Hanford Company

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1.0 INTRODUCTION

1.1 PURPOSE

An investigating team consisting of two principal engineers from ICF Kaiser Hanford Company and one principal engineer from Westinghouse Hanford Company has performed a natural phenomena hazards (NPH) survey to appraise the ability of the 225-B Waste Encapsulation and Storage Facility (WESF) to resist the natural forces of earthquake, extreme wind, and volcanic ashfall. Qualification summaries for the investigating engineers are shown in Appendix C. The survey supports the development of the WESF Interim Safety Basis (ISB). A necessary input to the ISB is the structural qualification of the buildings and systems where failure or malfunction might cause or increase the severity of an accident that would affect the public health and safety or are necessary for the safe shutdown and isolation of the processes involving radioactive materials. The current WESF mission is long-term capsule storage in the pool cells.

This survey is an assessment of the thoroughness and applicability of the existing design analysis documentation to serve as the structural design basis for the current WESF mission and an assessment of the potential need and likelihood of success for additional NPH evaluations. Included in this survey is the confirmation that design modifications, as recommended by the design analysis, were implemented. Lateral force-resisting systems for 225-B structures and anchorage and support of safety systems were determined and an assessment made, based on engineering judgement, of the probability of survival and ability to function following design basis events (see Section 1.3).

1.2 SCOPE

The scope of this survey included a technical review of the original WESF seismic design analysis and review (ref 1, 2, and 6). The design criteria applied (ref 15 and 16), other available design analyses, the construction drawings, and specifications (ref 30) were reviewed. Inspections were conducted for the 225-B Building and capsule storage pool structures, and the systems and equipment required to operate the pool cells (see Figures 1 and 2). Several engineering reports for inspections and repairs to the reinforced concrete 225-B Building and the process cell floor liners were obtained and reviewed. The lateral force-resisting systems for the WESF structures were determined. The configuration, anchorages, and lateral supports for the systems and equipment items were determined. Evaluations were made on the probability of NPH survival for the WESF structures and systems, based on engineering judgements about whether the responses of the items could be expected to be within acceptable limits. The following WESF structures and systems that have been assessed for NPH are shown in Table 1.

TABLE 1

ASSESSED 225-B WESF STRUCTURES AND SYSTEMS		
Structures		
225-B	Area 1	Instrumentation and HVAC Area
225-B	Area 2	Process Cell Area
225-B	Area 3	Pool Cell Area
225-BB		K3 Filter Building
225-BA		K-1-1 Filter Building
221-B		B-Plant Canyon End Wall
Systems		
Pool Cell Drain (also known as Waste Water Removal)		
Pool Cell Demineralized Water Addition		
Pool Cell Recirculation and Heat Removal		
Process Cell Operating Gallery Fire Protection		
K3 Filter		
Alternating Current Power Backup		
Process Cell Shielded Viewing Windows		
K1 and K3 Fans		
Bridge Cranes and Support Systems		
Pool Cell Powered Catwalk		

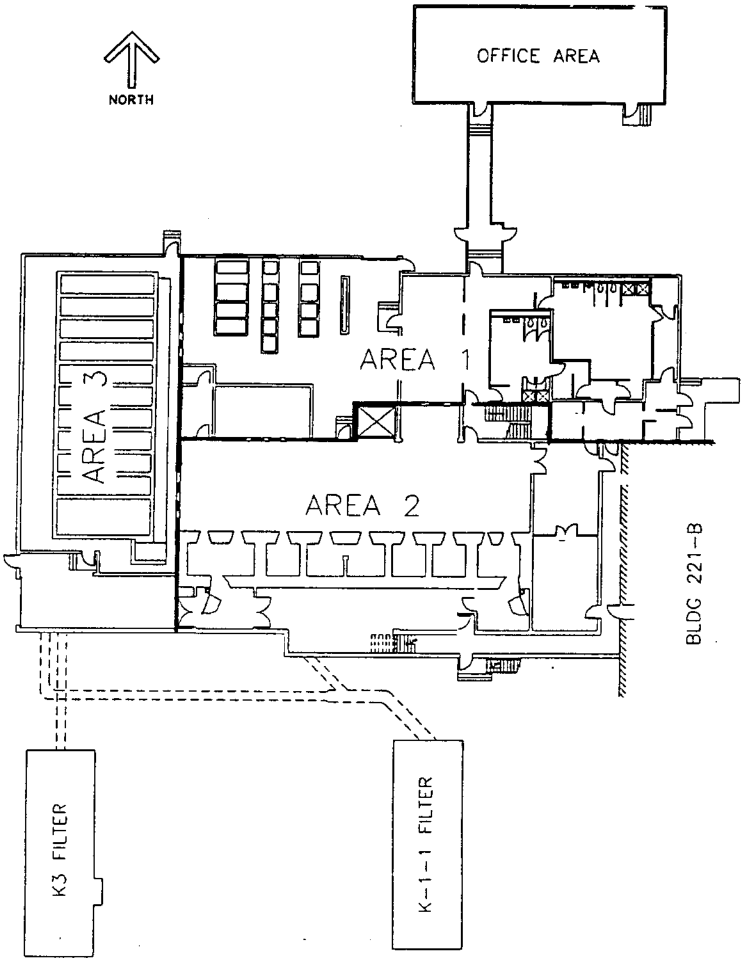


Figure 1 Waste Encapsulation and Storage Facility Layout

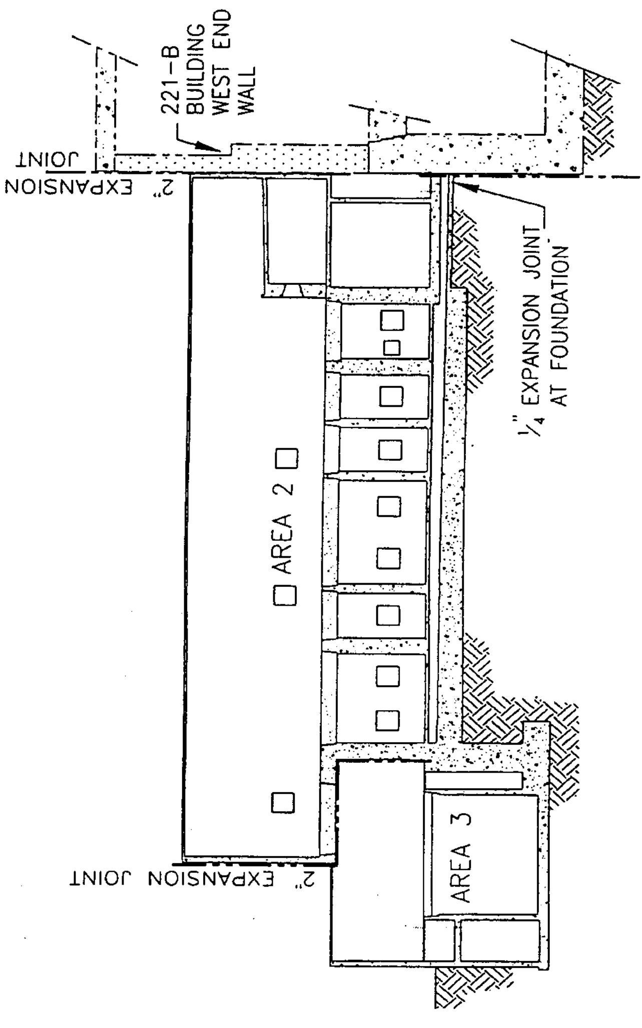


Figure 2 Waste Encapsulation and Storage Facility cross section

The WESF, including the 225-B Building, design and construction, as well as the design criteria used (ref 15 and 16), precede the current Hanford Site design criteria, SDC 4.1, Rev. 12 (ref 13); a U.S. Department of Energy (DOE) safety analysis preparation guide, DOE-STD-3009-94 (ref 14); and Safety Analysis Manual, WHC-CM-4-46, Rev. 2 (ref 29). The design conditions, loads, and requirements for three NPH events are defined in reference 13. The events are the: 0.20 gravity design basis earthquake (DBE), 90 mph design basis wind (DBW), and volcanic ashfall and snow loads, combined. Safety Classes 1, 2, 3, and 4 are the safety-related design categories for structures, systems, and components (ref 13). Two safety-related categories, Safety Class and Safety Significant, are described in references 14 and 29. This survey and the NPH assessments made for the WESF are based mainly on the current site NPH criteria (ref 13). The WESF structures, with the exception of Area 1 of the 225-B Building, which was assessed as Safety Class 3, were assessed as Safety Class 1. The WESF systems and equipment included in the survey were assessed as Safety Class 1. Similarly, if references 14 and 29 were applied for this survey, the WESF structures, except Area 1, and systems could be in the safety class category. Area 1, presuming potential consequences to facility workers and mission but not to the public or other onsite workers, could be in the Safety Significant category.

1.3 HAZARD SURVEY METHOD

The methodology for the NPH survey of the WESF is based on the experience of the investigating engineers and documented knowledge on the effects of earthquakes on structures and equipment. The structural design documentation and the competence of the design team were also considered in the survey. With this experience and available knowledge, systematic assessments were made of how the WESF structures and systems could be expected to respond to the DBE ground motions and other NPH. Then, evaluations were made on the probability of NPH survival for the WESF structures and equipment based on the engineering judgement of the investigating engineers.

Systematic NPH assessments are encouraged and previously have been made for existing DOE facilities (ref 14, 23, 26, and 27). The safety analyses preparation guide for DOE facilities allows for immediately available design information to be used to maximum advantage for the methodology used to characterize (existing) facility safety without complete well-documented design information. The evaluations determine if the facility was designed and constructed to seismic and NPH criteria commensurate with the present criteria, if construction was closely supervised, and if facility improvements are needed. The objective is to ensure that an adequate level of safety is achieved economically, versus bringing the facility into strict compliance with current NPH requirements.

There is, as indicated by references 17, 18, 19, and 22, an available large body of knowledge on the effects of earthquakes on engineered structures and equipment. This knowledge has been compiled from the results of post-earthquake investigations and other seismic performance research. The fundamental purpose for documenting and continuing to expand this knowledge is to discover, in a scientific and systematic manner, how structural systems perform in earthquakes (expected response).

The current seismic design principles, provisions, and requirements have been established and are improved (from the observations of what has worked and what has not) by experienced practicing structural engineers (ref 19). The judgement of the design engineer and the quality of construction effect the seismic and NPH performance of structures just as do the numerical calculations and the other scientific principles that design codes and criteria may require (ref 17 and 18). A similar systematic inspection and judgement process can be used for assessing the adequacy of existing facilities to resist earthquake motions and other NPH (ref 19, 20, 21, 24, and 25). As an alternative to using the procedures and analyses applicable to new modern plants, the use of experience data is a practical cost-effective method to develop the documentation needed for seismic and NPH requalification of existing nuclear power plants (ref 23 and 24).

This survey is an evaluation of the (existing) WESF buildings and equipment characteristics and features that would be relied upon to resist seismic and other NPH demands. For buildings, the characteristics and features are: the lateral load path or system, redundancy, connections, configuration, condition of materials, and the influence of adjacent buildings (ref 17, 21, and 25). The equipment systems depend on their anchorages and lateral load resisting supports. Equipment can also be damaged by adjacent (poorly anchored) equipment (ref 17, 21, 22, and 23). The evaluation process is based on the concepts that seismic and NPH resistant design requirements employ. The process uses the available knowledge on NPH performance and engineering experience to determine the strengths for NPH resistance and to uncover weak links that need strengthening or further evaluation (ref 21, 23, and 25).

To begin the WESF survey, the facility was inspected by the investigating engineers. The construction drawings and specifications were studied. The lateral load paths and structural condition were determined for the buildings. The anchorages (for seismic resistance) and other seismic support features were determined for the equipment important to the capsule storage mission. The original design (including seismic) analyses were reviewed. The differences between the current Hanford Site NPH design requirements and the original criteria used for the WESF were assessed. The available seismic and NPH performance knowledge, largely for reinforced concrete structures, ductile piping, and anchored equipment were reviewed and compared to existing conditions at WESF (ref 17, 18, 21, and 22). Then, assessments were made of how the buildings and equipment would be expected to respond to NPH demands, and their probability of being able to survive the current site DBE and design NPH conditions.

Further justification for concluding that the use of experience data can be acceptable to requalify the WESF for NPH safety (versus additional analyses and strict compliance with the current site NPH criteria) develops when the reference 28 process is applied. The purpose for this process includes safely screening out or minimizing the expense of detailed additional NPH evaluations for existing DOE facilities. The approach is to develop numerical scores for four facility attributes. The product of the attribute scores is the priority score (PR) for the facility. Higher PRs recognize facilities as potentially having hazardous seismic or other NPH deficiencies.

The four attributes for a facility rated by the reference 28 process are: Remaining Life, Consequences of NPH Facility Failures, Building Vulnerability, and Mission Importance. These can be scored using available information with little additional evaluation or analysis. This overall prioritization scoring process for DOE facilities, including the WESF, is shown in Figure 3. For the WESF, using the reference 28 process, the maximum scores given for three of the attributes are: Remaining Life (10), Failure Consequences (10), and Mission Importance (5). Reference 28 prescribes the use of seismic and NPH experience data, existing seismic analyses, and engineering judgements to rate building vulnerability. The available design analyses documentation for the WESF shows that the buildings are sufficient enough to resist safely the potential site NPH demands. Accordingly, the WESF condition is rated from good to fair and, for a condition of fair, the score is 1. A facility judged to be in very poor condition would have a score of 10. According to reference 28, if PR equals or exceeds 2,000, additional detailed NPH analyses are required. The product of the attribute scores or the WESF PR is (1)x(10)x(10)x(5) = 500, and little additional NPH evaluation is required. This NPH survey shows that the WESF was designed and constructed commensurate with current site seismic and NPH criteria.

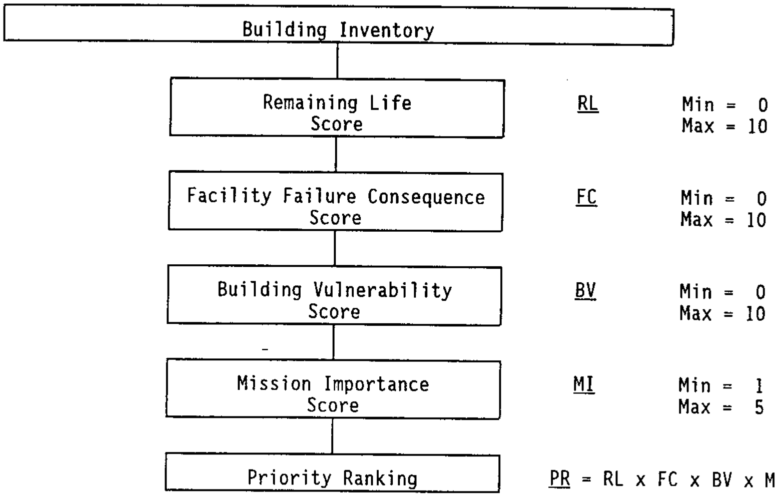


Figure 3. DOE Guide for NPH Evaluation

2.0 SUMMARY

A survey has been completed to appraise the ability of the 225-B Waste Encapsulation Facility (WESF) to resist design basis earthquake (DBE) ground motions, and other natural phenomena hazards (NPH). The purpose of the survey is to support the development of the WESF Interim Safety Basis (ISB). The method for conducting this survey is described in the Introduction. Technical reviews were made of the original WESF seismic design analyses by John A. Blume and Associates (ref 1 and 2), the VITRO Engineering Corporation design structural calculations (ref 6), the design drawings, and specifications (ref 30). Inspections were conducted for the 225-B Building, specifically the capsule storage pool structures (see Figures 1 and 2), and the systems and equipment used to operate the pool cells. The lateral force-resisting systems for the 225-B Building structures were assessed. The configuration, functional requirements, anchorages, and lateral supports were assessed for the systems and equipment important to the capsule storage mission. Evaluations were made on the probability of seismic and NPH survival for the WESF structures and systems (see Tables 2 and 3). A summary of the main conclusions of this survey are:

- Area 1 of the 225-B Building with the HVAC room and most of the control and surveillance instrumentation for the pool cells was originally designed for a 0.12 gravity operating basis earthquake. Therefore, Area 1 and the equipment therein are well within the minimum UBC seismic and wind provisions in reference 13 currently required for Safety Class 3 facilities. It is also likely that Area 1 could survive the current Hanford Site DBE. Additional seismic analysis would be necessary to confirm this.

