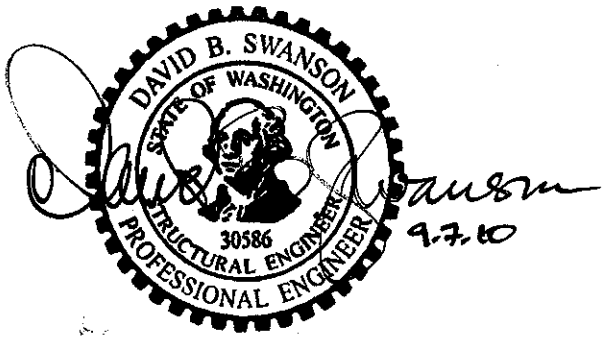
# Natural Resources Building Seismic Evaluation

ASCE 31-03 Tier 3 Seismic Evaluation

September 2010

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# List of Acronyms

| | |
|---|---|
| ASCE | American Society of Civil Engineers |
| ATC | Applied Technology Council |
| BSE | Basic Safety Earthquake |
| BSO | Basic Safety Objective |
| BSSC | Building Seismic Safety Council |
| C2 | ASCE 31-03 Concrete shear Walls (With Stiff Diaphragms) Checklist |
| CP | Collapse Prevention |
| DBE | Design Based Earthquake |
| DCR | Demand Capacity Ratio |
| ELF | Equivalent Lateral Force |
| FEM | Finite-Element Model |
| FEMA | Federal Emergency Management Agency |
| IBC | International Building Code |
| IEBC | International Existing Building Code |
| ICSSC | Interagency Committee on Seismic Safety in Construction |
| IO | Immediate Occupancy |
| LDP | Linear Dynamic Procedure |
| LFRS | Lateral Force Resisting System |
| LS | Life Safety |
| MCE | Maximum Considered Earthquake |
| NDP | Nonlinear Dynamic Procedure |
| NPC | Nonstructural Performance Category |
| NRB | Natural Resources Building |
| OP | Operational |
| PBEE | Performance Based Earthquake Engineering |
| PEER | Pacific Earthquake Engineering Research Center |
| REACH | Rapid Evaluation and Assessment Checklist |
| REAP | Rapid Evaluation and Assessment Program |
| RP | Recommended Practice |
| S1 | ASCE 31-03 Steel Moment Frame (With Stiff Diaphragms) Checklist |
| SPC | Structural Performance Category |
| SRSS | Square Root of the Sum of Squares |
| UBC | Uniform Building Code |
| UFC | Unified Facilities Criteria |
| USGS | United States Geological Survey |
| WSMF | Welded Steel Moment-resisting Frame |

# Executive Summary

A seismic screening and structural evaluation were performed for the State of Washington Natural Resources Building to investigate seismic deficiencies and possible weakening of the building structure due to recent earthquake activity. Constructed in 1993, the eight-story building consists of structural steel moment framing and concrete shear walls. The building is instrumented with seismograph with 17 channels and a seismic monitoring system. Seismologists from the state of Washington Department of Natural resources reviewed the records registered by these systems for earthquakes in 1999 and 2001. Their works suggests that the recent earthquake activity may have weakened the building structure.

A Tier 3 seismic evaluation was performed in accordance with ASCE 31-03, *Seismic Evaluation of Existing Buildings*, and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings*. The Natural Resources Building's expected structural performance for the Life Safety (LS) and Collapse Prevention (CP) performance objectives was evaluated. A report on the building was prepared, including a description of the structural system and results of the evaluation. This report includes color-coded demand-to-capacity ratio (DCR) key plans illustrating the degree and distribution of deficiencies identified for the building.

Findings show that the Natural Resources Building is expected to experience displacements in excess of the prescribed targets defined by applicable codes and standards. The detailed analysis also showed most shear walls had enough strength in both shear and flexure to resist loads in a 10%/50-year earthquake. However, the shear walls showed unfavorable results in shear strength, with DCRs larger than 1.0, for the 2%/50-year earthquake. Select moment-frame beams have limited strength and ductility due to the high forces and displacements for both earthquakes. In addition, the building is constructed with welded steel moment-frame (WSMF) beam-column joint connections of a type known to perform poorly, even in moderate seismic events. Connections of this type have not been permitted since the 1994 Northridge Earthquake.

