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Rev. 0

B Reactor Structural Analysis

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B Reactor Structural Analysis

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EXECUTIVE SUMMARY

The purpose of this document is to present results from the *B Reactor Structural Analysis* (BHI 2002) and provide future options for modifications, if desired. The B Reactor structural analysis evaluated the 105-B Building and the 116-B Reactor Stack against potential natural phenomena hazards at the Hanford Site. The analysis was performed to determine whether the 105-B Building and the 116-B Reactor Stack meet the current building codes. This information would then be used to assist in the planning of the long-term uses of the facility.

A *Comprehensive Environmental Response, Compensation, and Liability Act of 1980* interim removal action decision for an approximate 10-year time frame was documented in an Action Memorandum in 2001, which included hazard mitigation and potential public access of the 105-B Facility (DOE-RL 2001). Portions of the 105-B Reactor Building and 116-B Stack are contaminated with chemical and radiological hazardous substances. Failure of all or portions of the structures pose a potential risk to human health and the environment. The authorization basis states that the likelihood ranking for the seismic event is occasional, or likely to occur sometime in the life of an item (BHI 2000), and the consequence is an unplanned release resulting in a minor environmental contamination. Therefore, identification of those portions of the structure that would fail in a seismic condition allows proper planning when accessing the structures for allowed activities as outlined in the *Engineering Evaluation/Cost Analysis for the 105-B Reactor Facility* (DOE-RL 2001). Because the current building codes are not mandatory for current occupancy, the decision that will need to be made is what amount of risk associated with any potential failure of the structure is acceptable to the members of the public who participate in these tours.

The results of the 105-B Reactor Building and 116-B concrete stack structural analysis against the natural phenomena hazard at the Hanford Site are provided in Section 4.0 of this document. The results show that the 116-B concrete stack meets the current seismic evaluation criteria, the elevated masonry walls of the 105-B Building do not meet the current building code seismic evaluation criteria, and the remaining masonry walls meet the current building code for static force. However, no further dynamic structural analysis was performed for these walls. This means the following:

- The 116-B stack will withstand wind or a seismic phenomena hazard at the Hanford Site.

- The elevated masonry walls of the 105-B Building will fail in the seismic phenomena hazard at the Hanford Site based on the static force method in the UBC. However, a dynamic analysis of the elevated masonry walls was not completed as part of this analysis. Consequently, the areas present a potential risk to personnel during a seismic phenomena.

- The lower masonry walls will not fail in the seismic phenomena hazard at the Hanford Site, based on the static force method in the UBC. However, a dynamic analysis of these lower masonry walls was not completed as part of this analysis. Consequently, these areas present a potential risk to personnel during a seismic phenomena.

- The areas that do not present any structural concerns at this time are the control room, the entrance hallway leading from the control room to the front-face work area, and the part of the front-face work area not affected by a failure of the elevated walls.

The risk for continued occupancy in the areas not analyzed dynamically is low during the estimated 10-year period because of the following:

- All of the walls are attached to make one complete structure, thereby providing added stability and a high confidence factor that the structure will pass the dynamic analysis.

- The structure is normally unoccupied.

- Personnel (other than structural inspectors) are not allowed into the structure following a seismic event until the facility has been inspected and cleared for access.

Finally, this document provides various options (and a rough cost estimate) to complete the dynamic analysis and meet the current seismic evaluation criteria for the long-term use of the 105-B Building. However, this document does not provide any recommendations. Final recommendations will be established at a later date.

TABLE OF CONTENTS

FIGURES

TABLE

ACRONYMS

DOE	U.S. Department of Energy
EE/CA	engineering evaluation/cost analysis
IBC	International Building Code
NPH	national phenomena hazard
UBC	Uniform Building Code

METRIC CONVERSION CHART

Into Metric Units			Out of Metric Units		
If You Know	*Multiply By*	*To Get*	*If You Know*	*Multiply By*	*To Get*
Length			**Length**		
Inches	25.4	millimeters	millimeters	0.039	inches
Inches	2.54	centimeters	centimeters	0.394	inches
Feet	0.305	meters	meters	3.281	feet
Yards	0.914	meters	meters	1.094	yards
Miles	1.609	kilometers	kilometers	0.621	miles
Area			**Area**		
sq. inches	6.452	sq. centimeters	sq. centimeters	0.155	sq. inches
sq. feet	0.093	sq. meters	sq. meters	10.76	sq. feet
sq. yards	0.836	sq. meters	sq. meters	1.196	sq. yards
sq. miles	2.6	sq. kilometers	sq. kilometers	0.4	sq. miles
Acres	0.405	hectares	hectares	2.47	acres
Mass (weight)			**Mass (weight)**		
Ounces	28.35	grams	grams	0.035	ounces
Pounds	0.454	kilograms	kilograms	2.205	pounds
Ton	0.907	metric ton	metric ton	1.102	ton
Volume			**Volume**		
Teaspoons	5	milliliters	milliliters	0.033	fluid ounces
Tablespoons	15	milliliters	Liters	2.1	pints
fluid ounces	30	milliliters	Liters	1.057	quarts
Cups	0.24	liters	Liters	0.264	gallons
Pints	0.47	liters	cubic meters	35.315	cubic feet
Quarts	0.95	liters	cubic meters	1.308	cubic yards
Gallons	3.8	liters			
cubic feet	0.028	cubic meters			
cubic yards	0.765	cubic meters			
Temperature			**Temperature**		
Fahrenheit	subtract 32, then multiply by 5/9	Celsius	Celsius	Multiply by 9/5, then add 32	Fahrenheit
Radioactivity			**Radioactivity**		
Picocuries	37	millibecquerel	millibecquerels	0.027	picocuries