- Areas 2 and 3 of the 225-B Building, the process cells and storage pool cells, were originally designed for a 0.25 gravity earthquake. The documentation that remains available is incomplete. It is believed that further comprehensive seismic analyses, if conducted, would successfully show these areas adequate for DBE requirements. However, the safety analyses preparation guide for DOE facilities allows for existing design information to be used without complete documented design information.

Accordingly, it would not be necessary to repeat or do additional seismic analyses for the 225-B Building.

- The K3 high efficiency particulate air filters (HEPA) are unanchored in the filter building (vault) which allows them to be remotely changed with ease. However, the filters could be dislodged by seismic motions and the K3 ventilation system would not be functional.

- DBE seismic motions could interrupt the make-up demineralized water for the storage pools and raw water to the recirculation system heat exchangers and the WESF fire protection system. An emergency make-up water supply system or procedure could ensure that sufficient water levels are maintained in the pool cells if a DBE occurs. It is presumed that the loss of the recirculation system heat exchangers and the fire protection system could be tolerated.

- The 221-B Building west-end wall could be damaged by DBE motions and cause local structural damage to the east end of the WESF. This is remote from the storage pools. However, if such WESF damage is not safely acceptable, structural retainers for a portion of the 221-B Building end wall could be designed and installed.

- The two overhead bridge cranes, the electric powered catwalk, and their support systems in the 225-B Building are configured and equipped to prevent seismic movements and derailing. It is concluded that the cranes and catwalk have adequate seismic resistance.

3.0 CONCLUSIONS

The following summarizes the findings and conclusions of this survey which, for the ISB, could provide sufficient seismic and other NPH qualification for the WESF buildings (Figures 1 and 2), and systems.

The probabilities of NPH survival as judged by this survey for the WESF structures, and the systems and equipment important to the capsule storage mission are given in Tables 2 and 3, respectively.

Area 1 of the 225-B Building was designed originally for a 0.12 gravity operating basis earthquake (OBE). Area 1 includes the air lock entrance to the pool cells, Area 3. Area 1 also has the HVAC room and the control and surveillance instrumentation for the pool cells. The equipment and instrumentation is anchored to the concrete floor. Therefore, Area 1 of the 225-B Building is well within the minimum UBC seismic and wind provisions currently required for a Safety Class 3 facility (ref 13). It is also likely, based on the original OBE analysis and design requirements, that Area 1 could survive the current Hanford Site DBE. Additional structural evaluations would be required to confirm the ability of the Area 1 structure to safely withstand the DBE.

The original DBE analyses for 225-B Building Areas 2 and 3, the process cells, and storage pool cells were conducted with state-of-the-art methods by competent seismic or earthquake engineers. Proper design codes were used and were properly applied in the determination of structural capacity and safety margin. The DBE applied was estimated to be double the current site requirements (ref 13). The design documentation that remains available is incomplete. It is believed that further comprehensive seismic analyses, if conducted, would successfully show that Areas 2 and 3 are adequate for DBE requirements. However, the safety analyses preparation guide for DOE facilities allows for immediately available design information to be used to maximum advantage for the methodology used to characterize (existing) facility safety without complete well-documented design information. Accordingly, it would not be necessary to do additional complete seismic analyses for the 225-B Building.

A previous structural evaluation of the 221-B Building west-end unreinforced concrete wall recommended further technical review to address in-plane diagonal shear cracking and the effect the cracking would have on the out of plane stability of the wall (ref 5). It is concluded from this evaluation that it is unlikely that further evaluation of the end wall would be successful in qualifying the 221-B Building west-end wall for the DBE. The potential exists for localized damage to the east end of the 225-B Building Area 2 as a result of seismic damage to the 221-B Building west-end wall. This is remote from the 225-B storage pool cells, Area 3. If localized damage to Area 2 of the 225-B Building is not safely acceptable, a system of structural retainers could be designed and installed on the 221-B Building end wall to preclude pieces of the wall from falling on the 225-B Building.

It is unlikely that the pool cell demineralized water addition system will survive the DBE. An emergency make-up water supply system or procedure could ensure that sufficient water levels are maintained in the pool cells if a DBE occurs.

It is unlikely that the raw water system supplying water to the pool cell recirculation system heat exchangers and the 225-B Building fire protection system will survive the DBE. It is presumed that the loss of these systems could be safely tolerated.

The fire protection system is not qualified for DBE loads. It appears unnecessary for safety purposes that fire protection piping supports and lateral restraints be evaluated and upgraded to qualify the fire protection system for DBE loads.

Although found likely to survive the DBE, the diesel/electric AC backup electrical power system has not been qualified for NPH loads. Additional evaluation would be required for the AC backup electrical power system to confirm the ability of the system to withstand the NPH loads.

The K3 filters are not anchored in the vault structure. They are held or wedged in place in the duct piping. This allows them to be remotely changed with ease. It is highly unlikely that the filter system would survive the DBE. A system of structural retainers could be designed and installed to restrain the K3 filters if a DBE occurs. The filter retainers would need to be installed remotely and be removable for ease of remote filter replacement. The K3 filter fans have been qualified for NPH loads.

In the 225-B Building, there are two overhead bridge cranes and an electric powered catwalk. A 15-ton process cell canyon crane is in Area 2. There is a 10-ton pool cell crane in Area 3. The catwalk, also in Area 3, spans the pool cells. These are all configured and equipped with devices to prevent unrestrained movements and derailing due to seismic motions. Original seismic design efforts were conducted for the crane support systems. For this survey, these were evaluated and additional confirming seismic analyses were made. The catwalk, not original WESF equipment, is a bridge-crane-like structure designed for a 75 psf live load. The investigating engineers conclude that both overhead cranes, the catwalk, and their support systems have a high probability for surviving the DBE.

4.0 BUILDINGS AND STRUCTURES

The following sections describe the WESF structures (Figures 1 and 2) including the structural system each has for resisting vertical and lateral forces. The features that affect the resistance of each structural system to NPH are discussed. The investigating engineers assigned probabilities (high, moderate, or low) to the capability of the structure of being able to safely withstand the design basis NPH. These probabilities are based on the review of the existing structural documentation and the engineering judgement of the investigating engineers. The probabilities of NPH survival as judged by this survey for the WESF buildings and structures and the adjacent B-Plant canyon west end wall are shown in Table 2.

TABLE 2

225-B WESF BUILDINGS AND STRUCTURES, ESTIMATED PROBABILITY OF SURVIVAL

Structure	Estimated Probability of Survival		
	DBE	Wind	Ashfall
225-B Building Area 1 Instrumentation and HVAC Area	H	H	H
225-B Building Area 2 Process Cells	H	H	H
225-B Building Area 3 Pool Cells	H	H	H
225-BB Building K3 Filter Building	H	H	H
225-BA Building K-1-1 Filter Building	H	H	H
221-B Building B-Plant Canyon End Wall	L	H	H (No direct exposure)

H = High probability, defined as having a 90% or better chance of survival.
M = Moderate probability, defined as having a 50% to 90% chance of survival.
L = Low probability, defined as having a less than 50% chance of survival.

4.1 225-B BUILDING

The reinforced-concrete 225-B Building has plan dimensions of approximately 156 by 91 ft. It is 41 ft above grade at the highest point and 16 ft below grade at the lowest point. The 225-B Building is contiguous with, but structurally independent of the existing 221-B Canyon Building to the east. A 1/4 in. expansion joint exists between the 225-B Building and the 221-B Building at all elevations. The original project design included NPH analyses for the 225-B Building. The onsite architect engineer, VITRO Engineering Corporation, conducted the necessary UBC seismic analysis, wind load analysis, and soil pressure analyses for retaining walls in the pool cells, plus the 225-BB K3 and 225-BA K-1-1 Filter Buildings; and hydrodynamic (seismic sloshing) analysis for the pool cells (ref 6). Further seismic analyses and design were conducted by an offsite firm, J.A. Blume and Associates (ref 1 and 2). Section 6 reports the technical review conducted for the references 1 and 2 analyses as part of this survey.

The 225-B Building is divided functionally and structurally into three areas. Area 1 has the control and surveillance instrumentation for the pool cells, the HVAC mechanical room, and offices. Area 2 has the waste encapsulation process cells. Area 3 has the capsule storage pool cells. For this survey, based on reference 13 (see Section 1.2), Area 2, Area 3, and the K3 Filter Building were assessed as Safety Class 1 facilities; Area 1 was assessed as a Safety Class 3 facility.

4.1.1 225-B Building, Area 1

Area 1 is a one-story abovegrade reinforced masonry wall structure with a metal deck diaphragm roof supported on open-web steel joists and steel beams (drawing H-2-66415). The foundation consists of continuous and spread footings at a depth of 3 ft below the grade slab, bearing on compacted fill (drawing H-2-66416). Area 1 is structurally isolated from Area 2, Area 3, and the 221-B Building by 1/4 in. expansion joints at the foundation and base slab, and by 2-in. gaps at the walls and roof (drawing H-2-66416). The roof deck joists and beams bear on

Area 1 masonry shear walls at the north and east, on the Area 3 reinforced-concrete wall at the west, and on bearing brackets on the Area 2 reinforced concrete wall and the 221-B Building wall at the south. Slotted holes in both the north/south and the east/west directions are provided in the bearing brackets on the Area 2, Area 3, and 221-B Building walls which allow a 2-in. (seismic) joint between Area 1 roof and the adjacent structures (drawing H-2-66415). Thus, Area 1 is structurally independent of the adjacent structures for resisting lateral seismic and wind loads. However, the Area 1 roof is dependent on the adjacent structures for resisting vertical gravity loads including dead, live, snow, and ashfall.

Area 1 design calculations indicate that the building was designed for the UBC seismic and wind loads (ref 6). The building was further evaluated for an OBE (see Section 6.2.1) which qualified the Area 1 building for approximately twice the UBC seismic loads (UBC seismic load = .067 W [ref 6], OBE seismic load = .15 W [ref 1], .15W/.067W = 2.2) (ref 1). The lateral seismic and wind forces are resisted by the reinforced masonry shear walls on the north and east and interior of the building. The Area 1 roof was designed for dead loads plus a live load of 20 psf.

Area 1 of the 225-B Building is designed for UBC seismic, wind, and live loads including snow (ref 6). The building is further analyzed and qualified for seismic loads of approximately twice the UBC. Therefore, the building qualifies for NPH resistance as a Safety Class 3 facility (ref 13). The building has not been evaluated for Safety Class 1 NPH loads of the DBE, 90 mph wind, or ashfall loads defined in reference 13. Based on the seismic analysis completed for OBE loads using 2% damping and allowable stresses appropriate for normal operating conditions, Area 1 is judged to have a high probability of surviving the DBE (ref 1). The roof of Area 1, designed for a live load of 20 psf, is judged to have a high probability of surviving the snow plus ashfall loading condition (ref 6). Based on the lateral load capacity to resist seismic loads, Area 1 is judged to have a high probability of surviving the 90 mph winds.

4.1.2 225-B Building, Area 2

Area 2 is the largest and most massive of the three 225-B Building areas. Area 2 is a two-story abovegrade structure with a reinforced concrete roof and floor slabs supported by reinforced concrete shear walls (drawings H-2-66417 through H-2-66420). The high density concrete shielding hot cells provide additional shear walls for resisting lateral loads on both the first and second floors (drawing H-2-66423). The building foundation consists of continuous and spread footings at a depth of 6 ft below the grade slab, bearing on compacted fill (drawing H-2-66420). The high density concrete shielding hot cells have a separate foundation, a continuous base mat slab on compacted fill, 6 ft below the grade slab (drawing H-2-66423). The east end of the foundation spread footings, grade slab, and floor slab of Area 2 are separated from the 221-B Building by a 1/4-in. expansion joint. The walls and roof of Area 2 are separated from the 221-B Building by a 2-in. gap (drawings H-2-66418 and H-2-66420) identified on the design drawings (drawing H-2-66415) as a seismic joint. The 2-in. gap, or seismic joint, allows relative motion between the structures without load transfer, and prevents impact or seismic pounding.

The Area 2 and Area 3 foundations are continuous and structurally connected at all common edges below grade (drawing H-2-66422). A portion of the second floor of Area 2 between column lines C and E and between column lines 2 and 3 cantilevers out over the Area 3 pool cells. The Area 2 reinforced concrete shear walls and floor slabs above grade, including the portion cantilevered out over Area 3, are separated from Area 3 by a 2-in. gap (seismic joint, drawing H-2-66415). Therefore, Area 2 is structurally independent of the adjacent structures for resisting lateral seismic loads, wind loads, and vertical gravity loads including dead, live, snow, and ashfall.

Area 2 design calculations indicate that the building was designed for the UBC seismic and wind loads (ref 6). The building was further evaluated and designed for a 0.12 gravity OBE and a 0.25 gravity DBE (ref 1). As the wind loads are much less demanding than the seismic design requirements, the structure is adequate for wind resistance (ref 6). The Area 2 roof, designed for dead loads plus a live load of 20 psf (ref 6), is judged to have a high probability of surviving the snow plus ashfall loading conditions. The 3-ft thick process cell shielding walls enhance the lateral load capacity. The shielding view windows in the process cell walls do not appreciably weaken the walls. Precautions have been taken to ensure that the windows remain in place when subjected to seismic forces (see Section 5.7).

The Area 2 structure has been designed to survive the 0.25 gravity DBE and 90 mph winds and, accordingly, has been assigned a high probability for surviving these NPH.

4.1.3 225-B Building, Area 3

Area 3 is a two-story structure. The first story pool cells are below grade (drawings H-2-66421 and H-2-66422). The reinforced concrete roof and pool cell floor slab at grade are supported by reinforced concrete shear walls. The foundation consists of a reinforced concrete mat on undisturbed soil at approximately 16 ft below grade and soil retaining shear walls. The concrete foundation for the pool cells is 1'-9" thick. The concrete foundation for the pipe trench is 1'-6" thick. The concrete foundation for the transfer aisle is 2'-11" thick. (See Figures 2 and 4.) The Area 2 and Area 3 foundations are continuous and structurally connected at all common edges below grade. The Area 3 abovegrade shear walls and roof are separated from Area 2 by a 2-in. (seismic) joint (drawing H-2-66415). Thus, the Area 3 abovegrade structure is independent of the adjacent structures for resisting lateral seismic and wind loads and vertical gravity loads including dead, live, snow, and ashfall. The Area 3 belowgrade structure (the pool cell area) is structurally connected to the Area 2 foundation (drawing H-2-66422). This will enhance the lateral load capacity for resisting earthquake motions.

Area 3 design calculations indicate that the building was designed for the UBC seismic and wind loads (ref 6). The building was further evaluated and designed for a 0.12 gravity OBE and a 0.25 gravity DBE (ref 1). As the wind loads are much less demanding than the seismic design requirements, the structure is adequate for wind resistance (ref 6). The Area 3 roof, designed for dead loads plus a live load of 20 psf (ref 6), is judged to have a high probability of surviving the snow plus ashfall loading conditions.

The Area 3 structure has been designed to survive the 0.25 gravity DBE and 90 mph winds and accordingly has been assigned a high probability for surviving these NPH.

4.2 225-BA, K-1-1 FILTER BUILDING

The 225-BA, K-1-1 Filter Building, is a one-story, abovegrade, 39- by 17- by 11-ft high structure with a reinforced concrete roof slab supported by reinforced concrete shear walls. The foundation is a continuous footing at 2.5 ft below the grade slab. The building is structurally independent of the adjacent structures.

The design calculations indicate that the building was designed for the UBC seismic and wind loads (ref 6). The building was further evaluated and designed for a 0.125 gravity OBE and a 0.25 gravity DBE (ref 1). As the wind loads are much less demanding than the seismic design requirements, the structure is adequate for wind resistance. The 225-BA, K-1-1 Filter Building roof, designed for dead loads plus a live load of 20 psf (ref 6), is judged to have a high probability of surviving the snow plus ashfall loading conditions.

The 225-BA, K-1-1 Filter Building, has been designed to survive the 0.25 gravity DBE and 90 mph winds and, accordingly, has been assigned a high probability for surviving these NPH.

4.3 225-BB, K3 FILTER BUILDING

The 225-BB, K3 Filter Building (drawing H-2-66433) is a 38- by 10- by 17-ft high reinforced concrete structure. The building is a one-story belowgrade retaining wall structure or vault with five cells, four open at the ground surface and one opens 6 ft above the ground surface. The tops of the cells are normally closed with cover blocks. The foundation is a continuous reinforced concrete slab 10 ft below grade. The K3 filter vault is structurally independent of the adjacent structures.