The recommendations section describes options for mitigation of seismic deficiencies. One recommendation consists of enhancing the existing concrete walls, installing a concrete topping slab overlay, and strengthening collectors or installing additional collectors. Since this approach tends to be a disruptive approach, a second, less disruptive, recommendation is preferred. To improve the global strength and stiffness of the building, install additional steel braced frames and stiffen and strengthen the existing Welded Steel Moment-Frame (WSMF) members and connections. The full extent of a seismic upgrade is not known and a detailed construction cost estimate has not been prepared. Based on the deficiencies, building age, occupancy, and the assumption that the building must remain in operation while under construction, construction cost could range from $5 M to $15 M.

# 1.0 Introduction

## 1.1 Background

Over the years, there has been a concern over the seismic risk of the State of Washington's Natural Resources Building (NRB). Various events have occurred from 1999 to present that bring to question the seismic risk of the NRB. The following figure shows a timeline of events that occurred since 1999 until the publishing of this report. The concern initiated a seismic structural evaluation of the building. The evaluation reviewed the expected seismic performance of the NRB to identify potential structural deficiencies that may affect the building's performance during an earthquake.

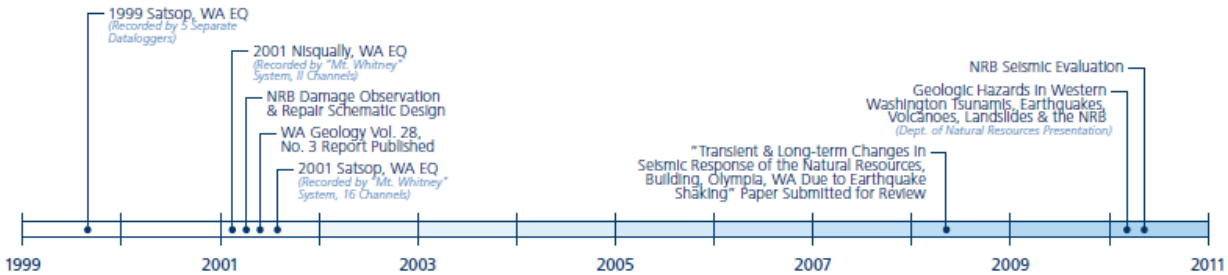



Figure 1-1. Natural Resources Building Timeline of Events.

The seismic evaluation does not consider compliance with the seismic requirements of the current building code for new construction. Buildings designed prior to the current or previous building codes often include structural configurations and detailing that do not comply with current code requirements. The NRB was designed to the 1988 Edition of the Uniform Building Code (UBC). The 1988 UBC was superseded by various editions of the UBC and International Building Codes (IBC). Currently, the governing code is the 2009 IBC. Buildings designed to older building code standards, such as the NRB, are evaluated using evaluation and design guidelines specifically developed for existing structures by the Federal Emergency Management Agency (FEMA) and the American Society of Civil Engineers (ASCE). The International Building Code includes these documents as reference standards for the seismic evaluation of existing buildings.

The current standard for the seismic evaluation of existing buildings is the ASCE Standard 31-03, *Seismic Evaluation of Existing Buildings* (ASCE 31). ASCE 31 is a screening and evaluation document used to identify potential seismic deficiencies that may require additional seismic evaluation or hazard mitigation. The document presents a three-tiered review process implemented by following a series of comprehensive checklists and "quick check" calculations. Each successive tier is designed to perform an increasingly refined evaluation procedure for seismic deficiencies identified in previous tiers in the process. A summary of each tier is provided in Figure 1-2.

Tier 1 checklists screen for potential seismic deficiencies by examining the lateral systems and details of the structure in comparison with configurations that have historically caused poor seismic performance in similar buildings. Tier 1 includes basic analyses for primary components

of the lateral system, such as columns, frames, and beams. Tier 2 provides more detailed evaluations for deficiencies identified in the Tier 1 review. Tier 3 involves an even more detailed analysis and review of the demand and capacity of each building component.

**TIER 1 – Screening Phase**

- Checklists of evaluation statements to quickly identify potential deficiencies
- Requires field investigation and/or review of record drawings
- Analysis limited to "Quick Checks" of global elements
- May proceed to Tier 2, Tier 3, or rehabilitation design if deficiencies are identified

**TIER 2 – Evaluation Phase**

- "Full Building" or "Deficiency Only" evaluation
- Address all Tier 1 seismic deficiencies
- Analysis more refined than Tier 1, but limited to simplified linear procedures
- Identify buildings not requiring rehabilitation

**TIER 3 – Detailed Evaluation Phase**

- Component-based evaluation of entire building using reduced ASCE 41 forces
- Advanced analytical procedures available if Tier 1 and/or Tier 2 evaluations are judged to be overly conservative
- Complex analysis procedures may result in construction savings equal to many times their cost