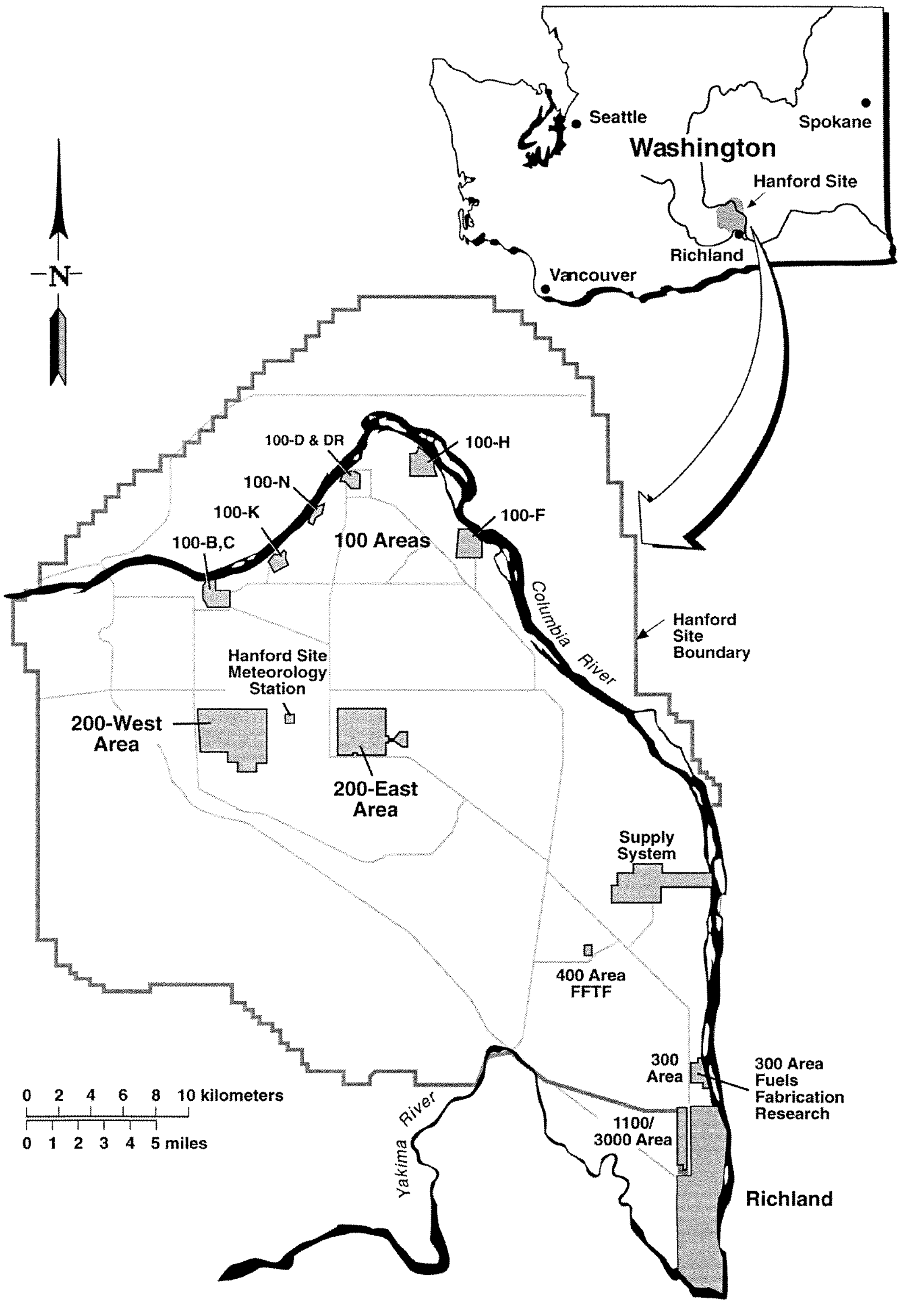
1.0 INTRODUCTION

This document presents the results of a structural analysis (BHI 2002) that was performed to evaluate the 116-B concrete stack (also referred to as the reactor stack) and the 105-B Reactor Building (also referred to as the 105-B Facility) against the current natural phenomena hazard (NPH) at the Hanford Site. This analysis was performed to determine whether the 116-B Stack and the 105-B Reactor Building meet the current building codes for the planning of the long-term uses of the 105-B Reactor Building.

The requirements for this analysis to the current building codes was identified in the *Engineering Evaluation /Cost Analysis for the 105-B Reactor Facility* (DOE-RL 2001). Although the engineering evaluation/cost analysis (EE/CA) identifies that the structure will be analyzed to the current building codes, the present occupancy of this facility is not contingent on compliance with the current building codes. However, any major modifications or change in occupancy during the 10-year timeframe may require compliance to the current building codes.

The 105-B Facility and reactor stack are located in the 100-B/C Area of the Hanford Site (Figure 1-1). The Hanford Site is located in southeastern Washington State and is operated by the U.S. Department of Energy (DOE). In accordance with previous commitments, the DOE is continuing to seek a sponsor with interest in preserving all or part of the 105-B Facility for historical purposes.

Figure 1-1. Hanford Site Map.



2.0 SITE CHARACTERIZATION

2.1 HISTORICAL SIGNIFICANCE OF THE 105-B REACTOR

The 105-B Facility was the world's first full-scale production reactor. The historical significance of the 105-B Reactor has entitled it to numerous declarations, including National Historic Mechanical Engineering Landmark by the American Society of Mechanical Engineers in 1976, and the Nuclear Historic Landmark Award. Because of its historical significance, the 105-B Facility (including the 116-B stack and the area defined by the 105-B Reactor Area exclusion fence) was listed in the National Register of Historic Places in 1992 and was designated a National Historic Civil Engineering Landmark in 1994. Since the late 1980s, guided tours have been led through portions of the 105-B Facility. Interpretive items and historical displays are exhibited in the facility along the current tour route.

In recognition of the need to preserve the physical legacy of the Manhattan Project, the DOE has declared in the "Record of Decision: Hanford Comprehensive Land Use Plan Environmental Impact Statement (HCP EIS)" (64 *Federal Register* 61615) designated land use for the 105-B Facility as high-intensity recreation to support visitor-serving activities and facilities development.

Although the DOE has stated that the 105-B Facility will be preserved for an interim period of up to 10 years, the final configuration of the reactor, and thus the requirements for remediation under *Comprehensive Environmental Response, Compensation, and Liability Act of 1980*, has not been determined.