The 225-BB, K3 filter vault design, based on the references 1 and 6 analyses, qualifies for the 0.25 gravity DBE, 90 mph winds, and volcanic ashfall plus snow load. Accordingly, it is assessed to have a high probability for surviving these NPH.

4.4 221-B BUILDING END WALL SEISMIC INTERACTION STUDIES

The west end of the 221-B Canyon Building is next to the WESF 225-B Building at the east end of Area 2 (process cell area). The 221-B Building is a reinforced concrete structure about 810 ft long, 66 ft wide, and 77 ft high. The structure is embedded 24 to 26 ft below grade. The 225-B Building is structurally isolated from the 221-B Building by 1/4-in. expansion joints at the foundation and base slab and by 2-in. gaps at the walls and roof. Both buildings have ample shear walls in the east-west direction to resist strong seismic ground motions with very small deflections. Therefore, seismic pounding between these adjacent buildings is not a concern.

The west end of the 221-B Canyon Building, except for the belowgrade monolithic cell wall, is closed with cast-in-place unreinforced concrete infill wall panels. The infill wall panels are unbonded (not monolithic) with the 221-B Building structure. Paper joint materials were placed between the canyon structure and the edges of the infill wall panels to break the bond. Some of the unbonded joints have shear keys and others have mating flat surfaces. It appears that the purpose was to make the wall panels easier to remove for any future 221-B Building expansion.

A previous seismic analysis of the 221-B Building end wall concrete infill panels has been reported (ref 5). This analysis was conducted for east-west (out-of-plane) seismic forces on the infill wall panels. An initial finite element analysis showed that the shear keys on the perimeter of the infill walls would be over stressed. The shear keys were then assumed to be ineffective and the infill wall panels were reanalyzed as rigid bodies subject to out-of-plane rocking and overturning.

Reference 5 showed that the rigid end walls would be stable against overturning in the east-west direction. Reference 5 recommended further time-history seismic analyses to confirm this.

Reference 5 did not resolve the in-plane shear loading on the infill wall panels due to north-south seismic response of the 221-B Building. Reference 5 did recommend further seismic analyses to resolve the wall stresses resulting from the north-south seismic displacements of the 221-B Canyon Building. Earthquake experience data has shown that unreinforced concrete infill walls are vulnerable to seismic induced diagonal shear failures and collapse.

Seismic analyses for the 221-B Canyon Building showed that inelastic stresses will develop in the canyon walls at relatively low north-south seismic accelerations or motions (ref 3). The unreinforced infill end-wall panels would be forced to comply with the 221-B Building transverse, north-south, deflections. The resulting large diagonal shear forces could fail and collapse the panels. The demineralized water piping at this point, where it passes through the infill wall, would be in jeopardy if a DBE occurs. Only the portion of the west end infill wall panel that extends above the 225-B Building roof elevation has the potential to do significant local structural damage to the 225-B Building. The pool cells would be unaffected structurally by a collapse of the 221-B Building end wall.

5.0 SYSTEMS AND EQUIPMENT

The following sections describe the systems and equipment considered, including the anchorages and other features for resisting vertical and lateral forces. The dependency of the systems onsite water and electrical power and the proximity to other items whose failure could jeopardize the system are appraised. The investigating engineers assigned probabilities (high, moderate, or low) to the capability of the systems to safely withstand the design basis NPH. These probabilities are based on the review of the existing documentation and the engineering judgement of the investigating engineers. The probabilities of NPH survival for the WESF systems, as judged by this survey, are shown in Table 3.

TABLE 3

225-B BUILDING WESF SYSTEMS, ESTIMATED PROBABILITY OF SURVIVAL

System	Estimated Probability of Survival		
	DBE	Wind	Ashfall
Pool Cell Drain or Waste Water Removal	H	Not Exposed	Not Exposed
Pool Cell Demineralized Water Addition	L	Not Exposed	Not Exposed
Pool Cell Recirculation and Heat Removal	L	H	H
Process Cell Operating Gallery Fire Protection	M	H	H
K3 Filter	L	Not Exposed	Not Exposed
Alternating Current Power Backup	H	H	H
Process Cell Shielded Viewing Windows	H	Not Exposed	Not Exposed
Process Cell Bridge Crane	H	Not Exposed	Not Exposed
Pool Cell Bridge Crane	H	Not Exposed	Not Exposed
Pool Cell Powered Catwalk	H	Not Exposed	Not Exposed

H = High Probability, defined as having a 90% or better chance of survival.
M = Moderate Probability, defined as having a 50% to 90% chance of survival.
L = Low Probability, defined as having a less than 50% chance of survival.

5.1 POOL CELL DRAIN SYSTEM (ALSO KNOWN AS WASTE WATER REMOVAL SYSTEM)

Each pool cell is provided with a 2-in. diameter, Schedule 10S stainless steel waste water removal pipe extending from an open end at 1/2 in. above the pool cell floor, through the pool cell liner and wall into the pipe tunnel, and through the pipe tunnel ceiling into the pipe trench to a flange connection at 6 in. below the pool cell first floor grating (ref 30 and drawing H-2-67019) (see Figure 4). The pool cell waste water removal pipes from cells 1, 3, 4, 5, 6, 7, 8, and 12 are connected through valves to a header pipe located above the pool cell first floor grating where a steam jet is used to jet cell water to the low level waste drains. These low-level waste drain lines are encased in shielding concrete and located along the west exterior wall immediately above the pool cell first floor grating. The pool cell waste water removal pipes from cells 2, 9, 10, and 11 are not connected to the steam jet waste water removal system. The removal of waste water from the pool cells requires the action of the steam jet. The steam jet and low level waste drain pipes are located above the water level in the pool cells and, as such, siphon action cannot occur.

Each waste water removal pipe penetrates the pool cell liner and wall into the pipe tunnel at an elevation of 3 ft above the pool cell floor, extends horizontally 7-1/2 ft to the exterior wall, then vertically up 9 ft where it penetrates the pipe tunnel ceiling. The waste water removal pipe in the pipe tunnel is all welded pipe having no valves or flange connections. If the waste water removal pipe were to rupture in the pipe tunnel, the pool cell would be in jeopardy of draining down to the 3-ft level. If the waste water removal pipe were to rupture above the pipe tunnel ceiling, the pool cell would not be in jeopardy of draining. The pipe rupture would be above the water level in the pool cell.

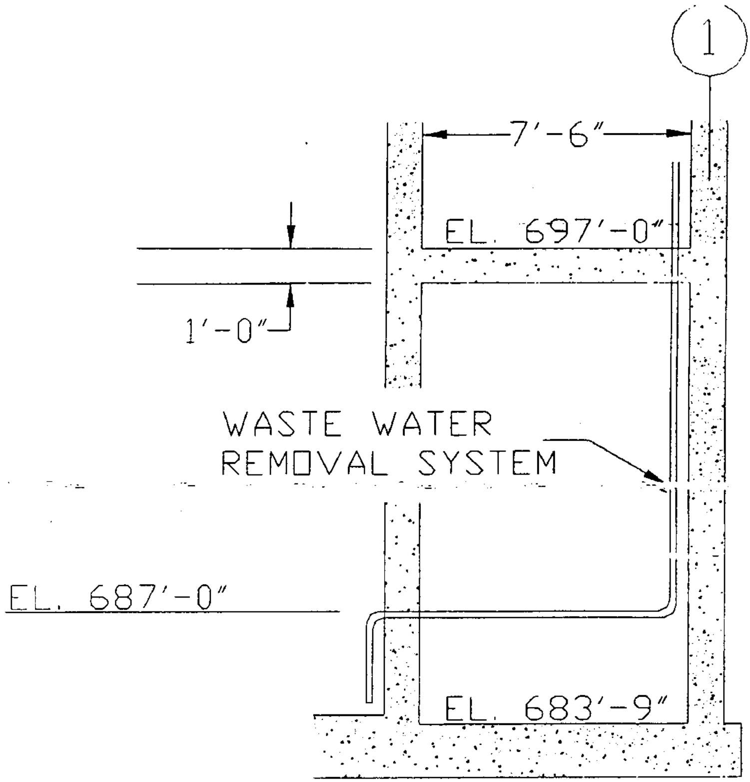


FIGURE 4, WASTE WATER REMOVAL SYSTEM

Neither pipe supports nor pipe hangers are specified on the design drawings. The waste water removal pipes in the pipe tunnel are not visible for inspection. A videotape of the pipe tunnel was viewed. No pipe supports or hangers exist in the pipe tunnel. The seismic evaluation of the waste water removal pipes, where they pass through the pipe tunnel, is part of this survey (see Appendix D). This conservative evaluation qualifies the waste water removal pipes for current DBE loads. Accordingly, they have been assigned a high probability for DBE survival.

5.2 POOL CELL DEMINERALIZED WATER ADDITION SYSTEM

The pool cell water addition system originally consisted of raw water supplied by a 3-in. carbon steel pipe header in the pipe trench and was distributed to the cells by 1-in. stainless steel pipes embedded in the cell divider walls approximately 8 in. above the water level in the pools. In 1982, the pool cell water distribution pipes were disconnected from the raw water header. The pool cell water distribution pipes for cells 1, 3, 4, 5, 6, 7, and 8 were reconnected to a new stainless steel demineralized water system pipe header in the pipe trench. Water is added to pool cell 2 from the pool cell 12 transfer aisle by opening the valve on the capsule transfer port near the bottom of the pool cell. The 1-1/2-in. stainless steel demineralized water pipe header is supported at 14-ft intervals on the existing pipe support racks in the pipe trench. The distribution pipes from the header to the existing pipes embedded in the cell divider walls are supported from the pipe trench floor approximately 2-1/2 ft from the header and 2-1/2 ft from the cell divider wall.

Demineralized water is supplied to the WESF from the 271-B Building. The demineralized water line routes through the 221-B Building and out the 221-B Building west end wall into the WESF AMU area. Demineralized water, supplied in a 2-in. diameter fiberglass pipe, routes through the process cell AMU area, the operating gallery, the airlock between the operating gallery and the pool cells, the HVAC room in Area 1, and the pool cell area to the distribution header.

The demineralized water addition pipes in the pool cell area are small and very ductile and have a high probability being functional following the DBE. The demineralized water supply line is routed in non-ductule pipes and is vulnerable to damage from adjacent pipes. The demineralized water line routes through buildings, including 271-B, 221-B, and the 225-B Building Area 1 HVAC room, that are not DBE qualified. The water addition system is judged to have a low probability of being functional if a DBE occurs.

5.3 POOL CELL RECIRCULATION AND HEAT EXCHANGER SYSTEM

Each pool cell has a cell water recirculation system. The pool cell recirculation system pumps heated water out of the top of the pool cells, circulates it through the heat exchangers, and reinjects the cooled water through distribution pipes in the bottom of the pool cells. Cells 1, 3, 4, 5, 6, and 7 have a heat exchanger system. The heat exchanger secondary or cooling system consists of raw water circulated once through the heat exchangers to a drain line.

There are 3-in. diameter Schedule 10S stainless steel recirculation water pipes from the pumps to the distribution pipes. Each 3-in. recirculation pipe route penetrates the pipe tunnel ceiling at the center of the tunnel and drops vertically 9 ft to 3 ft above the floor. Then, the pipe route extends horizontally 4 ft to a distribution pipe embedded in the cell wall.

Similar to the pool cell drain line (see section 5.1), the recirculation pipe in the pipe tunnel is all welded pipe having no valves or flange connections. Neither pipe supports nor pipe hangers are specified on the design drawings. The pipes in the pipe tunnel are not visible for inspection. A videotape of the pipe tunnel was viewed. No pipe supports or hangers exist in the pipe tunnel. The seismic evaluation of the pipes where they pass through the pipe tunnel is shown in Appendix D. This conservative evaluation qualifies the pipes for current DBE loads. Accordingly, they have been assigned a high probability for DBE survival.

The pool cell recirculation and heat exchanger piping systems including the heat exchangers and the pumps are seismically resistant. Design criteria were provided for 0.25-gravity DBE loads (ref 2). No design calculations were recovered. The pumps and heat exchangers are anchored and have a high probability of being functional following a DBE. However, the AC power and the raw water supply line required for operation of the recirculation and heat removal systems are vulnerable to the DBE.

5.4 225-B BUILDING FIRE PROTECTION SYSTEM

The 225-B Building fire protection system includes overhead sprinkler distribution lines in the Area 1 HVAC room and in Area 2 in the first floor operating gallery and in the second floor AMU room. The system is supplied from the underground raw water system on the north side of Area 1. The supply header and distribution lines are routed overhead across Area 1, across the operating gallery, up the stairwell to the second floor, and across the ceiling of the AMU room. The fire protection piping is suspended from the ceilings with vertical rod hangers. Some lateral (seismic) restraints to the walls and ceilings are provided. The hangers and lateral restraints likely were designed and installed to National Fire Protection Association and UBC standards.

The fire protection piping system is laterally braced and the areas of the 225-B Building where it is routed have a high probability of DBE survival. Therefore, the fire protection piping and sprinklers have a moderate probability of surviving the DBE. However, the underground raw water supply is likely to be interrupted if a DBE occurs. Thus, the 225-B Building fire protection system has a low probability of being functional following the DBE.

5.5 K3 FILTER SYSTEM

The K3 high efficiency particulate air (HEPA) filter system is located in the 225-BB K3 Filter Building. The exhaust air from the 225-B Building Area 2 process cells is routed through underground ducts to the 225-BB Building and these filters. The exhaust fans for air flow are downstream of the K3 filters. The K3 filters, as originally installed in the 225-BB Building, were unanchored and vulnerable to earthquakes. The stud anchor bolts on the leveling pads were removed as shown on as-built drawing H-2-66433.

The K3 filter system was upgraded in 1988. New filter assemblies were installed. The new filter assemblies are considered as replacement in kind, and, again, set unanchored on the leveling pads in the 225-BB Building. The filters are held or wedged in place in the duct piping. It is unlikely that seismic analysis would qualify the unanchored filters for DBE motions. The K3 filter system is judged to remain vulnerable to earthquakes with a low probability of functioning if a DBE occurs.

A direct buried 24-in. steel duct connects the 225-B Building to the K3 vault. An assessment for seismic loads on the underground ducts is included in reference 2. Stress conditions in the underground ducts were approximately calculated and found to be of no adverse consequence. Reference 2 concludes that excessive duct stresses would be expected only under the condition of localized vertical differential settlement. The primary concern was the post-construction building settlements. Reference 2 recommended that duct sleeves be installed where the ducts penetrate the building foundation walls. The 24-in. duct enters the building below the foundation grade beam (it does not penetrate a foundation wall), and no duct sleeve is needed.

5.6 AC POWER BACKUP SYSTEM

The 225-B Building AC backup electrical power is supplied by the diesel/electric generator located outside the 225-B Building north and west of the Area 3 pool cell building. The generator unit is set on a structural steel frame that is anchored to a concrete slab. The generator unit is housed inside a dampered metal enclosure. The diesel fuel system including the 1,000-gal fuel tank is located north of the generator unit. The AC backup power system has a high probability of being functional following the DBE.

The diesel fuel system for the AC backup power system could be damaged by a wind-generated 2 by 4 lumber missile. Volcanic ashfall and the potential for dust ingestion could affect the ability of the generator to function.

5.7 SHIELDED VIEWING WINDOWS

The process cells have a number of shielded viewing windows (CVI 14056). These include the large windows for remote operation viewing from the Operating Gallery. The windows in the Operating Gallery weigh about 14,000 lbs each. The rectangular window openings in the heavy density concrete process cell walls have embedded structural steel liners. The shielding window assembly is secured into its concrete-filled steel-plate frame structure with lead sheet shims and compacted lead wool on all sides. The frame structure with the window assembly is installed into the opening. The space between the frame and the steel liner is filled with compacted lead wool to secure the frame and window in the opening. The windows are sealed by bolt-on frames that hold flexible gaskets.

At the time of the WESF equipment seismic design review, the in-progress design for the shielding windows depicted the window frame structure with roller skids for ease of removal (ref 2). Bolted keeper plates to hold the windows in place were to be developed. The seismic review concluded that the keeper plates for each window should be designed for resisting a horizontal force of 30% of the window weight.

Subsequent to the reference 2 evaluation and recommendation, the window frame was redesigned and the roller skids on the frame structures were eliminated. Therefore, the bolted keeper plates were not required. Horizontal seismic forces on the shielding windows are resisted by the friction developed by the lead shims and compacted lead wool surrounding them. A static coefficient of friction of 0.3 minimum would be sufficient to resist the DBE forces. The original windows have been replaced. The jacking forces required to remove the original windows and install the replacements indicate that the lead compacted around the windows provides adequate friction to resist the DBE.