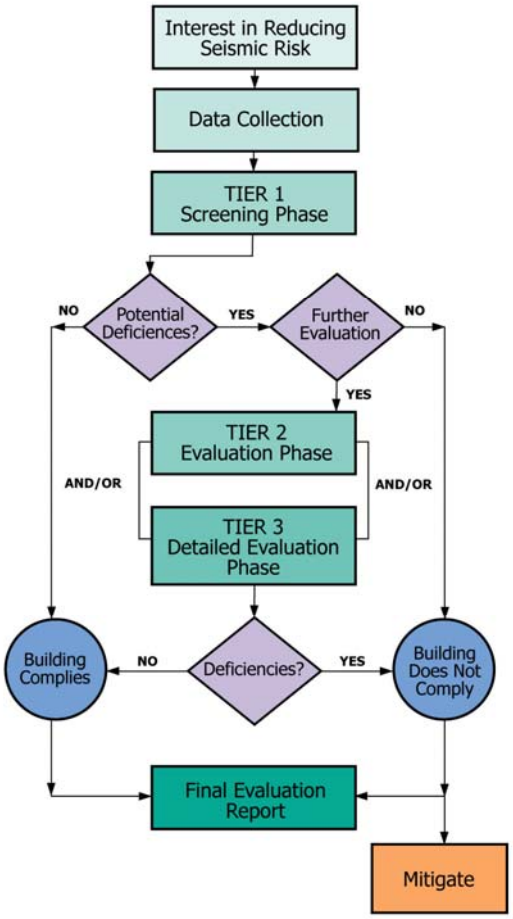



Figure 1-2. Flow Chart and Description of ASCE 31 Seismic Evaluation Procedure.

Since ASCE 31 is an evaluation standard, it is written to accept greater levels of damage within each performance level than permitted by seismic retrofit design standards. This is consistent with the historic practice of evaluating existing buildings for slightly lower criteria than those used for design. ASCE 31 quantifies this difference by using a 0.75 reduction factor on seismic load demand when performing a Tier 3 evaluation. This essentially lowers the reliability of achieving the selected performance level from about 90 percent to 60 percent. This practice generally minimizes the need to rehabilitate structures with comparatively modest deficiencies relative to the desired earthquake performance level. Using this approach, it is possible that buildings that would otherwise pass the FEMA 310 LS evaluation may need rehabilitation. Please refer to the Appendices for a more thorough description of performance-based earthquake engineering.

## 1.2 Purpose

This report presents the findings from ASCE 31-03 and ASCE 41-06 seismic evaluations of the Natural Resources Building. The baseline performance level for a standard building is referred to as the Basic Safety Objective (BSO). The BSO is defined as providing Collapse Prevention performance at the 2%/50-year event and Life Safety performance at the 10 percent probability of exceedance in a 50-year event (10%/50). The Natural Resources Building was evaluated with respect to Life Safety (LS) and Collapse Prevention (CP) Performance Levels. After a seismic event, buildings that meet a LS Performance Objective are expected to have moderate damage with a functioning gravity system to prevent falling hazards, but may experience damage to architectural finishes, mechanical systems, electrical systems, and other nonstructural items. Buildings that meet a CP Performance Objective are expected to have an extensive damage, but still contain a functioning gravity system, although may be near collapse

## 1.3 Scope of Services

The following is a summary of the procedures used to perform the structural evaluation. All buildings within the scope were evaluated in accordance with the requirements of ASCE 41-06.

1. Review record drawings provided by the State of Washington to determine the extent and configuration of the structural system.

2. Field-examine existing conditions. Conduct a walk-through to examine and obtain photographic documentation of building framing, finishes, and layout.

3. Complete an ASCE 31-03 Tier 2 evaluation for the Natural Resources Building. The building is being evaluated for the Life Safety Performance Objective. A copy of the evaluation is included in Appendix A.

4. Create a three-dimensional finite-element analysis model for use in a linear dynamic time history analyses. These analyses assist in determining the distribution of expected seismic force and displacement from the 1999 and 2001 earthquakes. This information is used to determine if the past earthquakes reduced the safety of the building structure. The linear dynamic model is also used to determine the building's expected seismic performance from current code-level earthquake loads.

5. Complete an ASCE 31-03 Tier 3 detailed seismic evaluation for the Natural Resources Building using ASCE 41-06 component-based procedures. The building is being evaluated for Basic Safety Objectives; Collapse Prevention in 2%/50-year earthquake and Life Safety in 10%/50-year earthquake. Evaluation of possible seismic deficiencies in both the primary and secondary structural elements and components throughout the building will be completed.