2.2 105-B FACILITY STRUCTURAL INTERIM REMOVAL ACTION

In 2001, the *Engineering Evaluation/Cost Analysis for the 105-B Reactor Facility* (hereinafter referred to as "interim removal action EE/CA") (DOE-RL 2001) was prepared to analyze removal actions that may be performed at the 105-B Facility to protect human health and the environment. The interim removal action recommended in the EE/CA and selected in the associated Action Memorandum was hazard mitigation for a 10-year interim period to support public access along a tour route. The hazard mitigation alternative included the analysis (to the current codes) of the 105-B Facility and the reactor stack against potential NPHs at the Hanford Site. Although the interim removal action EE/CA identified that the structure will be analyzed to the current building codes, the approximate 10-year use of this facility is not contingent on compliance with current building codes; however, any major modifications or change in occupancy during the 10-year timeframe may require compliance to the current building codes; therefore, any actions and associated costs for structural upgrades to meet the current codes are not required for public use of this facility at this time. Potential structural upgrades will be identified during this interim time period to adequately assess the feasibility and cost of minimizing risks for public use of the facility when the long-term use for these structures is determined. This document does not make a determination of long-term public use or structural preservation.

2.3 FACILITY DESCRIPTION

2.3.1 116-B Reactor Stack

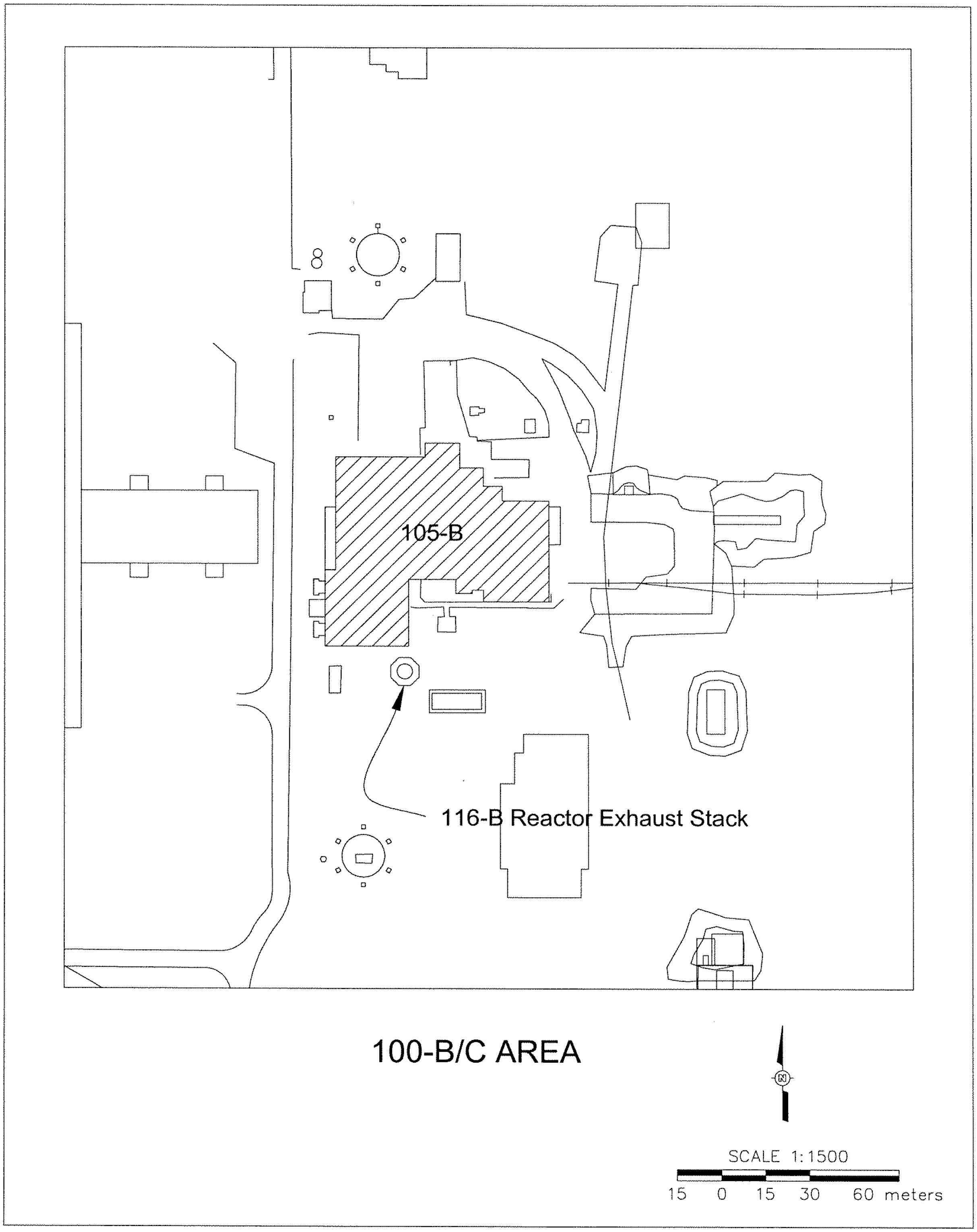
The reinforced concrete stack is located at the southeast corner of the fan house in the Hanford Site's 100-B Area. The stack is a circular shape and has an outside diameter of 5.05 m (16.58 ft) at the base. The base of the stack sits on the top of an octagonal foundation pedestal leveled with the finished grade at elevation 0'-0". The stack tapers from the base to the top with a height of 61 m (200 ft) above the finished grade. At the very top, the stack has an outside diameter of 3.1 m (10.33 ft). The lower 10.66-m (35-ft) section of the stack above the finished grade has a 0.45-m (1.5-ft) concrete thickness and a concrete strength, f_c', of 20,685 kPa (3,000 lb/in^2). The concrete thickness of the top 50.29-m (165-ft) section varies from 0.45 m to 12.7 cm (1.5 ft to 5 in.) at the very top. This 50.29-m (165-ft) section has an original concrete strength, f_c', of 17,237 kPa (2,500 lb/in^2). The octagonal concrete pedestal has a 3.5-m (11.5-ft) embedded depth and a perpendicular distance of 5.63 m (18.5 ft) between two parallel sides. The 1.8-m (6-ft)-thick base mat under the pedestal is also an octagon that has a perpendicular distance of 8.22 m (27 ft) between two parallel sides.

2.3.2 105-B Facility

The 105-B Facility (Figure 2-1) contains a reactor block, a control room, a spent fuel discharge area, a fuel storage basin, fans and ducts for ventilation and recirculating inert gas systems, water cooling systems, support offices, shops, and laboratories. The only service in operation is the lighting to support the tours as well as surveillance and maintenance. The reactor facility is a reinforced concrete structure. On top of the reinforced concrete structure, there is a steel superstructure with masonry walls built as building siding along the four sides of the superstructure. Within the reactor facility, massive reinforced concrete walls (0.9 to 1.5 m [3 to 5 ft] thick) extend upward to the height of the reactor block to provide shielding, with the upper sections constructed of concrete block (DOE-RL 2001). Asbestos, radiological, and hazardous material contamination exists in the building.