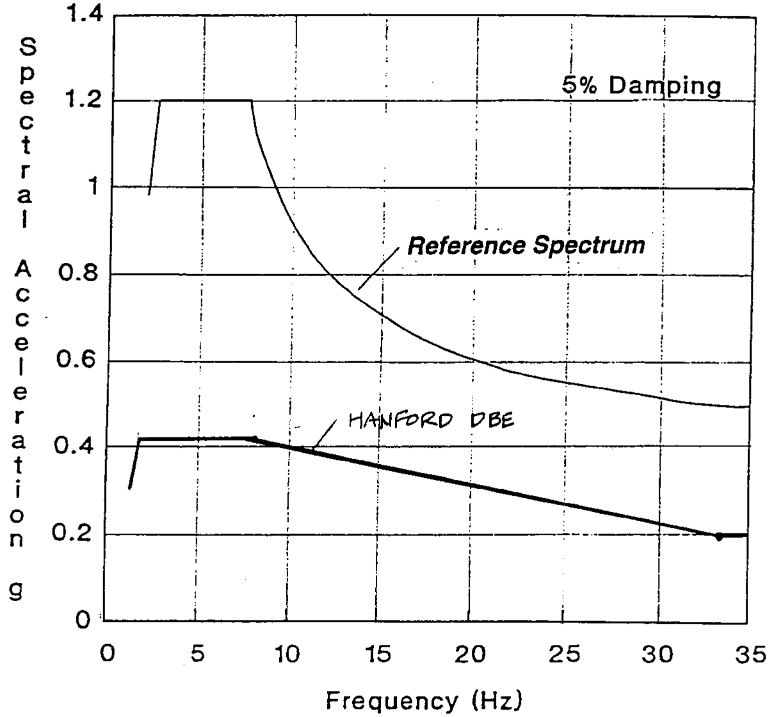
5.8 K1 AND K3 EXHAUST FANS

The WESF K1 and K3 exhaust ventilation systems are routed through underground ducts to HEPA filters in the 225-BA (K1) and 225-BB (K3) buildings. Air flow is provided by exhaust fans downstream of the filters.

The K1 and K3 exhaust fans are qualified for DBE forces. This is based on a comparison of the K1 and K3 fans to the seismic experience data base for commercial nuclear power plants found in reference 23. The procedure outlined in reference 23 for seismic qualification of existing equipment is to compare the existing equipment to equipment known to have survived earthquakes (the earthquake experience data base). When the existing equipment satisfies the inclusion and exclusion rules of the data base, the equipment is considered to be qualified to withstand the earthquake ground motions that would be enveloped by the reference earthquake response spectrum for the data base.

The reference response spectrum (earthquake) for the reference 23 data base envelopes the Hanford DBE at all frequencies (see Figure 5). Therefore, the equipment that meets the rules of the data base are qualified for the Hanford DBE. The reference 23 inclusion and exclusion rules are discussed below.

- The fans are similar to and bounded by the fans described in the data base. The fans included in the data base are axial and centrifugal fans having air flows up to 50,000 ft^3/min, differential pressure up to 12 in. of water, and drive motors up to 200 horsepower. The two K1 fans and the two K3 fans are centrifugal fans. The K1 fans are rated at 16,650 ft^3/min flow volume, 1 in. of water differential pressure, and have a 50-horsepower drive motor. The K3 fans are rated at 8,000 ft^3/min flow volume, 8 in. of water differential pressure, and have a 60-horsepower drive motor.



Frequency (Hz)	2.0	2.5	7.5	8.0	10	12	16	20	28	33
Spectral Acceleration (g)	.98	1.2	1.2	1.13	.90	.80	.68	.59	.53	.50
HANFORD DBE	.42			.42						.20

FIGURE 5, SEISMIC CAPACITY REFERENCE SPECTRUM

- The drive motors and fans are each connected by a common base which will limit differential displacement. The shafts are supported at the fan and at the drive motor. The concern is that differential displacement between the drive motor and the fan may cause shaft misalignment.

- The ducts leading to the fans from the filters and from the fans to the stack are flexible and supported so that the possibility of duct distortion during an earthquake will not bind or misalign the fans.

The fans are anchored adequately to remain in place during the DBE. The K1 fans have eight 3/8-in. diameter bolts anchoring the frame to the concrete slab (two bolts at each corner spaced 5 in. apart and 57 in. from corner to corner in the north-south direction). Using a conservative estimate for the center of gravity of 57 in. above the base slab, a conservative estimate of 1,000 lb for the weight of the fan assembly, and a conservative estimate of the seismic acceleration of 0.42 gravity (the peak of the 5% damping, data base response spectrum), a bolt tension force T and a bolt shear force V is calculated:

$$T = \{ (0.42 \times 1{,}000 \text{ lb} \times 57 \text{ in}) / 57 \text{ in} \} / 4 = 105 \text{ lb per bolt}$$
$$V = (0.42 \times 1000 \text{ lb}) / 8 = 53 \text{ lb per bolt}$$

The minimum capacity for cast-in-place or expansion anchors of 3/8 in. diameter is 1,400 lb in both tension and shear. The anchor bolts for the K1 fans are adequate.

The K3 fans have four, 1/2-in. diameter bolts anchoring the frame to a concrete grout pad (four bolts on each side spaced 24 in. apart and 33 in. from side to side in the east-west direction). These are likely anchors that penetrate through the grout pad into the structural concrete beneath. Anchorage installed only in a grout pad have failed in earthquakes comprising the experience data. Using a conservative estimate for the center of gravity of 66 in. above the base slab, a conservative estimate of 1,000 lb for the weight of the fan assembly, and a conservative estimate of the seismic acceleration of 0.42 gravity (the peak of the 5% damping, data base response spectrum), a bolt tension force T and a bolt shear force V is calculated:

T = { (0.42 x 1,000 lb x 33 in) / 66 in } / 4 = 210 lb per bolt
V = (0.42 x 1000 lb) / 8 = 53 lb per bolt

The minimum capacity for cast-in-place or expansion anchors of 1/2 in. diameter in structural concrete is 2,300 lb in both tension and shear. The anchor bolts for the K3 fans, if imbedded in the structural concrete beneath the grout pad, are adequate.

5.9 CRANES AND SUPPORT SYSTEMS

There are two overhead bridge cranes and an electric powered catwalk on rails in the 225-B Building. A 15-ton capacity process cell canyon crane runs above the second floor in Area 2. In Area 3, there is a 10-ton overhead crane located above the pool cells. The catwalk, also in Area 3 at the deck level, spans the pool cells.

The fundamental components for each crane are the crane bridge, trolley, hoist, rail girders, and the lifted load. They are configured and equipped with devices to prevent unrestrained movements of the cranes and derailing if strong seismic motions occur. The crane bridge, trolly, and hoist were designed by the crane manufacturer (CVI 14019 and CVI 14027). The cranes are original WESF construction and seismic design efforts were conducted for the crane support systems (ref 6). These systems were evaluated, and additional confirming seismic analyses for the crane supports are provided in Appendix E.

Design analyses were conducted for the rail girders and their anchored supports (ref 6). These design analyses presumed a 25-ton capacity Area 2 canyon crane and a 15-ton capacity Area 3 pool cell crane. Horizontal and vertical impact loads based on the UBC seismic provisions (ref 11) were included for seismic design. The lifted load was considered to contribute to the horizontal loads.

Subsequently, a simplified procedure for determining the 0.25 g DBE and 0.12 g OBE design forces at the rail girder support points was developed and used (ref 2). Equivalent spring mass systems for the cranes were studied to estimate the maximum seismic equivalent static forces at the crane supports. Due to the flexibility of the hoisting cables, the lifted load was not considered to contribute to the horizontal seismic loads.

Revisions or updates for the reference 6 analyses for the reference 2 DBE and OBE forces and the lower capacity cranes installed were not located. Therefore, in Appendix E, the reference 2, 0.25 g DBE forces are compared to those used for the Reference 6 analyses. For the Area 2, 15-ton canyon crane, the reference 6 seismic design forces govern. However, the opposite was determined for the Area 3, 10-ton pool cell crane. The investigating engineers conducted additional seismic analyses that confirm that the 10-ton crane support system is adequate for the DBE forces. The engineers conclude that both WESF overhead cranes and support systems have a high probability for surviving the DBE.

The catwalk on rails over the pool cells is not original WESF equipment, and is not addressed in the references 2 or 6 design calculations. Design calculations for vertical loads are included (seismic loads were not included) in CVI 22637. The catwalk, designed and constructed in 1994, is a bridge-crane-like structure designed for a live load of 75 lb/ft^2. The catwalk is stable due to having a low center of gravity and a wide wheel base. The catwalk drive gears restrain movement and the wheels are flanged to prevent derailing if strong earthquake motions occur. The investigating engineers conclude that the catwalk has a high probability for surviving the DBE.

6.0 NATURAL PHENOMENA STRUCTURAL DESIGN AND ANALYSES REVIEW

A technical review of the existing original seismic and other NPH design analyses for the WESF 225-B Building was conducted during this NPH survey. John A. Blume and Associates, Engineers reports (ref 1 and 2) and the VITRO design calculations (ref 6) were evaluated to determine whether the reports are adequate enough to demonstrate that the facility is in compliance with design basis criteria, particularly the seismic criteria. The overall completeness and correctness of the documents were evaluated for determining the adequacy of the design basis documentation for the mission of storage of strontium and cesium capsules in the storage pool cells. The analysis elements subjected to technical review include:

- Design categories
- Earthquake criteria
- Structural material properties and degradation
- Soil-structure-interaction
- Analysis procedure
- Structural analysis models
- Piping and equipment
- Resolution of structural design modifications
- Resolution of equipment design modifications

6.1 DESIGN CATEGORIES FOR THE WESF STRUCTURES

The WESF, including the 225-B Building, design and construction precede the current Hanford Site (structural) design categories Safety Class 1, 2, 3, and 4 in SDC 4.1 reference 13, or Safety Class and Safety Significant found in WHC-CM-4-46, Revision 2 (ref 29). The three design category Types A, B, and C, later included in reference 16, and also, based on safety (the importance of the function with regard to public health and safety) were specified for the original WESF design (ref 1).

Type A facilities were those whose failure or malfunction might cause or increase the severity of an accident that would affect the public health and safety. Also included as Type A facilities were those necessary for the safe shutdown and isolation of the processes involving radioactive materials.

Type B facilities were those facilities, other than Type A facilities, that were necessary for continued operation without undue risk to the health and safety of the public.

Type C facilities were those facilities that were not necessary for continued safe operation or safe shutdown and that were not Types A or B.

Facilities designated as Type A are the waste encapsulation process cells, Area 2; the capsule storage pool cells, Area 3; and the 225-BB K3 and 225-BA K-1-1 Filter Buildings. Area 1, the instrumentation room, HVAC mechanical room, offices, lunch room, and locker room was designated as a Type B facility. For this NPH survey and technical review of reference 1, 2, and 6 analyses, the Type A facilities were compared to the design requirements for the Safety Class 1 category using reference 23. Area 1 was compared to the design requirements for the Safety Class 3 category using reference 13.

6.2 EARTHQUAKE CRITERIA

Often existing facilities originally were designed and evaluated for seismic criteria that are very different and less demanding than the subsequent current criteria. The earthquake criteria used for the reference 1 design evaluation of the WESF 225-B Building are different from the current criteria in reference 13. However, the seismic demands used for the reference 1 analyses are greater (at least double) than the comparable current requirements. The reason is that reference 13 allows, in combination, a lower peak ground acceleration, higher damping, and a ductility ratio for inelastic energy absorption. Therefore, as further explained below, the reference 1 seismic analyses and results, in general, are more demanding than required by subsequent changes in the Hanford Site seismic design criteria.

6.2.1 Peak Ground Acceleration

In reference 6, the WESF structures were designed originally for seismic loads in conformance with the UBC, the minimum site design criteria in effect in 1971 (ref 15). In reference 1, the Type A structural designs were analyzed for a DBE having a peak ground acceleration of 0.25 gravity and an operating basis earthquake (OBE) having a peak ground acceleration of 0.12 gravity. In reference 1, the Area 1 Type B structure was evaluated for the OBE, but not the DBE. This is in accordance with reference 16, the Hanford Site design criteria being developed at the time. The DBE and the OBE in reference 16, and used for the design of WESF, were those developed and used for the design of the Hanford Site Fast Flux Test Facility.

The DBE was defined in reference 1 as the ground motion to be resisted without loss of structural integrity of structural elements necessary to shutdown and maintain the plant in a safe condition without undue risk to the health and safety of the public. The OBE was defined in reference 1 as the ground motion to be resisted without loss of function of the elements necessary for continued operation of the plant without undue risk to the health and safety of the public.

Reference 13, the current Hanford site design criteria document for facilities, includes natural phenomena criteria. Reference 13 specifies that Safety Class 1 structures be designed for a DBE having a peak ground acceleration of 0.20 gravity, and that Safety Class 3 structures be designed in accordance with the UBC. Reference 13 does not require Hanford Site structures to be designed for an OBE. The Figure 6 plots are response spectra for the reference 1 DBE, the reference 1 OBE, the reference 13 DBE, and the reference 31 DBE. Reference 31, a seismic hazard study completed February 1996, is not implemented yet for compliance at Hanford. The DBE spectrum from reference 31 for 200-East Area is included for comparison with the WESF seismic design criteria. Note that there are different damping values for these spectra. See Section 6.2.2 for a discussion on damping values. It is evident that the seismic ground acceleration criteria used in the design of the WESF structures is greater than current requirements.

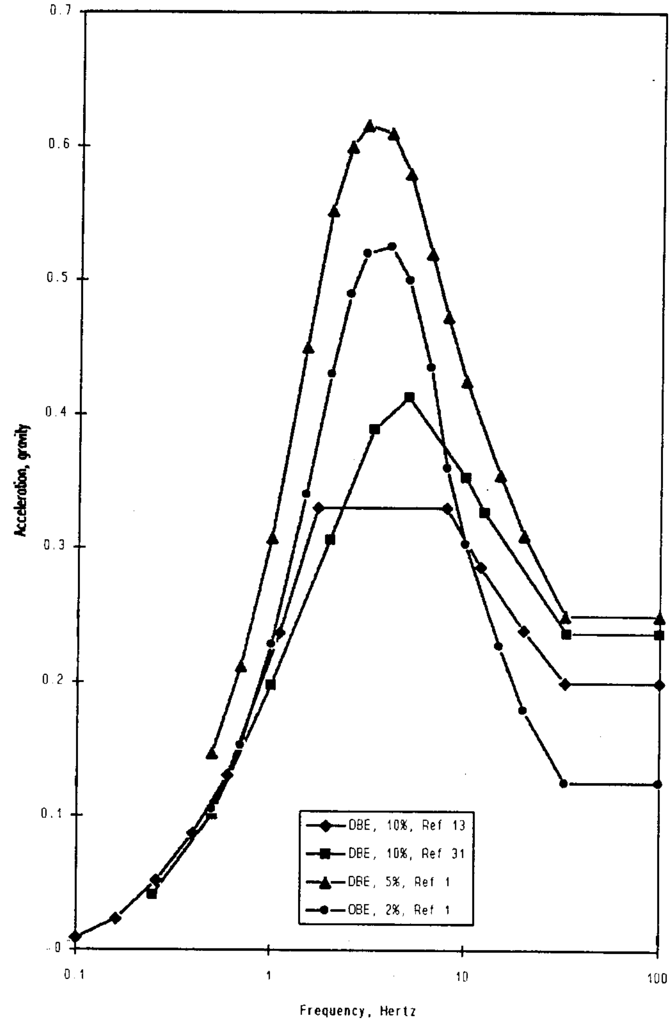


Figure 6. Design Response Spectra Comparison

6.2.2 Damping

For structural dynamic analyses, damping is a measure of forces and energy dissipation which oppose dynamic vibrations. The mathematical models used for the dynamic seismic analyses of the WESF structures were lumped-mass systems with damping. The predicted building seismic response (forces, stresses, deflections, etc) will be higher with a lower percent of damping (less energy dissipation) used for the dynamic analysis (see Figure 6). The current reference 13 seismic criteria allows up to 10% damping for the DBE analysis of existing reinforced concrete shear wall (WESF Areas 2 and 3) structures. The reference 1 seismic analyses of the WESF Areas 2 and 3 Type A structures limited the damping to 5% for the DBE and 2% for the OBE. The higher damping permitted by the current reference 13 criteria is based on earthquake experience data obtained since the 1970s. It is evident that the seismic demands used for the design of the WESF significantly exceed the current site requirements.