6. Complete and submit a written draft report and final report summarizing the findings of the above stated services.

7. Materials testing, hazardous materials studies, and destructive testing of the Natural Resources Building is not included in the scope of work. Geologic and nonstructural checklists are not included in the scope of work.

# 2.0 Evaluation Criteria

This report contains a detailed seismic evaluation of the Natural Resources Building. A seismic evaluation was performed using ASCE 31-03 (Tiers 2 and 3) based on ASCE 41-06 component-based procedures. This section provides a general description of the analysis procedures, mathematical modeling, and seismic evaluation and design criteria used for the evaluation.

## 2.1 General Criteria

### *2.1.1 ASCE 41-06 Performance Objectives*

ASCE 31 references the procedures of FEMA 356 for performing Tier 3 evaluations. However, ASCE 41-06 has superseded FEMA 356. ASCE 31 and ASCE 41 are codified design documents for the seismic evaluation of existing structures derived from previous standards (FEMA 310 and FEMA 356).

The codification of FEMA 310 into ASCE 31 preceded FEMA 356, resulting in the out-of-date reference to FEMA 356 for performing Tier 3 evaluations. FEMA 356 has since been superseded by ASCE 41 and is the current document for evaluating and retrofitting existing structures. Consequently, the Tier 3 structural evaluation presented in this report is performed using the criteria of ASCE 41-06.

Minimum regulations for existing buildings are also described in the International Existing Building Code (IEBC). The IEBC uses prescriptive and performance-related provisions to provide alternative approaches to repairing existing buildings in compliance with current building code requirements. Both ASCE 41-06 and the 2009 International Building Code (IBC), the current new-building design code, are referenced for repairing and altering existing buildings. Consequently, the conclusions and recommendations presented in this report should take into account the criteria of the IEBC.

Building performance is a combination of the performance of structural and nonstructural components. Structural performance is related to the amount of lateral deformation or drift of the structure and the capacity or ability of the structure to deform. Structural performance levels, based on the shear demand on the building and the deformation for each performance level, are shown in Figure 2-1.

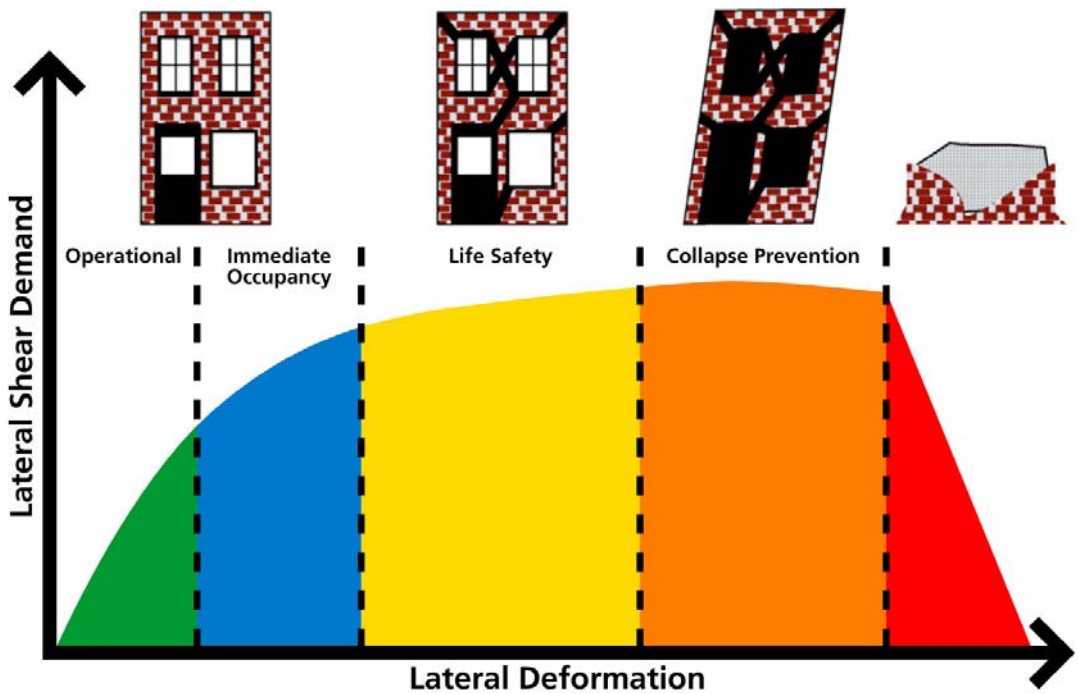



Figure 2-1. Building Performance Levels.