General specifications and drawings for the 100 Area buildings provide information about the masonry walls, but it is not clear whether these details and specifications specifically apply to 105-B. Therefore, the as-built information regarding the masonry walls that were removed at 105-DR and 105-F Reactors was reviewed. This information confirmed the details and specification that the masonry walls were hollow concrete blocks with no reinforcing steel and dowels except where the steel columns and beams met with the masonry wall. At this point a pilaster filled with brick and mortar wraps around the column or beam.

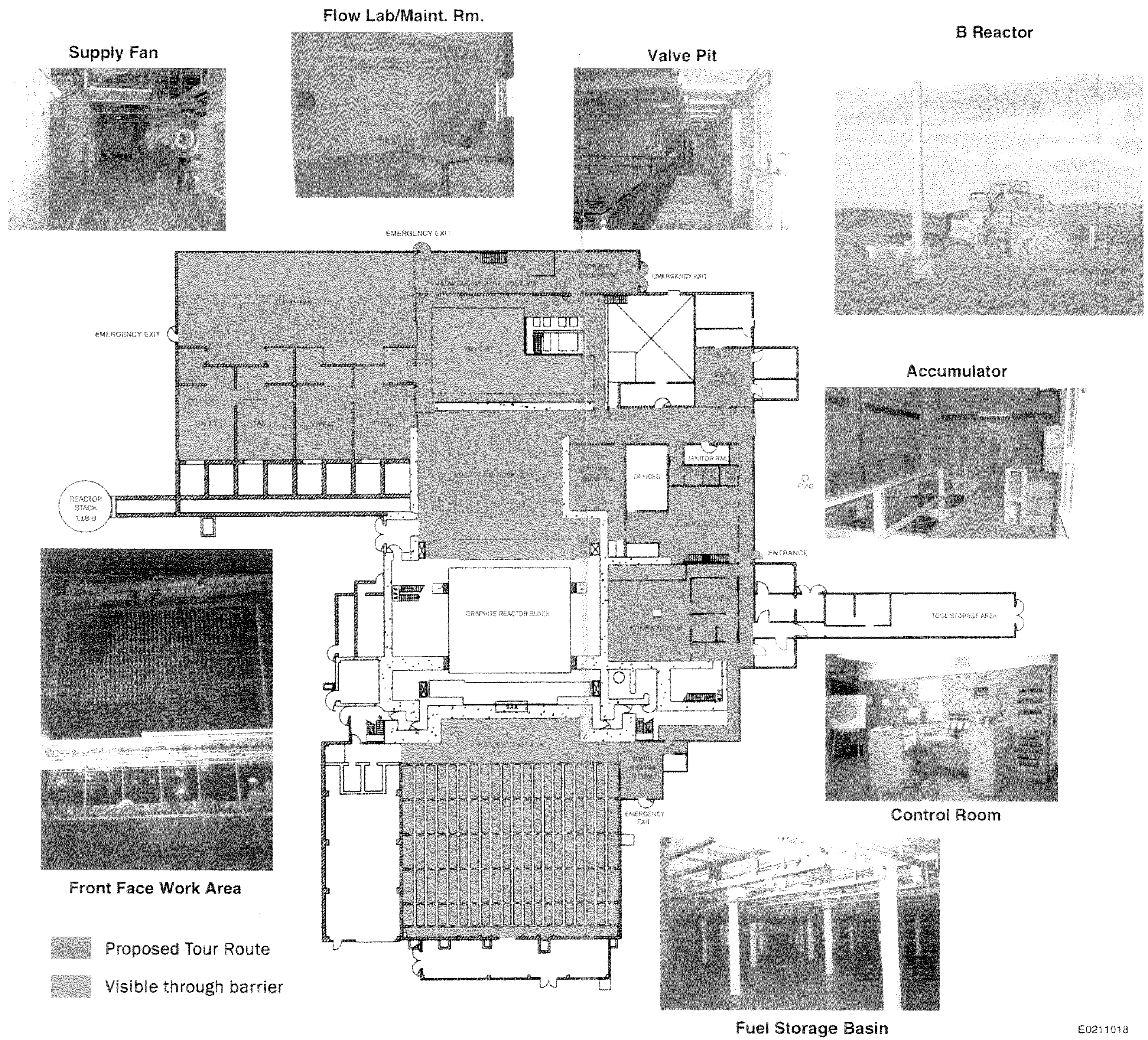
Figure 2-1. 105-B Facility and 116-B Reactor Exhaust Stack.



Roof construction of the 105-B Facility is composed of precast concrete roof tile, except over the discharge area enclosure (the rear face), the inner horizontal rod room, and the exhaust plenums. Over those areas, the roof is composed of 1.8-m (6-ft)-thick reinforced concrete (Gerber 1993). The original precast concrete tiles remain in place in all other areas of the building. Internal bracing has been added to individual precast roof panels that were showing signs of excessive deflection and corrosion (WHC 1994). The 105-B Facility underwent interim roof repair to replace flashing and mitigate drainage issues in fiscal year 2001. Total roof replacement will be contingent on the determination of the final configuration of the overall reactor structure.

Until September 11, 2001, guided public tours were conducted on a tour route through a controlled portion of the building. Figure 2-2 shows the location of the guided tour route.

Figure 2-2. Guided Tours at the 105-B Facility.



3.0 METHOD OF CALCULATION

The 105-B Facility and the reactor stack are designated as Performance Category 2 (PC 2) structures. Thus, the seismic evaluation is based on model building code seismic provision in accordance with Section 2.3.1 (page 2-8) of DOE-STD-1020-94 (DOE 1996). The structural evaluation first determined the wind and the seismic loads on the structures to determine the governing case. The loads from the governing case are used to calculate the structural demands. At the same time, the corresponding structural capacities are also calculated. The minimum factor of safety against overturning and sliding is 1.10 for code compliance. A factor of safety greater than 1.10 for stability evaluation indicates that sufficient capacity exists to resist all applicable lateral loads, and thus the structure complies with code requirements. Values below 1.10 indicate insufficient capacity.