6.2.3 Ductility Ratio

Structural ductility is the ability of a structure to withstand inelastic, energy absorbing deformations. If the configuration, joints, structural materials, and other details of a building structure provide ductility, the structure would be able to sustain seismic response beyond the elastic limits, and experience large inelastic energy absorbing deformations. Therefore, the seismic response for a ductile structure could exceed the elastic capacity by a finite amount and remain functional. The ductility ratio (F_u) in the current reference 13 criteria is a seismic analysis factor by which the calculated seismic response is allowed to exceed the elastic capacity. The permitted F_u value for reinforced concrete shear wall (Areas 2 and 3) structures is 1.4. It is based on earthquake experience data. A comparable ductility factor was not used for the reference 1 WESF seismic design analyses. Therefore, it appears that the original seismic design for the WESF structures has an additional safety factor of about 1.4.

6.3 STRUCTURAL MATERIAL PROPERTIES AND DEGRADATION

Reference 1 and the design drawings (see reference drawing list section 7.B) were reviewed for the structural properties of materials used in the design and analysis of the reinforced concrete structures. The investigating engineers determined that the appropriate material properties were used for the reference 1 analyses (see Table 4).

TABLE 4

MATERIAL PROPERTIES USED FOR REFERENCE 1 ANALYSIS	
Material	**Density**
Normal density concrete	f_c' = 3,000 psi, wt = 145 pcf, E = 3,220,000 psi, Poisson's Ratio = 0.20
Heavy density concrete	f_c' = 3,000 psi, wt = 228 pcf, E = 4,720,000 psi, Poisson's Ratio = 0.20
Axial reinforcing steel	Fy = 60,000 psi
Reinforcing steel stirrups, ties, etc.	Fy = 40,000 psi

The WESF was evaluated further during this NPH hazards survey and reference 1 and 2 were reviewed for potential in service reductions in these material properties due to degradation. These efforts and the results reported below show that the material properties used for the WESF design and analyses remain appropriate.

In Area 2 of the 225-B Building, a radiation leak from the Process Cell E into the Service Gallery developed in 1980. The process cells have 14 gage Inconel sheet floor liners (reference drawing H-2-66423). In Cells D and E, holes developed in the floor liners resulting from damage during operations. Damage or degradation was suspected for the reinforced concrete floors and walls exposed to the leaks. The material degradation was evaluated.

Pulse echo (sonic) nondestructive testing (NDT) for concrete degradation was completed by the Portland Cement Association in 1981 (ref 7). The NDT was first conducted in the Operating and Service Gallery on the floors and cell walls along Cells C, D, E, and F. Then, the NDT was conducted remotely on the floors of these four cells. The cell walls are heavy density concrete. The floors are normal density concrete.

In the area of the leak from E Cell into the Service Gallery, extreme but local degradation was found in the gallery floor and the cell wall. The NDT indicated a 3 ft long, 3 in. thick severe concrete degradation at the construction joint at the base of the cell wall. In D and E Cells, the NDT indicated 3/4 in. depth of top floor slab damage. Only very minimal concrete degradation was found in the other floor and wall areas tested.

The holes in the process cell liners were repaired and the leaking ceased (ref 8). However, the concrete deterioration in the leak area was expected to continue. Therefore, in July 1982, additional sonic NDT was conducted (ref 9). The concrete degradation in the cell wall had increased upward to about 5 or 6 in. above the construction joint. Some areas on the inside of the cell walls showed 1/2 in. of concrete delamination not detected by the 1981 NDT.

It was concluded and reported that the process cell walls had no significant reduction of structural capacity due to the concrete degradation. It was determined that the Cell D and E floor slabs still had adequate capacity to carry design loads. The investigating engineers agree.

In 1982, it was necessary to conduct sonic NDT for concrete degradation in the ceiling slabs for the Operating Gallery, Service Gallery, and the two manipulator shops. There was visible concrete cracking in some of the anchor areas for the monorail hangers. The NDT found microcracks and cracks in many of these anchor areas. The worst concrete damage of this type was in the Operating Gallery near curves in the monorail. Also, through-depth cracks were found in the ceiling slabs for the two galleries and one of the manipulator shops (ref 9).

Epoxy injection repairs were recommended and made for the concrete ceiling slab damage found by NDT in order to restore structural capacity. The epoxy repairs were inspected and confirmed with follow-on NDT (ref 10). Lateral bracing for the monorail was installed to relieve lateral loads, and avoid redamage to the repaired concrete around the anchors.

6.4 SOIL-STRUCTURE-INTERACTION

Reference 1 studied the translational and rocking effects of the foundation of the WESF 225-B Building due to soil-structure-interaction. Reference 1 concluded that the soil-structure-interaction effects would be small and that refinement of the evaluation was not justified. The investigating engineers concur, based on related earthquake experience data.

6.5 ANALYSIS PROCEDURE

For the reference 1, Area 1 seismic (OBE) analysis, the structure was analyzed as a one-mass system lumped at the roof. The fundamental natural period or frequency of this system was calculated. The maximum roof level seismic acceleration response was determined from the OBE design spectrum using the fundamental period. The equivalent lateral static load for this response was applied at the roof level of the structure. The horizontal shear forces were distributed to the masonry walls. The seismic stresses for the walls were calculated, combined with the nonseismic stresses, and compared to the working stress limits for the masonry in accordance with reference 16 concurrently developed during the analysis. There are no current OBE design requirements in reference 13.

The minimum current seismic design requirements for the Area 1 structure would be the UBC seismic provisions (ref 11). This review concludes that the reference 1 seismic analysis procedure is adequate to show that Area 1 complies with the reference 13 Safety Class 3 requirements. Further, this review concludes that, with additional evaluation, Area 1 likely could be shown in compliance with the current reference 13 Safety Class 1 DBE requirements. One reason is that the reference 13 allowable DBE stress limits significantly are higher than the working stress limits conservatively used for the reference 1 OBE analysis.

For Areas 2 and 3, reference 1 reports dynamic time-history seismic analyses for the DBE and OBE. The calculated dynamic shear forces and overturning moments in the walls are tabulated for both structures. These seismic force and moment results were combined with the nonseismic forces and used to design the structures. The OBE stresses, similar to Area 1, were compared to the working stress limits for reinforced concrete. The DBE stresses were allowed to approach the ultimate stresses for reinforced concrete. There are no current OBE design requirements in reference 13. The reference 13 DBE allowable stresses for structures whose failure could affect public health and safety are comparable to the reference 1 DBE limits for Areas 2 and 3. This review concludes that the reference 1 seismic analysis procedure is adequate to show that Areas 2 and 3 comply with reference 13 Safety Class 1 requirements.

6.6 STRUCTURAL ANALYSIS MODELS

Reference 1 provides a simplified description of the Area 2 and Area 3 structural seismic analysis models used in the analysis. The analysis models are sketched in Figure 7. These two analysis models reviewed for completeness, correctness, and applicability. The review included the approach and methods used to idealize or consider the foundations, translational and torsional response, pool cell cover blocks, and pool cell walls.

6.6.1 Foundation

Areas 2 and 3 were analytically modeled for the reference 1 seismic analyses as cantilevered lumped-mass, structurally independent systems fixed at a rigid base at the foundation level of the respective structures (elevations 702 and 684 ft for

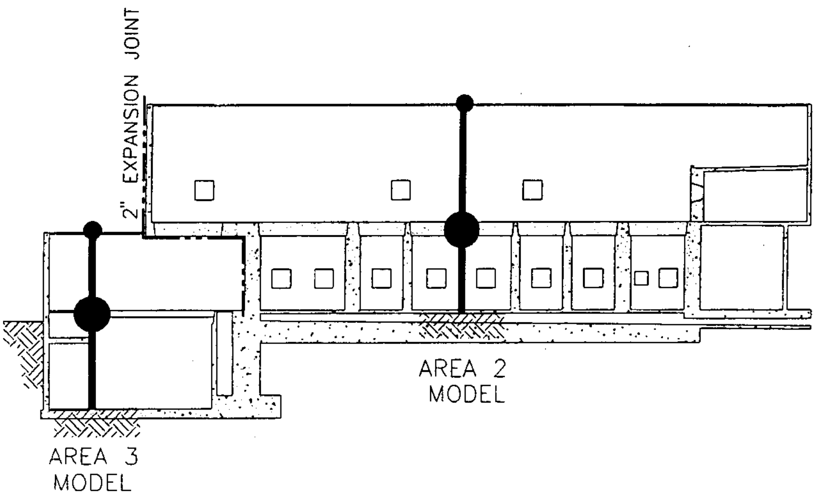


FIGURE 7, SEISMIC ANALYSIS COMPUTER MODEL

Areas 2 and 3, respectively). There is a 2-in. wide gap or seismic separation between the structures above elevation 702 ft. However, the west end of the foundation for Area 2 is monolithic (structurally connected) with the east wall of Area 3 below elevation 702 ft (as-built drawings H-2-66421 and H-2-66422). The lateral restraint this foundation connection provides for Area 3 is conservatively omitted from the independent analytical model. If this restraint were included in the analytical model, the reference 1 dynamic analyses would show the seismic response for the Area 3 pool cells below elevation 702 ft equal to the free field ground motions, not amplified as the results and the reference 2 first floor response spectra indicate. The idealization of the foundations in these analysis models is conservative and applicable.

6.6.2 Translational and Torsional Response

The nodes for the Area 2 and Area 3 seismic analysis models are at the building floor levels. The values for each lumped-mass at the nodes are tabulated in Figure A12, reference 1. The effective shear area and moment of inertia for the force-resisting elements between the nodes are also tabulated in Figure A12. This tabulated information shows that the models accommodate both north-south and east-west translational seismic response. The models and the information provided correctly depict, for each structure, which direction has the most lateral stiffness for resisting translational response.

Reference 1 does not include the details and calculations needed to determine the tabulated values for mass, shear area, and moment of inertia, that are necessary to calculate the translational stiffness and dynamic properties for the models (natural periods and mode shapes). The first three natural periods, two translational and one torsional, for each building model are reported in Figure A13, reference 1. Some torsional seismic response is expected for the Area 2 and Area 3 buildings due to structural eccentricity. However, reference 1 (the tables in Figure A12) does not include the geometry values and details necessary to calculate the torsional stiffness and natural periods for them.

This review included an effort to calculate the fundamental translational natural periods (modal analysis) for the Area 2 and Area 3 analysis models using the reference 1 values for mass, shear area, and moment of inertia for comparison to the periods reported. The review calculated period values are different from those reported in reference 1 as shown in the Table 5.

TABLE 5

REVIEW CALCULATED PERIOD VALUES		
225-B Building	**Fundamental Vibration Period Calculated by Technical Review (Seconds)**	**Fundamental Vibration Period Reported by Reference 1 (Seconds)**
Area 2	0.093	0.12
Area 3	0.043	0.06

The natural period calculation effort further confirms that reference 1 does not include the details necessary to repeat these seismic analyses. John A. Blume and Associates were contacted to inquire whether they have retained such details. Their files for this work, at this date, only have the analyses reports reviewed here. The results of this review show no reasons to conclude that the Area 2 and Area 3 seismic analysis models have unrealistic considerations for translational and torsional response, only that all the related modeling details are not provided in reference 1.

6.6.3 Pool Cell Cover Blocks

The Area 3 seismic analysis model in reference 1 includes the pool cell concrete cover blocks with the lumped-mass at the first floor level. The cover blocks for most of these cells are not in place, and have been removed from the 225-B Building. The reduced mass and diaphragm stiffness at the first level are two potential major related influences to the expected seismic response of Area 3, versus the calculated response using the analysis model with all the cover blocks. The reduced mass clearly reduces the expected seismic response and is beneficial to the seismic resistance of Area 3. The in-place cover blocks were not attached structurally to the building walls and the first floor. Therefore, the cover blocks provided no diaphragm stiffness contribution and their removal has no stiffness related influence to the seismic response. The permanent floor diaphragms at the pipe trench floor level between cell 11 and the north wall, and between cell 1 and the south wall compensate for the open cells (Drawings H-2-66421 and H-2-66422). These are important to the seismic resistance of Area 3, and the analysis model. The seismic analysis model for Area 3 remains conservative with the pool cell cover blocks removed.

6.6.4 Pool Cell Walls

A paramount safety function for the WESF is for the Area 3 pool cells to maintain water levels necessary for proper cooling and shielding of the stored capsules. The cells must continue to provide this function if a DBE occurs, and during and after.

The Area 3 seismic analysis model structural element between the base and first floor level nodes includes the pool cell walls. The shear area and moment of inertia for this element include contributions from the individual walls, but the walls are not individually idealized. The dynamic shear force and overturning moment calculated for this element were then distributed to the building walls, including the cell walls, that make up the element.

Reference 1 does not provide the process and details for determining the contributions from the cell walls to the model properties, and then distributing the model force and moment or seismic demands to them. At the time of the WESF design, this was a state of the art, acceptable method to calculate the seismic stresses for the cell walls. Many contemporary safety-related nuclear structures have been designed, constructed, and operated based on comparable seismic analyses. There is further assurance that the pool cell seismic analysis stresses are conservative, because no credit was taken for the additional restraint from the attached Area 2 foundation, and the removed cell cover blocks (see Sections 6.6.1 and 6.6.3).

Soil loads on the exterior basement walls of Area 3, and hydrodynamic seismic loads (sloshing) on the cell walls were accounted for in the VITRO design analysis (ref 6). The pool cell liners are not structural elements and would not be subject to significant seismic stresses. The liners are stainless steel sheet metal, 14 gage on the floors and 16 gage on the walls. The liners continuously are welded to stainless steel bars embedded in the concrete at 24 in. spacing. This review concludes that the pool cell walls were adequately idealized for the seismic analysis model.

6.7 PIPING AND EQUIPMENT

Reference 2 provides seismic analysis for four items of the WESF equipment and seismic design criteria for other equipment such as critical piping systems and supports for overhead cranes. The four items of equipment analyzed were the HVAC ducts between the 225-B Building and the K-1-1 and K3 Filter Buildings, the K3 filter units, the 225-B Building viewing windows, and the 225-B Building shielding airlock doors. Seismic design criteria for piping systems are provided by tables of pipe span-to-diameter limitations. The seismic design criteria for equipment located within the buildings include the internal floor response spectra provided for several locations in the WESF. The investigating engineers concur with these equipment analyses, and the design criteria provided, including the floor response spectra. They further conclude that the OBE and DBE response spectra for the first floor of Area 3 are especially conservative.

6.8 RESOLUTION OF STRUCTURAL DESIGN MODIFICATIONS

Several structural design modifications were recommended by reference 1. See Appendix A for a listing of the recommended design modifications and the results of a review of calculations and drawings to confirm the incorporation of the recommended design modifications. Of the 21 design modifications recommended, 16 were incorporated. It is concluded herein that 5 design modifications were not necessary. ~~The design changes were~~ incorporated in the original construction.

Also included in reference 1 was one recommendation for further evaluation of a stack on the roof of Area 1. No documented evidence of that evaluation was found. The investigating engineers conclude that related design modifications were not necessary.

6.9 RESOLUTION OF EQUIPMENT DESIGN MODIFICATIONS

Several equipment design modifications were recommended by reference 2. See A listing of the recommended design modifications and the results of a review of drawings to confirm the incorporation of the recommended design modifications are shown in Appendix B. Of the six design modifications recommended, three were incorporated, two are no longer applicable because of changes in design of replacement parts (see sections 5.7 and 6.5), and the recommendation to anchor the K3 filters was not incorporated. The design changes were incorporated in the original construction.

7.0 REFERENCES

A. DOCUMENTS

1. John A. Blume and Associates, "Earthquake Analysis of the Waste Encapsulation Facility," prepared by Westinghouse Hanford Company, Document No. RHO-R-22, (JABE-VITRO-01), April 1971.

2. John A. Blume and Associates, "Piping and Equipment Seismic Study, Building 225-B of the Waste Encapsulation Facility," prepared by Westinghouse Hanford Company, Document No. RHO-R-23 (JABE-VITRO-02), April 1971.

3. Winkel, B.V., G.R. Wagenblast, B Plant Canyon Structure Seismic Evaluation, prepared by Westinghouse Hanford Company, Document No. WHC-SD-WM-SA-005, January 16, 1990.

4. Langevin, A.S., B Plant Building Interface Study, prepared by Westinghouse Hanford Company, Document No. WHC-SD-WM-SA-014, 1991.

5. Scott, M.A., D.A. Moody, B Plant Canyon End-wall Study, prepared by Westinghouse Hanford Company, Document No. WHC-SD-WM-SA-013, 1991.