The selected Structural Performance Objectives and the recommended inter-story drift limits for the building are summarized in the following table:

Table 2-1. ASCE 41-06 Performance Objective Summary.

| Building | BSE-1 (10%/50-yr) Event | | BSE-2 (10%/50-yr) Event | |
|---|---|---|---|---|
| | Objective | Drift Limit | Objective | Drift Limit |
| Natural Resources Building | LS | 1% | CP | 2% |

For more information on seismic evaluation guidelines, see Appendix B, Performance Based Earthquake Engineering.

### *2.1.2 As-Built Information*

As-built information, including configuration of the structural system, connection details, and material strengths, was taken from existing record drawings. This information was used to determine building dimensions, build three-dimensional finite-element models, locate seismic joints, and estimate weights and loads. The accuracy of information contained within record drawings was confirmed through field verification, as prescribed by ASCE 41-06 and a review of original construction records. Material testing was not conducted.

### *2.1.3 Acceptance Criteria*

The Natural Resources Building was evaluated on its ability to meet Life Safety (LS) structural performance for the 10%/50-year event and Collapse Prevention (CP) structural performance for

the 2%/50-year event. Each element and component was classified as primary or secondary. All elements judged to contribute significantly to resisting seismic effects were considered primary. Such elements included concrete shear walls, moment-frame beams and columns, and roof and floor diaphragms.

### *2.1.4 Concrete Shear Wall Seismic Deficiencies*

Past earthquakes have indicated that reinforced concrete shear walls have good seismic performance. Concrete shear walls are placed around the building or at the perimeter during design to resist gravity and lateral loads. Their design is based on their behavior in both shear and flexure. Unfavorable performance is usually related to inadequate construction, which includes inadequate density, missing or improper detailing of reinforcement, and lack of confinement. The building may exhibit unacceptable seismic performance, even though it was designed in accordance with the appropriate building codes of the time. Performance is evaluated based on today's increased seismic load design code requirements and increased knowledge of shear wall reactions during seismic events.

### *2.1.5 Welded Steel Moment Frame (WSMF) Seismic Deficiencies*

The 1994 Northridge earthquake in southern California uncovered significant structural deficiencies in buildings with welded steel moment-resisting frames (WSMF). Approximately 100 buildings in the affected area experienced beam to column fractures. The damaged structures covered a wide range of heights (1 to 26 stories) and ages (several years old to 30 years old). Although no WSMF buildings collapsed as a result of the Northridge earthquake, the poor seismic performance of these buildings required several emergency code changes and a significant national research program to improve WSMF seismic performance.

The Natural Resources Building consists of two types of lateral force-resisting systems. One of the systems is concrete shear walls; the other was constructed with these older and earthquake-susceptible WSMF structural systems. Although the building was designed in accordance with the appropriate building codes, the older, code-prescriptive style of WSMF connections used in the building may exhibit unacceptable seismic performance. Therefore, a component of the seismic evaluation will be to determine how susceptible these WSMF systems are to fracture and the potential for undesirable seismic performance in a strong earthquake.

### *2.1.6 Site Characterization*

The primary geologic and seismic hazard affecting the Natural Resources Building is moderate to severe shaking in response to local moderate or more distant large-magnitude earthquakes. The seismic hazard due to ground shaking has been characterized on a probabilistic basis by general ASCE 41-06 acceleration response spectra. A site-specific ground motion study was not performed at this time. Shown below in Table 2-2 are the short-period ($S_s$) and long-period ($S_1$) response acceleration parameters for the assumed Soil Site Class D. The MCE corresponds to a 2%/50-year event and the Design-Basis Earthquake (DBE) is equivalent to a 10%/50-year event.

Table 2-2. Site Acceleration Parameters.

| Seismic Design Criteria | MCE $S_s$ (%g) | MCE $S_1$ (%g) | DBE $S_s$ (%g) | DBE $S_1$ (%g) |
|---|---|---|---|---|
| ASCE 41-06 (per USGS) | 117.9 | 43.4 | 77.9 | 23.2 |

A general 5 percent damped horizontal response spectrum was constructed per ASCE 41-06 and used for the seismic evaluation. Figure 2-2 shows a comparison of the ASCE 41-06 general response spectrum, using the acceleration parameters provided by ASCE 41-06 for the BSE-1 event. Note the figure also includes a curve for 75 percent of the general response spectrum, the minimum acceleration permitted for a Tier 3 Evaluation by ASCE 31-03. Acceleration values given by the evaluation level response spectra are higher in comparison with the general spectral values. These evaluation level values are further adjusted based on the C-factors recommended by ASCE 41-06 and used in the analysis of the building. Figure 2-3 shows a similar comparison for the BSE-2 event.