The evaluation was performed as two separate evaluations for the following structures: the reactor stack and the 105-B Facility.

3.1 REACTOR STACK

The reactor stack was analyzed at 3-m (10-ft) section from the base at elevation 0'-0" to the top of the stack at elevation 200'-0". The wind and seismic loads were determined for each section, and both calculated loads were compared to determine the governing load case. The governing seismic load case was used to calculate the structural demands and compared with the corresponding strength capacities of the building structure.

3.1.1 Wind Loads

The wind loads on the stack are evaluated in accordance with Chapter 4 of the American Concrete Institute's *Design and Construction of Reinforced Concrete Chimney*, ACI 307-98 (ACI 1998). The wind force (along-wind load) is used to compare with seismic design force to determine the governing case used for the evaluation of the stack.

3.1.2 Earthquake Loads

The 116-B stack is designated as a Performance Category 2 (PC 2) structure. The NPH evaluation can be based on model building code provisions in accordance with Section 2.3.1 (page 2-8) of DOE-STD-1020-94 (DOE 1996).

The following two codes were used to determine the seismic design forces that are present on the structure: the 1997 Uniform Building Code (UBC) and the 2000 International Building Code (IBC). Three procedures were used to calculate seismic forces: static force, equivalent lateral force, and dynamic analysis procedure.

3.1.2.1 Static Force Procedure. The static force procedure for non-building structures in accordance with Sections 1630 and 1634 of the 1997 UBC (ICBO 1997) was used to determine the design base shear. The total force is distributed over the height of the stack (vertical distribution of force). The seismic design force, story shear, and moment diagram on the stack were used for comparison with the wind loads to determine the governing load case.

3.1.2.2 Equivalent Lateral Force Procedure. The equivalent lateral force procedure for non-building structures in accordance with Sections 1617 and 1622 of the 2000 IBC (ICBO 2000) was also used to evaluate the earthquake loads on the stack. The intent was to determine whether the stack is also seismically qualified for the earthquake loads required by the IBC, because DOE-STD-1020-2002 (DOE 2002), issued in January 2002, restricts only the use of the IBC as the model building code for seismic provisions evaluation. However, this standard is not the Bechtel Hanford, Inc. controlling document in this calculation, and the results cannot be used at this time to qualify the structure.

3.1.2.3 Dynamic Analysis Procedure. The stack fundamental frequency calculated per ACI 307-98 is a fixed-base frequency. The dynamic analysis procedure will reduce this frequency and, thus, the seismic loads. As stated in the 1997 UBC, Section 1629.8.1, and the 2000 IBC, Section 1618.1, the dynamic analysis procedure may be used. The following codes and standards are used for the procedure:

- Software used: GT STRUDL Version 25.0, release date August 30, 2000. GT STRUDL is the Bechtel standard (Program No. CE029) for analysis, design, and evaluation of new and/or existing structures.

- The Hanford Site-specific elastic design response spectra with 10% damping (1,000-year return earthquake) is used for the analysis as the ground motion representation.

- The soil-structure interaction effects are considered in the modeling of the stack. The soil impedance functions were calculated in accordance with Section 3.3 of ASCE 4-98 (ASCE 2000).

- Thirty modes were analyzed in the dynamic analysis eigenvalues and 14 dominating modes were selected in the response spectrum analysis.

3.1.3 Strength of the Stack to Withstand Lateral Forces

When evaluating the requirements of the stack to withstand the forces generated per the static force, the equivalent lateral force procedures and/or dynamic procedure, the following requirements were reviewed:

- Soil-bearing pressure
- Overturning
- Lateral sliding.

When evaluating the strength of the stack to withstand the seismic forces calculated per the dynamic analysis procedure, the moment capacity and shear capacity of the stack using for demand and capacity ratios are calculated in accordance with the following codes:

- Moment capacity: ACI 307-98, *Design and Construction of Reinforced Concrete Chimney* (ACI 1998)
- Shear capacity: ACI 318-95, Building Code Requirements for Reinforced Concrete (ACI 1995).

3.2 105-B FACILITY

Three groups of walls were identified for analysis. The limiting wall for each group was determined and used to qualify the other walls within the group. The wind and seismic loads of the limiting walls were determined if the group has any exterior walls. Otherwise, only seismic loads are calculated for the group, which has interior walls only. The applicable governing loads were used for the limiting walls of the group to calculate the masonry wall maximum out-of-plane bending stress, which was compared with the allowable stress (modulus of rupture) in accordance with Chapter 21 of the 1997 UBC. The groups of walls and the analyzed elevations are as follows:

- Elevated masonry walls: 46'-4$^1/_2$" to 93'-0" elevation
- Masonry walls at ground level: 0'-0" elevation
- Interior walls: 0'-0" elevation.

3.2.1 Wind Loads

The wind loads on the exterior masonry walls were evaluated in accordance with wind loads established in Calculation No. 100DR-CA-C0003, Rev. 0 (BHI 2001a). The wind loads were calculated for the governing elevations using the basic Hanford Site wind maximum velocity of 38 m/s (85 mph).

3.2.2 Earthquake Loads

Accelerations corresponding to the elevations of the masonry walls in 105-DR were obtained from Calculation No. 0100X-CA-C0030, Rev. 1 (BHI 2001b) to establish the earthquake loads used for the evaluation of the masonry walls in this study. In BHI (2001b), a dynamic analysis was performed for DR Reactor interim safe storage in which the elevated masonry walls were removed and replaced with steel framing and metal siding. Because leaving the masonry walls intact would not significantly affect the response of the overall structure, using the accelerations from that calculation to calculate the seismic forces in 105-B would result in a more realistic evaluation for the masonry walls.

3.2.3 Strength of the Building to Withstand External Forces

The following information was used to determine the strength of the masonry walls:

- The Decontamination and Decommissioning/Interim Safe Storage Project stated that the masonry walls in the reactor buildings (i.e., those that have been demolished and removed) were unfilled and had no reinforcement.

- It was assumed that the cracks in most of the masonry walls were caused either due to lack of reinforcement or aging. These cracks were patched with silicone caulking materials to mitigate water intrusions without providing any additional structural integrity to the masonry walls.

- Because of the lack of masonry wall as-built drawings and specification information, the value of 10,343 kPa (1,500 lb/in^2) was used for the compressive strength of the masonry. This value, taken from Chapter 21, Section 2107, "Working Stress Design of Masonry," of the 1997 UBC, was used for the evaluation of masonry walls.