6. VITRO Engineering Corporation, "WESF Structural Design Calculations," prepared by Westinghouse Hanford Company, Document No. WHC-SD-WM-DA-219, Richland, Washington, 1970.

7. Abrams, M.S. and R.A. Meunow, "Nondestructive Tests of Reinforced Concrete in 225-B Building," prepared by Westinghouse Hanford Company, Document No. RHO-RE-CR-3, November 1982.

8. Stroup, C.R., "WESF Service Gallery Hot Spot and D/E Cell Floor Liner Repair," prepared by Westinghouse Hanford Company, Document No. RHO-CD-1407, March 1981.

9. Abrams, M.S. and R.A. Meunow, Portland Cement Association, "Concrete Nondestructive tests Conducted in 225-B Building," prepared by Westinghouse Hanford Company, Document No. WHC-SD-WM-ER-567, July 1982.

10. Abrams, M.S. and R.A. Meunow, Portland Cement Association, "Epoxy Injection Repairs To Concrete in 225-B Building," prepared by Westinghouse Hanford Company, Document No. WHC-SD-WM-ER-568, July 1982.

11. "Uniform Building Code," International Conference of Building Officials, Whittier, California.

12. National Fire Protection Association, "Installation of Sprinkler Systems," NFPA 13, Quincy, Massachusetts.

13. Hanford Plant Standards SDC 4.1, Rev. 12, "Standard Arch-Civil Design Criteria, Design Loads for Facilities," prepared by Westinghouse Hanford Company, 1993.

14. U.S. Department of Energy, "Preparation Guide for U.S. Department of Energy Nonreactor Nuclear Facility Safety Analysis Reports," Document No. DOE-STD-3009-94, Washington D.C., July 1994.

15. Hanford Plant Standards SDC 4.1, Rev. 5, "Standard Arch-Civil Design Criteria, Design Loads for Facilities," prepared by Westinghouse Hanford Company, 1968.

16. Hanford Plant Standards SDC 4.1, Rev. 6, "Standard Arch-Civil Design Criteria, Design Loads for Facilities," prepared by Westinghouse Hanford Company, 1973.

17. Earthquake Engineering Research Institute, Oakland, California, "Reducing Earthquake Hazards, Lessons Learned From Earthquakes," Document No. EERI 86-02, November 1986.

18. Earthquake Engineering Research Institute, "Learning From Earthquakes," Oakland, California, 1995.

19. Earthquake Engineering Research Institute, "Expected Seismic Performance of Buildings," Oakland, California, February 1994.

20. Senior Seismic Review and Advisory Group, "Use of Seismic Experience Data to Show Ruggedness of Equipment in Nuclear Power Plants," April 1990.

21. Electrical Power Research Institute, "A Methodology for Assessment of Nuclear Power Plant Seismic Margin," Document No. NP-6041, Rev. 1, Palo Alto, California, July 1991.

22. EQE Incorporated, "Seismic Experience Data Base," prepared for Seismic Qualification Utility Group (SQUG), San Francisco, California, 1996.

23. U.S. Department of Energy, "Generic Implementation Procedure for the Seismic Evaluation of Equipment in Existing U.S. Department of Energy Facilities" (DRAFT), Document No. UCRL-ID-122109DR, Washington, D.C., September 1995.

24. Nuclear Regulatory Commission, "Seismic Qualification of Equipment in Operating Plants," USI A-46, Washington, D.C., December 1980.

25. Federal Emergency Management Agency, "NEHRP Handbook for the Seismic Evaluation of Existing Buildings," FEMA-178, Washington, D.C., June 1992.

26. Kroon, R.J. et al, "Walk-Down Plan for Natural Phenomena Analysis," Fourth DOE Natural Phenomena Hazards Mitigation Conference, Atlanta, Georgia, October 1993.

27. Baldwin, F.M., and K.S. Herring, "Systematic Evaluation Program at Rocky Flats Plant," Fourth DOE Natural Phenomena Hazards Mitigation Conference, Atlanta, Georgia, October 1993.

28. Kimball, J.K. et al, "Developing a Generic Prioritization Guideline Document for Natural Phenomena Hazards Mitigation of Department of Energy Existing Facilities," Fourth DOE Natural Phenomena Hazards Mitigation Conference, Atlanta, Georgia, October 1993.

29. Manual, "Safety Analysis Manual," prepared by Westinghouse Hanford Company, Document No. WHC-CM-4-46, Rev. 2, 1996.

30. Westinghouse Hanford Company, "Specification for Waste Encapsulation and Storage Facility, 200-East Area," Document No. HWS-8951, 1971.

31. Westinghouse Hanford Company, "Probabilistic Seismic Hazard Analysis, DOE Hanford Site," Document No. WHC-SD-W236A-TI-002, Rev. 1, March 1996.

B. REFERENCE DRAWING LIST

H-2-66401, Architectural First Floor Plan, Area 1
H-2-66402, Architectural First Floor Plan, Area 2
H-2-66403, Architectural First Floor Plan, Area 3
H-2-66404, Architectural Second Floor Plan
H-2-66405, Architectural Sections
H-2-66406, Architectural Exterior Elevations
H-2-66408, Architectural Roof Plan & Details
H-2-66410, Architectural Interior Wall Sections & Details

H-2-66415, Structural Roof Plan, Areas 1 & 3

H-2-66416, Structural Floor Plan & Details, Area 1

H-2-66417, Structural First Floor Plan, Area 2

H-2-66418, Structural Second Floor Plan, Area 2

H-2-66419, Structural Roof Plan, Area 2, and Typical Details

H-2-66420, Structural Sections, Area 2

H-2-66421, Structural Foundation & Floor Plan, Area 3

H-2-66422, Structural Sections, Area 3

H-2-66423, Structural Process Cells Plans

H-2-66424, Structural Process Cells Sections

H-2-66425, Structural Cover Blocks

H-2-66426, Structural Sections

H-2-66427, Structural Liner Plate Details

H-2-66428, Structural Sections, Areas 2 & 3

H-2-66432, Structural Concrete, Filter Building K-1-1

H-2-66433, Structural Concrete, K3 System Filter Building

H-2-66865, Shielding Door Assembly and Instl, Cells A & G

H-2-66876, Shielding Door Details

H-2-66986, Engineering Flow Diagram, Storage Pool 3

H-2-67018, Pool Cell, Equipment and Piping Arrangement

H-2-67019, Pool Cell, Sections and Details

H-2-67020, General Services Piping, Area 3, Sections & Details

H-2-92722, Mechanical Pool Cell Contamination Control System

H-2-92723, Pool Cell Demineralized Water Addition System

H-2-92724, Pool Cell Waste Water Removal System

W-69333, Building 221-T-U-B, Sect 1 & 2 Concrete

W-69334, Building 221-T-U-B, Sect 1 & 2 Concrete

W-69565, Building 221-T-U-B, Standard Sections

W-69566, Building 221-T-U-B, Standard Sections

W-69978, Building 221-T-U-B, Sect 3 & Tail End Concrete

C. CERTIFIED VENDOR INFORMATION LIST

CVI 14056, Shielding Windows

CVI 14019, Pool Cell 10-ton Crane

CVI 14027, Process Cell 15-ton Crane

CVI 22637, Motorized Catwalk

Appendix A

Resolution of Structural Design Modifications

RESOLUTION OF STRUCTURAL DESIGN MODIFICATIONS

Several structural design modifications were recommended by reference 1. The recommended design modifications are listed on pages 11 through 13 of reference 1 and are reproduced herein along with the results of this current review of calculations and drawings to confirm the incorporation of the recommended design modifications.

Of the 21 design modifications recommended, 16 were incorporated and 5 were not incorporated. It is concluded herein that the 5 design modifications that were not incorporated were not necessary.

Listed below are the design modifications recommended by reference 1 and the results of this calculation and drawing review. The review comments are discussions of first, the configuration as analyzed; second, the configuration documented in the original design drawings; third, the configuration documented as "as-built"; and fourth, a conclusion as to whether the recommended design modification was incorporated. If not incorporated, a discussion of the adequacy of the "as-built" configuration is included.

For all design modifications recommended in reference 1 the "as-built" configuration reflects that the design changes were incorporated in the original construction.

A. **225-B Building, Area 1**

1. The seismic shear stress in the metal deck roof diaphragms at the top of the north-south masonry wall located 5 ft 4 in. east of column line 4 is 289 pcf. The allowable stress (with no 1/3 increase) for 1-1/2 in. by 22 ga. deck is 219 pcf.

   Suggested Modification

   Increase capacity of metal deck. [Although the over stressing occurs in only a few locations in the roof, we suggest using a 1-1/2 in. by 20 ga. deck of the same configuration shown on the design drawings, and thus increase the shear capacity and stiffness of the roof diaphragm. The comparable allowable stress for 1-1/2 in. by 20 ga. deck is 340 Pcf. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Configuration as Analyzed

- RHO-R-22, Appendix D, page 9d, 1-1/2-in. deep by 22-ga. metal deck, seam welds for metal deck, button punch at 24-in. o.c. with 1/2-in. diameter puddle welds.

Original Design Configuration

- Drawing H-2-66415, Rev. 0, note 3; Metal deck shall be 22 ga. by 1-1/2 in.
- Drawing H-2-66415, Rev. 0, note 4; Button punch panel side laps at 24 in.

As-Built Configuration

- Drawing H-2-66415, Rev. 3, note 3; Metal deck shall be 22 ga. by 1-1/2 in.
- Drawing H-2-66415, Rev. 3, note 4C; Button punch all side laps at 12 in.

Conclusion

A design modification to increase the diaphragm shear capacity of the metal deck was incorporated. The thickness of the metal deck was not increased from 22 ga. to 20 ga. as originally suggested by the draft calculation. The diaphragm shear capacity was increased, however, by changing the connection of the seams of the metal deck. By changing the spacing of the button punch welds from 24 in. to 12 in., the shear capacity was increased from 219 pcf to 270 pcf. While this is less than the calculated demand of 289 pcf, it is considered adequate.

2. A qualitative review of the structural connections shown on the design drawings has resulted in recommendations of modifications to certain connections to increase their capacity to transfer lateral loads.

Suggested Modification

2.A Connection of W10X33 to wall at column lines B-3 shown on detail 1/415 to allow for no north-south shear transfer between Areas 1 and 3.

Configuration as Analyzed

- Areas 1 and 3 were analyzed as separate structures

Original Design Configuration

- Drawing H-2-66415, Rev. 0, detail 1; connection has 7/8-in. diameter by 2-in. long slotted holes (north/south).

As-Built Configuration

- Drawing H-2-66415, Rev. 3, detail 1; connection has 7/8-in. diameter by 2-in. long slotted holes (north/south).

Conclusion

No design modifications were made. The original design detail provides slotted holes to preclude north/south shear transfer between Areas 1 and 3 as analyzed. No design modifications were necessary.

Suggested Modification

2.B Provision of additional lateral tie between continuous chord angle 3 by 3 by 5/16 and masonry walls on lines A and A2, between bearing supports of open web steel joists.

Configuration as Analyzed

- Not addressed in calculations.

Original Design Configuration

- Drawing H-2-66415, Rev. 0, detail 7; L3x3 connected to each open web steel joist with 1/8-in. fillet weld.

As-Built Configuration

- Drawing H-2-66415, Rev. 3, detail 7; L3x3 connected to each open web steel joist with 1/8-in. fillet weld and 1/8-in. penetration weld.

Conclusion

Additional lateral support between L3x3 and the open web joists has been provided. No additional ties were provided between the locations of the open web joists. Between the open web joists there is approximately a 3-in. gap between the L3x3 and the top of the wall. It is not clear what benefit would be gained by attaching the L3x3 to the top of the wall at intermediate locations between the joists. No design modifications were necessary.

2.C Clarification of welding of steel deck to structural steel and miscellaneous metal as follows:

Suggested Modification

2.C.1 Stagger welds of metal deck to top chord of open web steel joists so that loads are transferred symmetrically to the center of gravity of the top chord.

Configuration as Analyzed

- Not addressed in calculations.

Original Design Configuration

- Drawing H-2-66415, Rev. 0, note 4; welds at 6 in. on center.

As-Built Configuration

- Drawing H-2-66415, Rev. 3, note 4a; 5 welds per 24-in. wide panel, stagger welds on top chord of joist.

Conclusion

Welding has been clarified as staggered.

Suggested Modification

2.C.2 Clarify welding to perimeter chord angles and channel to insure a maximum weld spacing of 6 in.

Configuration as Analyzed

- Not addressed in calculations.

Original Design Configuration

- Drawing H-2-66415, Rev. 0, note 4; attachment of metal deck to structural steel by welds at 6 in. on center.

As-Built Configuration

- Drawing H-2-66415, Rev. 3, note 4b; perimeter welds, north and south sides, space at 6 in. on centers.

Conclusion

Perimeter welding has been clarified.

Suggested Modification

2.C.3 Distinguish between welding typical end support and typical intermediate supports of metal deck.

Configuration as Analyzed

- Not addressed in calculations.

Original Design Configuration

- Drawing H-2-66415, Rev. 0, note 4; attachment of metal deck to structural steel by welds at 6 in. on center.

As-Built Configuration

- Drawing H-2-66415, Rev. 3, note 4a; At end and intermediate supports equally space 5 welds per 24-in. wide panel.

Conclusion

Perimeter welding has been clarified. No distinction is made between end and intermediate support welding.

Suggested Modification

2.D Review of stack, equipment, and platform support located on column line B between lines 7 and 7.5 for dynamic response.

Configuration as Analyzed

- Not addressed in calculations.

Original Design Configuration

- Drawing H-2-66415, Rev. 0, plan and details 14 and 15; stack platform supported by 3 joists, grating is 1-1/2 in. by 1/8 in.

As-Built Configuration

- Drawing H-2-66415, Rev. 3, plan and details 14 and 15; stack platform supported by 6 joists, grating is 1-1/2 in. by 3/16 in.

Conclusion

No documented evidence that the dynamic characteristics of the stack have been accounted for. The roof support and the grating have been increased in strength and stiffness from the original design but there is no indication in the calculations for the reason for the design change.

B. **225-B Building, Area 2 (and 3)**

1. The large overturning moment on the second floor beam B-9 induces tensile stresses in the seven #11 reinforcing bars above the allowable.

Suggested Modification

Increase capacity of B-9. [Add additional three #11 bars, bringing the total tensile reinforcing to ten #11 bars. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Configuration as Analyzed

- RHO-R-22, Appendix D, page 23d to 25d, Beam B-9 capacity based on seven #11 bars.

Original Design Configuration

- Drawing H-2-66419, Rev. 0, Beam B-9 has seven #11 bars at top of beam.

As-Built Configuration

- Drawing H-2-66419, Rev. 2, Beam B-9 has nine #11 bars at top of beam.

Conclusion

Beam B-9 capacity has been increased. The beam capacity based on nine #11 bars satisfies the DBE requirements. The ten #11 bars recommended in the draft calculation were to satisfy demands from an OBE of 0.12g which is not now required per SDC 4.1, Rev. 12.

2. The large overturning moment on beam B-9 induces axial forces on the column and footing at column lines C-3 above the allowable.

Suggested Modifications

Increase capacity of column and footing. [Enlarge column C-3 to 24 by 24 with eight #9 vertical reinforcing bars. Column is to run from top of footing to roof. Footing to be enlarged to 11 ft 9 in. by 11 ft 9 in. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Column C-3

Configuration as analyzed

- RHO-R-22, Appendix D, page 26d, Column C-3 capacity based on T shaped column having eight #6 bars vertical.

Original Design Configuration

- Drawing H-2-66417, Rev. 0, Section 1: Column C-3 designed as T shaped column having eight #6 bars vertical.

As-Built Configuration

- Drawing H-2-66417, Rev. 3, Section 1: Column C-3 changed to 24- by 24-in. column having eight #9 bars vertical.

Conclusion

Column C-3 capacity has been increased as recommended.

Footing C-3

Configuration as Analyzed

- Not addressed in calculations.

Original Design Configuration

- Drawing H-2-66421, Rev. 0, Plan Below 699 ft 6 in.: Footing C-3 designed as 7 ft 2 in. by 7 ft 3 in. with six #5 and six #6 bars e/w and four #6 bars n/s in the spread footing.

As-Built Configuration

- Drawing H-2-66421, Rev. 3, Plan Below 699 ft 6 in.: Footing C-3 changed to 11 by 11 ft with sixteen #7 bars e/w and eight #7 bars n/s in the spread footing.

Conclusion

Footing C-3 capacity has been increased.