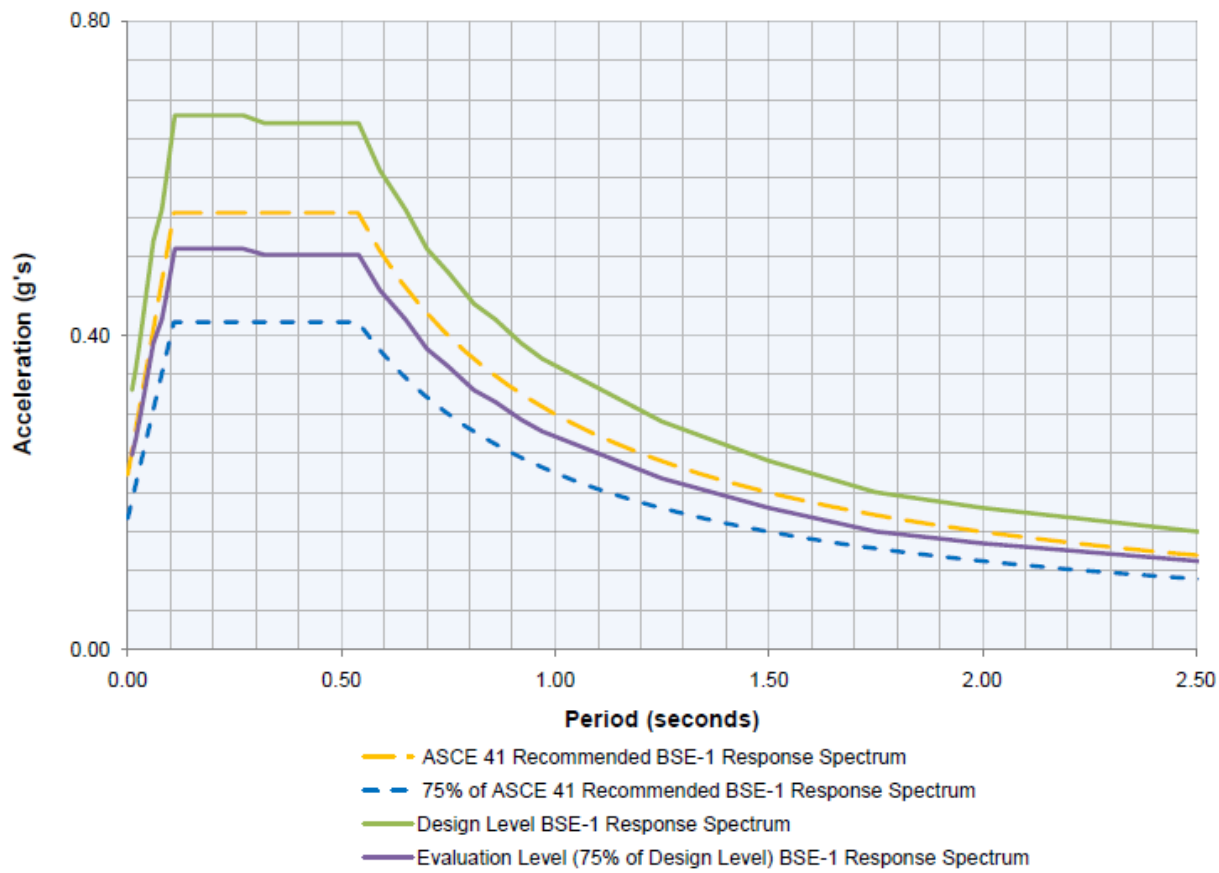



Figure 2-2. BSE-1 (10%/50-year) Acceleration Response Spectra Comparison.