A critical simple span or continuous beam was selected as a representative in each group for the use of evaluation. The steel beam or column used for the support of the masonry wall was evaluated further if the masonry wall met the lateral load requirements.

The following items were evaluated for the masonry walls:

- Out-of-plane bending
- Out-of-plane shear
- In-plane bending
- In-plane shear.

4.0 CONCLUSIONS OF CALCULATION

The conclusions of the calculation are provided for both the reactor stack and the 105-B Facility in the following subsections.

4.1 116-B STACK

Based on the performed analyses for the stack, the following information was determined:

- The calculated wind loads varied from 189 kg (416 lb) at the 3-m (10-ft) base section up to 1,784 kg (3,933 lb) at the very top 3-m (10-ft) section of the stack. The earthquake loads varied from 581 kg (1,280 lb) at the 3-m (10-ft) base section up to 12,002 kg (26,460 lb) at the very top 3-m (10-ft) section of the stack. These numbers identified that the earthquake loading provides the bases for the evaluation.

- The static force procedure in the 1997 UBC (which is the current code for structural analysis or evaluation) was used to determine if the factor of safety met the minimum requirement (equal to and/or greater than 1.10) for the following stability evaluations:

  - Overturning
  - Lateral sliding.

  The factor of safety for the lateral sliding was greater than 1.10, and the factor of safety for the overturning was less than 1.10. The actual soil-bearing pressure was also calculated and was out of the range of the chart used for the calculation of actual soil-bearing pressure. This indicated that the actual soil-bearing pressure due to earthquake forces is much greater than allowable (0.509 kPa [10.64 lb/ft^2]).

Based on the evaluation results summarized above, the stack does not meet the 1997 UBC static force procedure requirements.

The stack structural evaluation performed in accordance with the 2000 IBC equivalent lateral force procedure indicates that the stack base shear is 50% lower than that calculated in accordance with the 1997 UBC static force procedure. The factors of safety for the lateral sliding and against overturning were greater than 1.10, and the soil-bearing pressure was greater than the allowable. Therefore, the seismic capacity of the stack is marginal using the equivalent lateral force procedure. However, because 2000 IBC is not the Bechtel Hanford, Inc. controlling document in this calculation, the results cannot be used at this time to qualify the structure.

As a result of the above analyses, a dynamic analysis was completed to streamline the stack frequency and thus reduce the seismic loads. The dynamic analysis also took soil-structure interaction effects into consideration in the modeling of the stack.

Table 4-1 summarizes the calculated soil-bearing pressure, the stability, and structural capacities checks for the stack using the forces from the dynamic analysis output for the calculations. The applicable code allowable factor of safety for stability was used. The critical section of the stack jointed to the top of the foundation pedestal where maximum bending and shear occur was checked for its demands and capacities. The demands of stack critical section are based on the results from the dynamic analysis, and the corresponding bending moment and shear capacities are calculated in accordance with the stack as-built reinforcing steel. Table 4-1, column (1) indicates that the actual soil-bearing pressure is less than the allowable. Columns (2) and (3) show that both the actual factor of safety against overturning and against sliding, respectively, is greater than the allowable. Column (4) shows the stack critical section structural demand and capacity ratio for bending and shear. Both ratios are less than unity, which indicate that sufficient capacity exists to resist all applicable loads. The range of difference between unity and the ratio for any member is indicative of the additional margin that would exist in the member.

Table 4-1. Summary of Stack 116-B Soil-Bearing Pressure, Stability, and Demand-Capacity Ratio.

(1)		(2)		(3)		(4)	
Soil-Bearing Pressure (q_{max})		**Factor of Safety Against Overturning**		**Factor of Safety Against Sliding**		**Demand / Capacity**	
Actual	**Allowable**	**Actual**	**Allowable**	**Actual**	**Allowable**	**Moment**	**Shear**
10.16k/ft^2	10.64 k/ft^2	2.04	1.10	3.16	1.10	$\frac{14,085}{17,099} = 0.82$	$\frac{241^k}{794^k} = 0.30$

Based on the results from the dynamic analysis summarized above, the stack is seismically qualified per the 1997 UBC, Section 1629.8.1, and the 2000 IBC, Section 1618.1.

4.2 105-B FACILITY

Based on the performed analysis, the following information was determined:

- Wind loads govern the design of all exterior masonry walls at the grade level (0'-0"). These walls meet the wind load requirements in accordance with the static force method of the 1997 UBC. However, a dynamic analysis of these lower masonry walls was not completed as part of this analysis. Consequently, these areas present a potential risk for long-term use in the event of a seismic phenomena.

- Seismic loads govern the interior walls at the grade level (0'-0"). These walls meet the seismic load requirements in accordance with the static force method of the 1997 UBC. However, a dynamic analysis of these lower masonry walls was not complete as part of this analysis. Consequently, these areas present a potential risk for long term use in the event of a seismic phenomena.

- Seismic loads govern the elevated masonry walls (46'-4½" to 93'-0"). These walls do not meet the earthquake load requirements with the static force method of the 1997 UBC. A dynamic analysis of these walls was not part of this analysis. Consequently, these areas present a potential risk for long term use in the event of a seismic phenomena.

- A costly dynamic analysis was not completed on the structure because of the high degree of confidence that the existing upper elevations would also fail this analysis. Therefore, the dynamic analysis will be completed when a path forward is determined for the upper elevation walls, which will be made at a later date.

- The areas that do not present any structural concerns for long term use of the facility are the control room, the entrance hallway leading from the control room to the front-face work area, and the part of the front-face work area not affected by a failure of the elevated walls.

- The risk to personnel in the other areas of the facility during the 10 year interim state is minimal because of the following:

  - All of the lower walls are attached to adjacent structures thus making one complete structure thereby providing added stability and a high confidence factor that the lower structures will pass the dynamic analysis,

  - The structure is normally unoccupied,

  - Personnel are not allowed into the structure following a seismic event (other than structural inspectors) until the facility has been inspected and cleared for access.

5.0 OPTIONS

5.1 GENERAL

A number of options could be considered if it is determined that the long-term use of the facility is to become a museum. These options will ensure that the public will not be exposed to falling debris in a seismic event. Some options are identified below (not necessarily all-inclusive):

- Option 1: Remove all elevated masonry walls and replace with steel framing and light metal siding. Complete a dynamic analysis on the new elevated and existing lower portions of the structure, as outlined in Section 4.2.