3. The percentage of reinforcing of the column at D-3 is below the ACI-63 recommended minimum for a tied column.

Suggested Modification

Increase capacity of column. [Increase vertical reinforcing from eight #8 to eight #9 bars. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Configuration as Analyzed

- RHO-R-22, Appendix D, page 28d, Column D-3 is 23 by 35 col with eight #8 bars. Steel ratio (8 by 0.79)/(23 by 35) = 0.0078 < 0.01.

Original Design Configuration

- Drawing H-2-66421, Rev. 0, Plan Below 699 ft 6 in.: Column D-3 designed as 23 by 35 in. with eight #8 bars vertical.
- Drawing H-2-66420, Rev. 0, Det 15: Column D-3 designed as 23 by 35 in. with eight #8 bars vertical.

As-Built Configuration

- Drawing H-2-66421, Rev. 3, Plan Below 699 ft 6 in.: Column D-3 designed as 23 by 35-in. with eight #8 bars vertical.

- Drawing H-2-66420, Rev. 2, Det 15: Column D-3 designed as 23 by 35 in. with eight #8 bars vertical.

Conclusion

The design of column D-3 was not changed. In accordance with ACI-349, 10.8.4. For a compression member with a larger cross section than required by considerations of loading, a reduced effective area Ag not less than one-half the total area may be used to determine minimum reinforcement and design strength. From RHO-R-22, page 28d, the required column capacity is 401k. Total column area is 805 in^2. The minimum effective area Ag is 402.5 in^2. The capacity of a column having a concrete area of 402.5 in^2 and a steel area of 8x.79=6.32 in^2 is .8x.7x[.85x3ksix(402.4-6.32)+60ksix6.32)]=778k. Steel ratio is 6.32/402.5=0157. The capacity of the reduced size column satisfies the demand loading. The steel ratio satisfies the ACI-349 code. Therefore, the column is safely designed. No design changes were necessary.

4. Column and footing at D.4-2 are stressed over the allowable.

Suggested Modifications

Increase capacity of column and footing. [Place 18- by 18-in. pilaster in wall at D.4-2 and reinforce with eight #7 bars and enlarge footing at D.4-2 to 10 ft by 10 ft. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Conclusion

The calculations in RHO-R-22, Appendix D, page 29d, incorrectly calculate the load on column D.4-2 from the Area 2 Process Cells. The process cell floor between column lines 2 and 3 is supported by a foundation wall along column line D.4. Most of the loads attributed to a column at D.4-2 by calculation RHO-R-22, page 29d, should actually be applied to the column at D3. Column D3 (see comment 3 above) was found to be designed safely as shown on calculation RHO-R-22, page 28d. No design changes were necessary.

5. Column and footing at Col. E-2 are stressed over the allowable.

Suggested Modifications

Increase capacity of column and footing. [Enlarge column at E-2 to 20 by 20-in. with eight #8 bars vertical and enlarge footing to 10 by 10 ft. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Column

Configuration as Analyzed

- RHO-R-22, Appendix D, page 30d, Column E-2 is 18 by 18 col with six #6 bars.

Original Design Configuration

- Drawing H-2-66420, Rev. 0, Detail 15: Column E-2 designed as 18 by 18 in. with six #6 bars vertical.

As-Built Configuration

- Drawing H-2-66420, Rev. 2, Detail 15: Column E-2 designed as 18 by 18 in. with eight #8 bars vertical.

Conclusion

Column E-2 capacity has been increased by changing the vertical steel to be eight #8 bars as recommended. The column size was not increased. The column capacity using the methods of RHO-R-22, page 30d, is P=0.85x18x18x(.25x3ksi+ 6.32x24ksi/(18x18))=335k which exceeds the calculated demand of 322k. Therefore, the column is safely designed and constructed.

Footing

Configuration as Analyzed

- RHO-R-22, Appendix D, page 30d, Footing E-2 is 6-1/2 by 6-1/2 ft.

Original Design Configuration

- Drawing H-2-66421, Rev. 0, Plan Below 699 ft 6 in.: Footing E-2 designed as 6-1/2 by 6-1/2 ft.

As-Built Configuration

- Drawing H-2-66421, Rev. 3, Plan Below 699 ft 6 in.: Footing E-2 designed as 8-1/2 by 8-1/2 ft.

Conclusion

Footing E-2 capacity has been increased by changing to be 8-1/2 by 8-1/2 ft. The footing loading is P = 322 k/(8.5 by 8.5) = 4.5 ksf which is less than the soil bearing capacity of 5ksf. Therefore, the footing is safely designed and constructed.

6. Walls on lines C and E which act as cantilevered beams, cantilevering out 4 ft 10 in. east of line 7, have the tensile steel at top of wall stressed over the allowable.

Suggested Modifications

Increase capacity of wall. [Place four #9 by 20-ft tensile reinforcement at top of walls on lines C and E. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Wall on Line C

Configuration as Analyzed

- RHO-R-22, Appendix D, page 40d: Two #6 bars at top of wall.

Original Design Configuration

- Drawing H-2-66420, Rev. 0, Detail 10: Two #8 bars at top of wall.

As-Built Configuration

- Drawing H-2-66420, Rev. 2, Detail 10: Four #9 bars at top of wall.

Conclusion

Wall C capacity has been increased as recommended.

Wall on Line E

Configuration as Analyzed

- RHO-R-22, Appendix D, page 40d, two #6 bars at top of wall.

Original Design Configuration

- Drawing H-2-66426, Rev. 0, Detail 5: Three #11 bars at top of wall.

As-Built Configuration

- Drawing H-2-66426, Rev. 2, Detail 5: Three #11 bars at top of wall.

Conclusion

Wall E was not changed. Area of steel provided is 3 by 1-1/2 = 4-1/2 which exceeds the calculated requirement of four #9 = 4 by 1.0 = 4.0. Therefore, Wall E is safely designed. No design changes were necessary.

7. Columns C7 and E7 are stressed over the allowable because of overturning moments on the cantilevered walls described above in 6.

Suggested Modification

Increase capacity of column. [Enlarge columns at C7 and E7 to 20 by 20 in. with eight #8 bars vertical, and enlarge footing to 10 by 10 ft. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Column C7

Configuration as Analyzed

- RHO-R-22, Appendix D, page 41d suggested increasing capacity of column C7 to accommodate a demand of 334k.

Original Design Configuration

- Drawing H-2-66417, Rev. 0: 12 by 22 col with six #5 bars vertical.

As-Built Configuration

- Drawing H-2-66417, Rev. 2, and Drawing H-2-66420, Rev. 3, Detail 15: 12 by 22 col with eight #9 bars vertical.

Conclusion

Column C-6.8 capacity has been increased by changing the vertical steel to be eight #9 bars. The column size was not increased. The column capacity using the methods of RHO-R-22, page 30d, is P=0.85x12x22x(.25x3ksi+8x1x24ksi/(12x22))=331.5k which is within 1% of the demand of 334k. Therefore, the column is designed and constructed safely.

Column E7

Configuration as Analyzed

- RHO-R-22, Appendix D, page 41d suggested increasing capacity of column E7 to accommodate a demand of 334k.

Original Design Configuration

- Drawing H-2-66417, Rev. 0; Drawing H-2-66420, Rev. 0, Section 15; Drawing H-2-66426, Rev. 0, Section 5/418: No special column reinforcement provided. Wall reinforcement only.

As-Built Configuration

- Drawing H-2-66417, Rev. 3; Drawing H-2-66420, Rev. 2, Section 15; 12 by 26 col with eight #10 bars vertical.

Conclusion

Column E-6.8 capacity has been increased by changing the vertical steel to be eight #10 bars. The tied column size is 12 by 26. The column capacity using the methods of RHO-R-22, page 30d, is P=0.85x12x26x(.25x3ksi+8x1.23x24ksi/(12x26))=400k which exceeds the demand of 334k. Therefore, the column is designed and constructed safely.

Footing C7

Configuration as Analyzed

- RHO-R-22, Appendix D, page 41d suggested increasing capacity of footing C7 to accommodate a demand of 334k.

Original Design Configuration

- Drawing H-2-66417, Rev. 0; Footing C6.8 is 6 by 6 ft with seven #5 bars.

As-Built Configuration

- Drawing H-2-66417, Rev. 3; Footing C6.8 is 9 ft 8 in. by 7 ft 6 in. with fourteen #6 bars.

Conclusion

Footing C6.8 capacity has been increased by changing to be 9 ft 8 in. by 7 ft 6 in. The footing loading is P=334k/(9.67x7.5)=4.6ksf which is less than the soil bearing capacity of 5ksf. Therefore, the footing is safely designed and constructed.

Footing E7

Configuration as Analyzed

- RHO-R-22, Appendix D, page 41d suggested increasing capacity of footing E7 to accommodate a demand of 334k.

Original Design Configuration

- Drawing H-2-66417, Rev. 0; Footing E6.8 is 7-1/2 by 7-1/2 ft with ten #7 bars.

As-Built Configuration

- Drawing H-2-66417, Rev. 3; Footing E6.8 is 9 by 9 ft with fourteen #7 bars.

Conclusion

Footing E6.8 capacity has been increased by changing to be 9 by 9 ft. The footing loading is P=334k/(9x9)=4.1ksf which is less than the soil bearing capacity of 5ksf. Therefore, the footing is safely designed and constructed.

8. Bending stresses are induced into the wall on line C at elevation 734 ft 3 in. (top of elevator shaft walls) because of the north-south seismic shear forces transferred by weak direction bending in the wall from the top of the main roof to the top of the elevator and stairwell walls. The bending stresses cause the tensile reinforcing stress to be above the allowable.

Suggested Modification

Increase Capacity of Wall

[Add six #9 by 10-ft bars (three at each face of wall) at the intersection of the four 10-in. north-south walls of the elevator and stair well and the wall on line C. (Suggested modification obtained from draft Appendix D, dated March 2, 1971.)]

Configuration as Analyzed

- RHO-R-22, Appendix D, page 38d; #4 at 12-in. wall reinforcing in wall on line C.

Original Design Configuration

- Drawing H-2-66420, Rev. 0, Section 1; #4 at 12-in. wall reinforcing in wall on line C.

As-Built Configuration

- Drawing H-2-66420, Rev. 2, Section 1; six #9 bars (three each face) in wall on line C at the 4 intersecting walls.

Conclusion

The roof over the elevator shaft area was completely redesigned. The original design for a sheet metal roof was changed to be a reinforced concrete roof and the wall C reinforcement was changed to be six #9 bars (three each face) at the 4 intersecting walls. Wall C capacity has been increased as recommended.

9. The reinforcing in the mat footing of the process cell area is stressed over the allowable.

Suggested Modification

Increase capacity of footing. [Increase bottom layer of reinforcing from #6 at 6 in. to #7 at 6 in. (Suggested modification obtained from draft Appendix D, dated March 2, 1971)]

Configuration as Analyzed

- RHO-R-22, Appendix D, page 35d; 24-in. deep footing with #6 at 6 in.

Original Design Configuration

- Drawing H-2-66420, Rev. 0, Section 4/417: 24-in. deep footing with #6 at 6 in.

As-Built Configuration

- Drawing H-2-66420, Rev. 2, Section 4/417: 24-in. deep footing with #7 at 6 in.

Conclusion

Process cell area mat footing capacity has been increased as recommended.

APPENDIX B

Resolution of Equipment Design Modifications

RESOLUTION OF EQUIPMENT DESIGN MODIFICATIONS

Several equipment design modifications recommended by reference 2 are reproduced herein, with the results of this current review of drawings, to confirm the incorporation of the recommended design modifications. The review comments are discussions of the configuration documented in the original design drawings; the configuration documented as "as-built"; and a conclusion as to whether the recommended design modification was incorporated. If not incorporated, a discussion of the adequacy of the "as-built" configuration is provided.

The design review was conducted on in-progress drawings and in-progress design details for the viewing windows and the shielding doors. In most window and shielding door design details, the original drawing configuration reflects that the design changes recommended by reference 2 were incorporated into the original construction.

1. Reference 2, HVAC Underground Ducts, page 4: It is recommended that consideration be given to provision of duct sleeves at the building wall penetrations.

   Original Design Configuration
   - Drawing H-2-66417, Rev. 0: No duct sleeves.

   As-Built Configuration
   - Drawing H-2-66417, Rev. 3: 30-in. cmp sleeve installed around 24-in. duct.

   Conclusion
   Duct sleeve installed as recommended.

2. Reference 2, HVAC Filter Units, page 5: Independent anchorage of the filter units to the structure is suggested.

Original Design Configuration

- Drawing H-2-66433, Rev. 0: Original design included four 1-1/4-in. studs per filter frame.

As-Built Configuration

- Drawing H-2-66433, Rev. 3: As-built design removed all studs flush with support pad.

Conclusion

Filter frames not anchored as recommended.

3. Reference 2, HVAC Filter Units, page 6: it is suggested that the seal weld connecting the filter frame supporting angles be made at least 3/16-in. continuous fillet.

Conclusion

This comment is no longer applicable. The filters were replaced in 1987 by filters constructed to drawing H-2-99447.

4. Reference 2, Viewing Windows, page 7: The keepers should be capable of resisting elastically not less than about 30% of the window assembly operating weight.

Original Design Configuration

- Drawing H-2-66726, Rev. 0: Original design consisted of window frames on rollers to facilitate change out.

As-Built Configuration

- CVI-14056: As-built design consists of stepped window panels held into place by packed lead wool.

Conclusion

This comment is no longer applicable. The viewing windows have been redesigned.

5. Reference 2, Shielding Doors at Air Locks, page 8: It is suggested that consideration be given to providing a vertical-load supporting base plate welded to the bottom bracket.

Conclusion

Drawing H-2-66865, detail 22, shows a 12- by 16- by 1-in. thick base plate welded to the bottom of the lower hinge bracket.

6. Reference 2, Shielding Doors at Air Locks, page 8: at the upper brackets, add a number of anchor studs or bent flat bar lugs capable of transmitting both shear and tension.

Conclusion

Drawing H-2-66865, detail 21, shows six 1-in. anchor bolts capable of both tension and shear located in a symmetrical bolt pattern in the upper hinge bracket.

APPENDIX C

Qualifications of Investigating Engineers

KUEN-CHUN TU, Ph.D.,P.E.

EDUCATION

1974	Ph.D., Civil Engineering, (Structural Dynamics), University of Colorado, Boulder, Colorado.
1968	M.S., Civil Engineering, (Structural Engineering), Utah State University, Logan, Utah.
1965	B.S., Hydraulic Engineering, National Cheng-Kung University, Tainan, Taiwan.

PROFESSIONAL REGISTRATIONS

Registered Professional Engineer, Colorado, No. 12161, 1973
Registered PE-Civil Engineering, Washington, No. 17355, 1978

EXPERIENCE

Dr. Tu has more than 27 years of diverse experience in fields of structural design and engineering mechanics with particular emphasis on nonlinear and dynamic analysis. Experience related to structural design has included: steel framing, bracing, coal silos, concrete slabs, footings, heavy machinery foundations, and bridges. Dr. Tu has provided stress, dynamic, and impact analysis on pressure vessels, shipping canisters, underground nuclear waste storage tanks, and piping systems. Dr. Tu has been responsible for seismic, tornado, and dynamic related structural designs and analyses. Dr. Tu was the Principle Lead Engineer on the B-Plant Canyon Crane Replacement Project and Lead Civil/Structural Engineer on a definitive design project - B Plant Process Condensate Treatment Facility.

Responsibilities and duties as a consultant engineer in structural analysis have focused on providing highly complex analyses on a variety of nuclear-related projects. Dr. Tu has performed lead engineer duties on large and complex projects.

Analytical work has required extensive utilization of finite element programs, including ANSYS, ANACAP-U, SAP-IV, FLUSH, DRAIN-2D, STARDYN, STRUDL, and IMAGES-3D. Dr. Tu has developed several structural, and mechanical programs. Dr. Tu is thoroughly familiar with personal and main frame computers. He is experienced with response spectrum and time history methods used for seismic analysis of linear and non-linear structures which include soil-structural interaction. He is experienced with use of modal analysis to check experimental data. His projects have involved non-linear and creep properties of irradiated material.

Listed are several projects that Dr. Tu has performed static and dynamic analyses and designs:

- Multi-Function Waste Tank Facility. Evaluated long-term thermal and creep analysis of the 1 Mgal underground radioactive waste tank. Evaluated the safety factor of the tank for extreme vertical or horizontal loading.