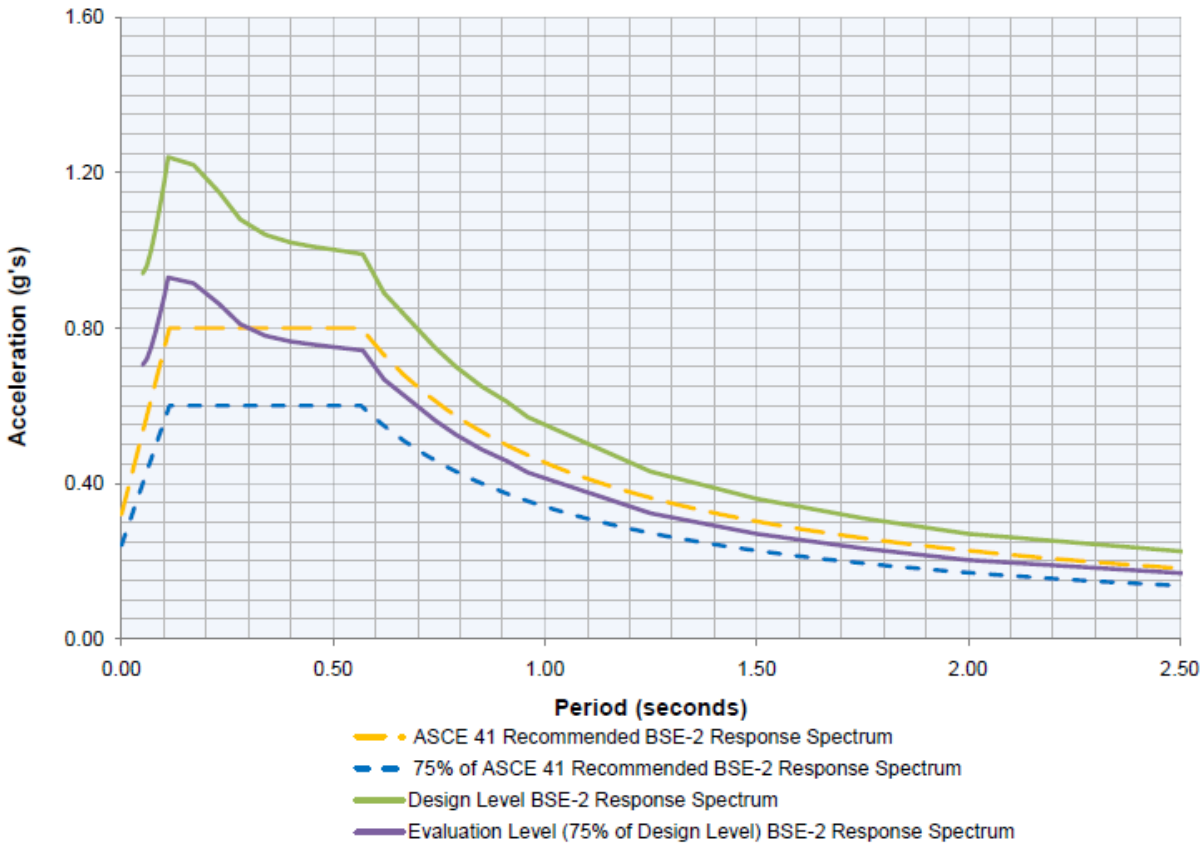



Figure 2-3. BSE-2 (2%/50-year) Acceleration Response Spectra Comparison.

### *2.1.7 Study of Building Response to Satsop and Nisqually Earthquakes*

The Natural Resources Building is currently equipped with multiple sensors that record building motion during seismic events. After multiple seismic events and research on the recorded outputs from the sensors, a paper was published that discusses possible changes in the seismic response of the Natural Resources Building. The paper is located in Appendix C of this report.

The paper, "Transient and Long-term Changes in Seismic Response of the Natural Resources Building, Olympia, WA Due to Earthquake Shaking," discusses the possibility of a reduction in building stiffness based on analysis of recordings from ambient building motions and seismic recordings of the building during the 1999 Satsop Earthquake, the Nisqually Earthquake, and the 2001 Satsop Earthquake. The paper argues that structural issues, such as both long and short-term damage and long lasting changes in building stiffness may have occurred during past seismic events that were registered by the building sensors. The report concludes long-term changes have occurred as a result of previous seismic events, and it is possible the building stiffness may have permanently dropped 12 percent.

The possible loss in stiffness can be analyzed by comparing the output from the three-dimensional finite-element building model to building motions recorded during the 1999 Satsop Earthquake, the Nisqually Earthquake, and the 2001 Satsop Earthquake. Earthquake time history records for these three earthquakes can be obtained from the U.S. Geology Survey (USGS), which digitally records and documents seismic events. Corrected USGS records, which are

original records that have been modified to account for irregularities such as instrument errors and noise contamination, should be used in the finite model. The 2001 Nisqually Earthquake corrected records were used in the finite model to determine the distribution of expected seismic forces and displacements during the event. The expected values were then compared to the recorded outputs from sensors installed in the building. To get the analytical model to resemble the building response obtained from the sensors located in the building, various adjustments and iterations of the model should be made. The comparison of model output and sensor output allow the Natural Resources Building's structural integrity and possible loss of stiffness to be evaluated.

A discussion of the results of the evaluation compared to building sensor outputs and the earthquake records is discussed in the Seismic Evaluation section of this report.