- Option 2: Keep the elevated masonry walls. Upgrade the structure by providing a lateral load-resisting system that could be reinforced by either vertical steel plates, reinforced fiber sheets, or any other system that would be cost effective. Complete a dynamic analysis on the new elevated and existing lower portions of the structure, as outlined in Section 4.2.

- Option 3: Move the area of occupancy back in the front-face work area and complete a dynamic analysis on the existing elevated and lower portions of the structure, as outlined in Section 4.2.

- Option 4: Provide a barrier in the front-face work area that will protect the public and complete a dynamic analysis on the existing elevated and lower portions of the structure, as outlined in Section 4.2.

- Option 5: Either move the area of occupancy back, or provide a barrier in the front-face work area and allow access only in the control room, front-face work area, and the hallway in between.

5.2 OPTION 1: REMOVE THE ELEVATED MASONRY WALLS AND REPLACE WITH STEEL FRAMING AND LIGHT METAL SIDING

This option will require the following steps:

- Installation of bracing before the masonry walls are removed
- Demolition
- Installation of permanent structural bracing
- Installation of metal siding
- Complete dynamic analysis on the elevated and the lower elevations of the structure.

5.2.1 Installation of Bracing

Installation of bracing would be necessary to ensure structural integrity of the existing roof when the concrete blocks above the reactor block are removed.

5.2.2 Demolition

Demolition would apply to the removal of the block walls above the reactor block, and may be preceded by dismantling facility components (i.e., severing and removing ductwork). Figures 5-1 and 5-2 show the location of the demolition. Demolition generally means small-scale facility destruction using hand tools or other industrial methods.

Figure 5-1. 105-B Facility Plan View.

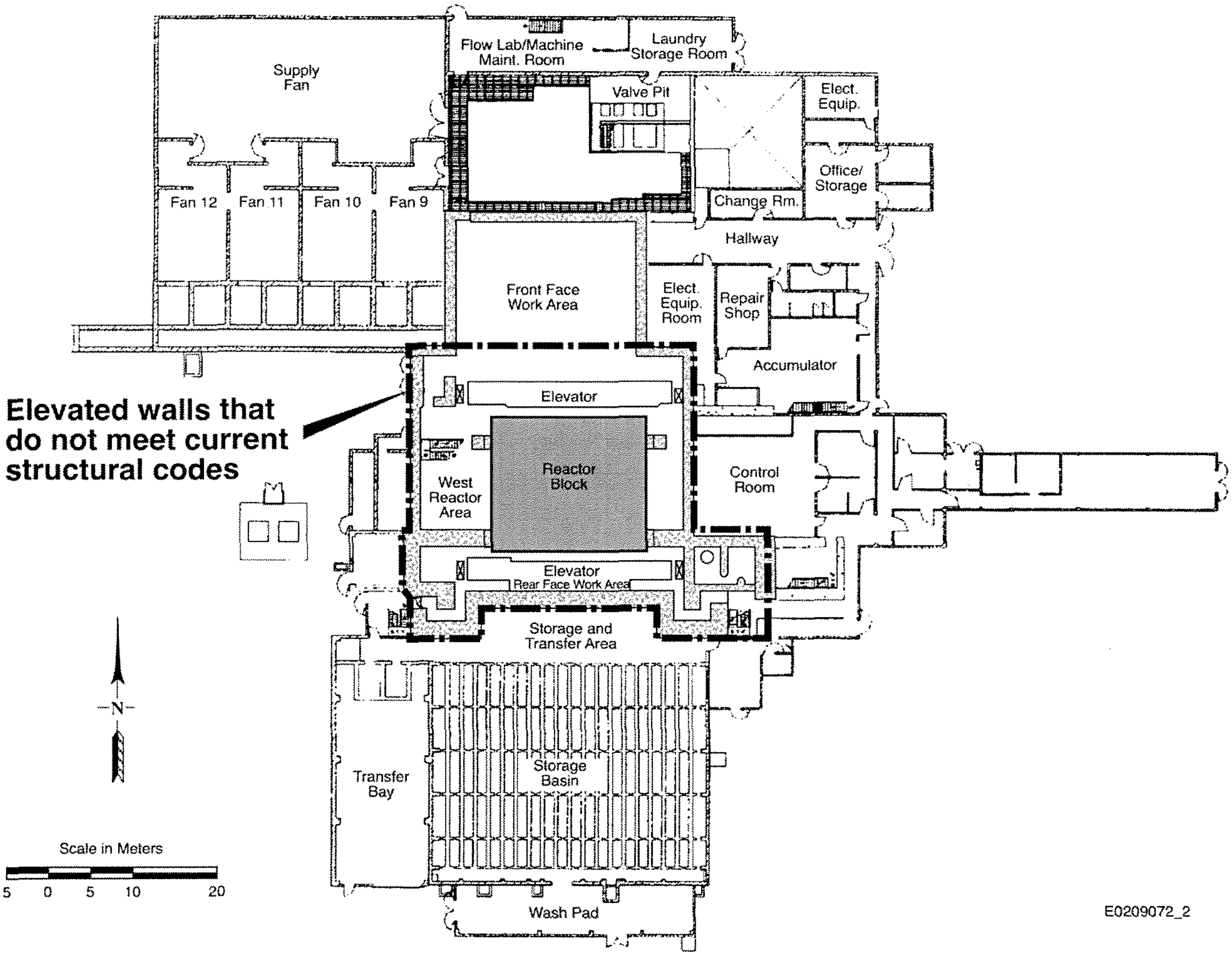
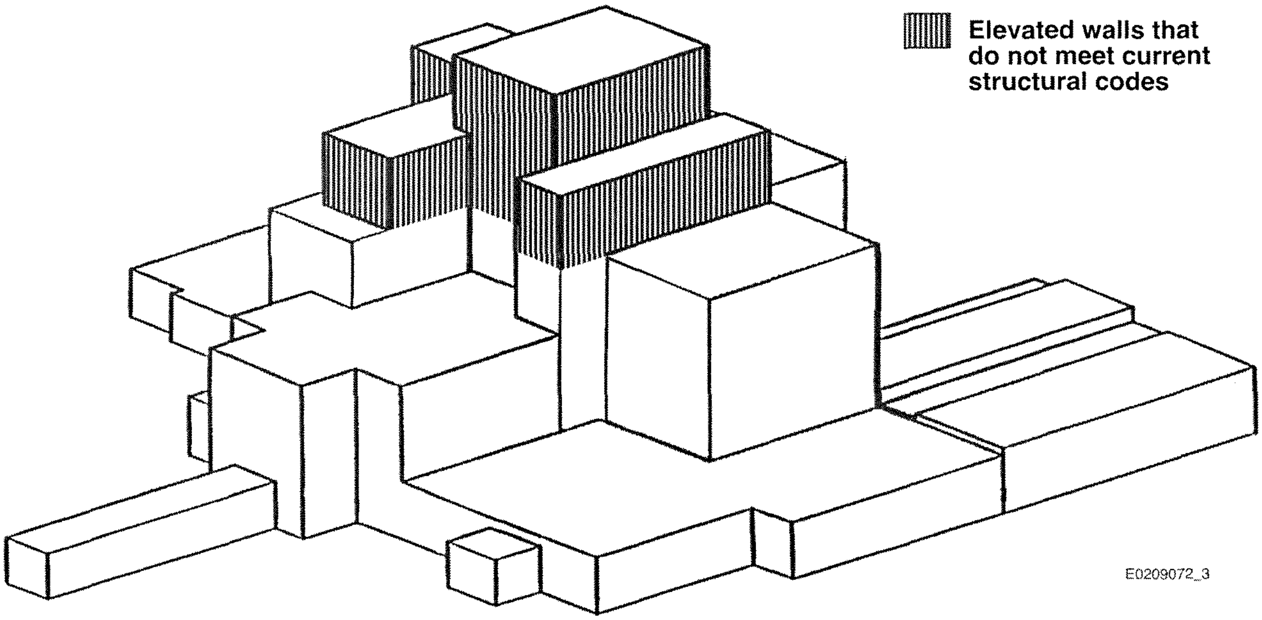