KUEN-CHUN TU, Ph.D., P.E. (continued)

- 241-AQ Tank Farm Storage Facilities. Seismic analysis for a series of double shell underground 1 Mgal waste storage tanks. Soil-structure interaction and tank to tank interaction due to Design Basis Earthquake was analyzed.

- Multi-Purpose Transfer Box Study. Lead Engineer to search for an optimum design of a new railroad transfer box to meet DOE regulatory criteria.

- Spent Fuel Shipping Canister Design. Developed a half spherical shell header to absorb impact energy of a 40-ft drop.

- Hanford N-Reactor Hydrogen Mitigation Project. Designed a 30-in. diameter duct and its supports to withstand tornado generated missile impact.

- Plutonium Storage Building. Modified the existing building to withstand missile and bomb impact loading.

- B Plant Process Condensate Treatment Facility. Lead Structural Engineer in designing a three story high concrete building adjacent to the Hanford B-Plant.

- Hanford N Reactor Building. Involved in several projects to upgrade the aging reactor building to meet nuclear industry standard.

- Fuel Supply FMEF Facility. Designed facility for the Design Basis Earthquake (DBE) and the Design Basis Tornado (DBT).

- Seismic analysis and design of Reactor Service Building (RSB) and the RSB Extension at the Fast Flux Test Facility (FFTF) for Westinghouse Hanford Company.

- Pawnee Power Plant in Colorado. Concrete design of the 500 MW power plant.

- North Valmy Power Station in Nevada for Sierra Pacific Power Co. Designed a 100 ft span highway and railroad bridge, and a turbine-generator foundation.

- Rock Island Hydroelectric Power Plant in Washington. Responsible for analysis and design of the plant for French-built horizontal bulb turbines of the 400 MW hydroelectric project.

SELECTED PUBLICATIONS/PRESENTATIONS

"A Theoretical Analysis of the Dynamic Response of Construction Cableway Systems," Co-author with Dr. R.S. Ayre, The Shock and Vibration Bulletin, Part 4, pp. 21, September, 1977.

PROFESSIONAL AFFILIATIONS

Member, American Society of Civil Engineers, 1970

KUEN-CHUN TU, Ph.D., P.E. (continued)

SECURITY CLEARANCE

Department of Energy (DOE), "O" (Work) Clearance

EMPLOYMENT HISTORY

ICF Kaiser Hanford Company	Consultant Engineer	1977 to Present
Stone and Webster Engineering Corp.	Structural Engineer	1975 to 1977
Stearns-Roger, Inc.	Design Supervisor	1968 to 1975

FRANK RAY VOLLERT, M.S., P.E.

EDUCATION:

University Degrees

1963	BS Civil Engineering, Gonzaga University, Spokane, Washington
1965	MS Structural Engineering, University of Minnesota, Minneapolis, Minnesota

Continuing Education and Training Highlights

Numerical Analysis, University of Washington
Structural Dynamics, University of Washington
Energy Methods, University of Washington
Earthquake Engineering, UCLA
Reinforced Concrete, American Concrete Institute
Statistics, Graduate Center
Fracture Mechanics, University of Washington
Engineering Training Program, Bettis Laboratory
Business Communications, Carnegie-Mellon University
Data Base Design, Bettis Laboratory

PROFESSIONAL REGISTRATION

Registered Professional Civil Engineer, State of Washington, Since 1969

CURRENT AND PAST PROJECTS

Conducting structural and civil engineering evaluations for all phases radioactive and hazardous waste management operations. Extensive applied experience includes structural and earthquake engineering: analyses, evaluations, and research for radioactive waste and isotope handling facilities, plus nuclear reactors. Member of Hanford Site Emergency Preparedness Team.

SPECIALTIES

Technical competence includes structural dynamics, earthquake engineering, finite element analyses of structures (including reinforced concrete) and equipment, fracture mechanics and stress corrosion cracking evaluations, structural design acceptance criteria development, and structural, seismic performance, and materials research and testing.

FRANK RAY VOLLERT, M.S., P.E. (continued)

PROFESSIONAL BACKGROUND

Structural Engineer, Thirty Years Experience, (25 years total on the Hanford Site, Employers other than Westinghouse Hanford (currently) starting with most recent have been, Westinghouse Bettis Atomic Power Laboratory, Rockwell International, Atlantic Richfield. Battelle Memorial Institute, and Boeing Aerospace.

PROFESSIONAL ASSOCIATIONS

Registered Professional Civil Engineer, State of Washington, Since 1969

American Concrete Institute (ACI), Member and Former Chair for Committee 227, Radioactive Waste Management, Member Committee 216, Fire Resistance

Earthquake Engineering Research Institute (EERI), Member Building Officials Education Committee

American Radio Relay League (ARRL)

PUBLICATIONS

"Structural Evaluations, Reinforced Concrete Waste Storage Tanks," ACI, 1978
"Corrosion, Waste Storage Tanks," NACE, 1993

ACHIEVEMENTS AND ACTIVITIES

Gonzaga University, Outstanding Senior, Civil Engineering
City of Kennewick, Board of Adjustment, and Planning Commissioner
State of Washington, Emergency Management Volunteer
Federal Communications Commission, Amateur Radio License, Advanced Class

GARY RAY WAGENBLAST, B.S., P.E. (continued)

GARY RAY WAGENBLAST, B.S., P.E.

EDUCATION

1973 B.S. Civil Engineering,
Washington State University, Pullman, Washington

PROFESSIONAL REGISTRATION

Registered Professional Civil Engineer, Washington, 1984.

EXPERIENCE

Mr. Wagenblast has 22 years experience in civil and structural engineering. He is currently a Senior Principal Engineer in the ICF Kaiser Hanford Engineering and Technical Support Services, Civil Structural and Environmental Engineering Group. He is well versed in the design criteria for nuclear and non-nuclear steel, wood, and reinforced concrete structures. He has been responsible for the structural qualification of many buildings, structures, systems, and components to the latest U.S. Department of Energy design criteria. Mr. Wagenblast was responsible for the structural linear and non-linear seismic and extreme wind evaluation of the existing 325 laboratory building and the existing 234-5Z plutonium processing building qualifying the facilities to current DOE criteria. He was also responsible for the soil-structure interaction seismic design evaluation for proposed buried radioactive-liquid-waste tanks.

Listed are several projects that Mr. Wagenblast has performed static and dynamic analyses and designs:

- Lead engineer for the seismic soil-structure-interaction design of an 83-ft diameter, 54-ft high, buried reinforced concrete tank for radioactive-liquid waste. The design evaluation included the development of simplified computer modeling and evaluation methods which significantly reduced the manpower and computer time to complete the evaluation.

- Performed linear and approximate non-linear evaluation of the reinforced concrete B-Plant canyon, B-Plant stack, AR vault, and AR stack. Evaluations included seismic capacity, extreme wind capacity, dead and live load capacity, and the development of internal floor response spectra for subsequent evaluation of equipment.

- Performed linear and approximate non-linear seismic and extreme wind evaluation of the structural steel frame 325 laboratory building qualifying this facility to current DOE criteria.

- Directed the activities of consultants in the linear and non-linear dynamic evaluation efforts for the qualification of the structural steel braced frame 234-5Z Building to current DOE criteria. The evaluation accounted for stiffness and structural capacity of reinforced concrete walls not originally intended to resist lateral loads.

GARY RAY WAGENBLAST, B.S., P.E. (continued)

- Evaluated and designed upgrades for the structural stability of glove boxes, hoods, and filters in the 234-5Z laboratory to current DOE criteria in support of PFP restart.

SELECTED PUBLICATIONS

C.K. Wong, M.D. Stine, G.R. Wagenblast, S.K. Farnworth, "Soil Structure Interaction Analysis of Buried Tank Subjected to Vertical Excitation." Fifth DOE Natural Phenomena Hazards Mitigation Symposium - 1995, Denver, Colorado.

G.R. Wagenblast and M.D. Northey, "Use of Personal Computers in Performing a Linear Modal Analysis of a Large Finite-Element Model." Seismic Engineering, American Society of Mechanical Engineers, ASME publication PVP-Volume 220, pp. 97-100, June 1991.

B.V. Winkel and G.R. Wagenblast, "Nonlinear Seismic Analysis of a Thick-Walled Concrete Canyon Structure." Second DOE Natural Phenomena Hazards Mitigation Conference - 1989, Knoxville, Tennessee. pp. 135-141.

EMPLOYMENT HISTORY

ICF Kaiser Hanford Company, Senior Principal Engineer	1-95 to present
Westinghouse Hanford Company	4-80 to 1-95
Rockwell Hanford Company	2-77 to 4-80
J.C. Lemons Co.	9-76 to 1-77
Northern Industrial Contractors	7-76 to 9-76
Wright-Schuchart-Harbor	4-74 to 6-76
Boeing Aircraft Co.	6-73 to 3-74

APPENDIX D

Qualification of Cell Drain Line

ICF KAISER HANFORD

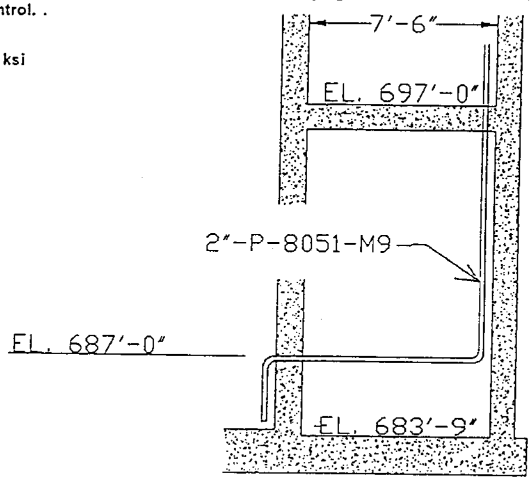
Calc. No. ________
Revision 0
Page No. D1 of ____

DESIGN ANALYSIS

Client	Westinghouse Hanford Company	WO/Job No.	E51162		
Subject	WESF Natural Phenomena Hazards Survey	Date	February 1996	By	K.-C.Tu
	Seismic Analysis of Waste Water Removal System	Checked	February 1996	By	Gary Wagenblast
Location	225-B Building, 200 East Area	Revised		By	

2"- P- 8051- M9 Pipe, Schedule 10S, Stainless steel, seamless or welded per ASTM A312, Type 304L See "HWS- 8951, Division XVII, Plumbing, Process and Service Piping" in Folder XIX of Box 31685 of Central Document Control. .

Minimun yield stress = 25 ksi



WASTE WATER REMOVAL PIPE (SECTION, LOOKING SOUTH)
Refer to dwg. H-2-67019,"POOL CELL SECTIONS AND DETAILS"

Pipe Data (From: The 'Piping Guide' by Syentek, Table P-1, 2" Standard Size schedule 10S pipe)

Outside Diameter $Od := 2.375 \cdot in$

Weights of Plain End Pipe and Water in Pipe $W := (2.64 + 1.58) \cdot \frac{lb}{ft}$ $W = 4.22 \cdot \frac{lb}{ft}$

Section Modulus $S := 0.421 \cdot in^3$

Assume the pipe is a 9' cantilever beam. $l := 9 \cdot ft$

Use equivalent static method: the seismic load = 1.5 X 0.2 ZPA spectrum peak g value (SDC 4.1)

Use 5% of critical damping (UCRL-15910, June 1990, Table 4-6)

$$g := 32.2 \cdot \frac{ft}{sec^2}$$

Acceleration $A := 1.5 \cdot 0.42 \cdot g$ $A = 20.286 \cdot \frac{ft}{sec^2}$

Moment $M := \frac{1}{2} \cdot \frac{A}{g} \cdot W \cdot l^2$ $M = 1292.08 \cdot lb \cdot in$

For simplicity combined three direction seismic loads with dead load

$$s := \frac{\sqrt{M^2 + M^2 + \left(\frac{2}{3} \cdot M\right)^2} + \frac{1}{2} \cdot W \cdot l^2}{S} \qquad s = 9.67 \cdot ksi \qquad < 25 \text{ ksi O.K.}$$

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DESIGN ANALYSIS

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Client	WHC	WO/Job No.	E51162
Subject	WESF NPHS	Date	By SRWagenblast 5-7-96
		Checked	By F.P. Vollert 5/23/96
Location	225-B, 200 E AREA	Revised	By

EVALUATE 3"ϕ WATER RECIRCULATION LINE IN TUNNEL:
3"-P-8021-M9 ASTM A312 TYPE 304L

PIPE DATA (FROM 'PIPING GUIDE' by SYENTEK, TABLE 1-2) SCH 10S

O.D = 3.5 in

WT OF PIPE = (4.33 + 3.62) lb/FT = 7.95 lb/FT WATER FILLED

SECTION MODULUS = 1.041 in^3

Assume Pipe is 9 FT CANTILEVER $l = 9$ FT

USE EQUIVALENT STATIC METHOD
SEISMIC LOAD = 1.5 x PEAK OF 5% RESP. SPECTRUM.
= 1.5 x 0.42 g = 0.63 g OR 20.29 Ft/s^2

SEISMIC MOMENT $= \frac{1}{2}\frac{A}{g} w l^2 = \frac{1}{2}(.63g)(7.95 \frac{lb}{F})(9F)^2 = 202.8$ FT lb
$= 2434$ in lb.

DEAD WT MOMENT $= \frac{1}{2} w l^2 = \frac{1}{2}(7.95 \frac{lb}{ft})(9 ft)^2 = 3864$ in lb

COMBINE 3 SEISMIC DIRECTIONS PLUS DW

$$f = \frac{\sqrt{M^2 + M^2 + (\frac{2}{3}M)^2} + \frac{1}{2} w l^2}{S} = \frac{3805 \text{ in lb} + 3864}{1.041 \text{ in}^3} = 7,367 \text{ psi}$$

< 25,000 psi

OK

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APPENDIX E

Seismic Analysis Crane Support Systems

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DESIGN ANALYSIS

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Client WHC		WO/Job No.	E51162
Subject WESF CRANE		Date 5-13-96	By GRWagenblast
		Checked 5/16/96	By F.R. Volcert
Location 225-B		Revised	By

EVALUATE AREA 3 POOL CELL 10 TON BRIDGE CRANE SUPPORT SYSTEM FOR SEISMIC LOADS.
REF DWGS H-2-66421, H-2-66422

SEISMIC LOADS: PER REF 2. DESIGN CRITERIA

$F_\perp = C_\perp \cdot W_c = 1.6\, W_c$ NORMAL TO RAIL

$F_{\parallel} = C_{\parallel} \cdot W_c = 1.2\, W_c$ PARALLEL TO RAIL

$F_v = C_v(W_c + W_L) = 0.8(W_c + W_L)$ VERTICAL

W_L = wt of lifted load = 20,000 lb

W_c = wt of bridge + trolley + hoist

bridge wt = 14054 lbs CVI 14019
bridge drive = 1171 " "
sheaves = 228 " "
trolley wt = 1275 " "
hoist wt = 1406 " CVI 14027 (10 ton hoist in Area 2)

W_c = 18134 lb USE 20,000 lb.

$F_\perp = 1.6(20000) = 32000$ lb OR 8000 lb/wheel

$F_{\parallel} = 1.2(20000) = 24000$ lb OR 6000 lb/wheel

$F_v = 0.8(20000 + 20000) = 36000$ lb.

NEED TO DETERMINE DISTRIBUTION OF F_v to WHEELS (maximum)
bridge span = 35'
hook coverage = 2' from rail

$F_{v_{max}} = 36000^{lb} \cdot \frac{33}{35} \cdot \frac{1}{2} = 16970$ USE 17000 lb/wheel

STATIC VERTICAL wheel load = 13100 lb per CVI-14019

THUS,
The design dynamic factor $= \frac{17000}{13100} = 1.3$ reasonable, and acceptable OK

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DESIGN ANALYSIS

Client	WHC	WO/Job No.	E51162
Subject	WESF CRANE	Date 5-13-96	By SRWagenblast
		Checked 5/16/96	By FRV
Location	225-B	Revised	By

DESIGN LOADS PER REF 6 (FOR POTENTIAL 15 TON CRANE)

$F_V = 23750$ lb/wheel > 17000 lb/wheel OK

$F_\perp = 1950$ lb/wheel < 8000 lb/wheel

$F_{\parallel} = 1900$ lb/wheel < 6000 lb/wheel

THE HORIZONTAL LOADS USED FOR THE DESIGN OF THE CRANE RAIL AND SUPPORT BRACKETS ARE LESS THAN THE SEISMIC REQUIRMENTS FOUND IN THE REF 2 DESIGN CRITERIA.

THEREFORE, ADDITIONAL CAPACITY CALCULATIONS ARE INCLUDED ON THE FOLLOWING PAGES.

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