Figure 5-2. Isometric View of the 105-B Building.



Demolition methods would be selected based on the elements to be demolished, remaining radionuclide contamination, location, and integrity of the walls. Any fixed contamination on sections of the structure to be demolished would be separated and disposed. Dust-suppression techniques would be employed during demolition activities.

5.2.3 Installation of Permanent Structural Bracing

Upon removal of the applicable components and decontamination and demolition of the concrete blocks above the reactor block, seismic resisting bracing will be installed.

5.2.4 Installation of the Metal Siding

Upon completion of the installation of the structural bracing, a skin would be installed around the structure to keep out the elements. The skin would have to be designed for wind or seismic loads.

5.2.5 Dynamic Analysis

A dynamic analysis will be completed on the revised elevated walls and the existing lower walls to allow access throughout the facility, as shown in Figure 2-2.

5.3 OPTION 2: KEEP ELEVATED MASONRY WALLS AND PROVIDE LATERAL LOAD-RESISTING SYSTEM

This option will require the design of reinforcement for the elevated masonry walls. The design of reinforcement for the elevated masonry walls is to strengthen the masonry walls as a lateral resisting system. The existing structural steel framing may require modifications to be able to transmit the lateral loads to the concrete deck at elevation 56'-4".

5.3.1 Removal of Obstructions

Removal of obstructions would apply to dismantling facility components (i.e., severing and removing ductwork). Figures 5-1 and 5-2 show the location of the demolition. Demolition generally means small-scale facility destruction using hand tools or other industrial methods.

Demolition methods would be selected based on the elements to be demolished, remaining radionuclide contamination, location, and integrity of the walls. Any fixed contamination on components to be removed would be separated and disposed. Dust-suppression techniques would be employed during removal activities

5.3.2 Installation of Lateral Load-Resisting System

Upon removal of the applicable obstructions above the reactor block, the lateral load-resisting system will be installed.

5.3.3 Dynamic Analysis

A dynamic analysis will be completed on the revised elevated walls and the existing lower walls to allow access throughout the facility, as shown in Figure 2-2.

5.4 OPTION 3: MOVE THE AREA OF OCCUPANCY BACK AND COMPLETE DYNAMIC ANALYSIS ON THE LOWER PORTIONS OF THE STRUCTURE

The wall between the front-face work area and the elevator would fall down in a seismic event, and the debris could fall on members of the public (Figure 2-2). Netting that is installed below that wall would catch limited debris; however, it is not intended to hold the whole wall in the event of failure. People are presently allowed to come close enough to the reactor face and in the electrical room that they would be below this wall. If the people are not allowed in the electrical room and close to the reactor face, the wall could not fall on them.

A dynamic analysis will be completed on the existing elevated and lower walls to allow access throughout the facility, as shown in Figure 2-2.

5.5 OPTION 4: PROVIDE A BARRIER TO PROTECT THE PUBLIC FROM THE FALLING WALL AND COMPLETE A DYNAMIC ANALYSIS ON THE LOWER PORTIONS OF THE STRUCTURE

As stated in option 3, the wall between the front-face work area and the elevator would fall down in a seismic event and the debris could fall on members of the public (Figure 2-2). Netting that is installed below that wall would catch limited debris; however, it is not intended to hold the whole wall in the event of failure. Rather than move the people further away from the reactor face, additional inside and outside barriers could be installed that would protect the public in the front-face area and the electrical room in the event of complete failure of that wall. However, the interior barrier would reduce the view of the front face of the reactor.

A dynamic analysis will be completed on the existing elevated and lower walls to allow access throughout the facility, as shown in Figure 2-2.

5.6 OPTION 5: MODIFY TOUR ROUTE

As stated in option 3, the wall between the front-face work area and the elevator would fall down in a seismic event, and the debris could fall on members of the public (Figure 2-2). In addition, the stability of the lower walls is uncertain until a dynamic analysis can be completed. These affected areas can be removed from the tour route, which would limit the tour route to the control room, parts of the front-face work area, and the hallway leading from the control room to the front-face work area.

5.7 ROUGH PRICE RANGE FOR THE OPTIONS

Although a detailed estimate was not prepared for these options, it can be assumed for preplanning purposes that a rough price range of costs for these options is anywhere from \$30,000 to as high as \$1,500,000. If it is decided to pursue any of these options in the future, a detailed rough order of magnitude estimate can be obtained.

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