## 2.2 Analysis Procedures

### *2.2.1 Linear Dynamic Procedure*

Linear Dynamic Procedure (LDP) was used to analyze and seismically evaluate the Natural Resources Building. Since the LDP characterizes dynamic response directly, it often provides greater insight into the expected structural response of the building to seismic ground motion than provided by the Equivalent Lateral Force (ELF) Procedure. However, as with all linear analysis methods, the LDP does not explicitly account for the effects of nonlinear response that the structure will experience when structural members crack or yield. A prescriptive acceleration response spectrum, constructed in accordance with the requirements of ASCE 41-06, was used to implement this procedure.

Dynamic loads were applied to the structure in orthogonal pairs at an eccentricity of five percent. Building responses in the principal directions were then combined using the Square Root of the Sum of Squares (SRSS) methodology to account for variability in the direction of the ground motions.

### *2.2.2 Procedures for Structural Evaluation*

The following is a summary of the procedures used to perform the structural evaluation. The Natural resources Building was evaluated in accordance with the requirements of ASCE 41-06.

1. Reviewed record drawings to determine the extent and configuration of the structural system.
2. Field-examined existing conditions. Conducted a walk-through to obtain photographic documentation of building framing, finishes, and layout. Checked record drawings for accuracy and completeness.
3. Performed an ASCE 31 Tier 3 Detailed Structural Evaluation. ASCE 41-06 was used as the evaluation standard.

a. Created three-dimensional finite-element building model of the building. Record drawings were used as the basis for model layout, geometry, material properties, member definitions, gravity loads, etc.

b. Analyzed building using LDP. Model analysis was performed using prescriptive ASCE 41-06 acceleration response spectra for the BSE-1 and BSE-2 events. Analysis was completed in accordance with the provisions of ASCE 41-06.

c. Calculated building drifts and DCRs for lateral force-resisting system elements in accordance with the provisions of ASCE 41-06.

d. Checked global stability of structure under seismic loading.

*Demand-Capacity Ratio*

The basic tool for evaluation of elements of the LFRS is the Demand-to-Capacity Ratio (DCR), a ratio of the force in the member (demand) to the strength of the member (capacity). A DCR of less than one signifies a member that is not loaded to its maximum capacity. At a DCR of one, the demand and capacity are equal. With a DCR greater than one, the member is overloaded. Components with higher DCRs will generally experience greater damage and have a higher potential of failure. The DCR color coding system shown in Figure 1.4-3 is used to characterize visually the levels of potential overload for the structural components in each building.

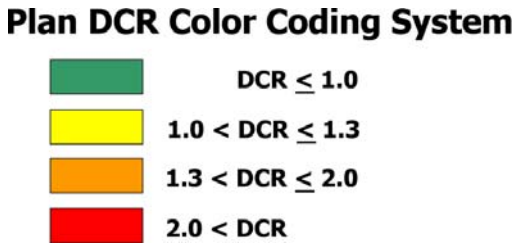



Figure 2-4. Demand-Capacity Ratio Color Coding System.

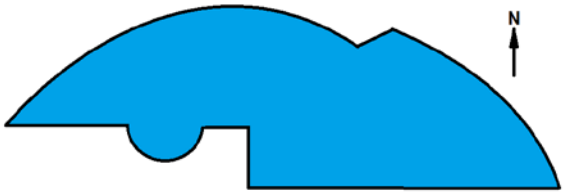
Members shown in green on the DCR key plans indicate elements that have adequate strength/ductility for the applied seismic loads and deformations. Members shown in yellow are marginally inadequate (DCR between 1.0 and 1.3). If all the members in a building were green or yellow, seismic rehabilitation of the building would typically not be required. Orange and red members indicate areas with greater seismic deficiencies. These members will likely experience significant damage and have a higher potential of failure under the design seismic loads.

# 3.0 Seismic Evaluation

## 3.1 Natural Resources Building Evaluation Summary

### 3.1.1 Building Description

| | |
|---|---|
| Year Built: | 1993 |
| Number of Stories: | 6 |
| Gross Square Footage: | 549,500 SF |




Key Plan

The Natural Resources Building is a six-story facility with a gravity system consisting of precast hollow core planks with concrete topping at the three sub-grade parking levels and concrete-on-metal deck at the six levels above. The gravity systems are supported by steel beams, steel columns, and concrete shear walls on a pile foundation system. The top two stories on the east side of the building cantilever beyond the stories below and are supported by steel trusses on concrete columns; three partial sub-grade levels are used for parking. The elongated building has a Lateral Force Resisting System (LFRS) that consists of full height concrete shear walls that extend from the basement level to the roof. The LFRS also contains steel moment frames located at the open circular bay, known as the rotunda, and at the end bays of the curved walls on the first and fifth floors. Figure 3.1 through Figure 3.4 show exterior views of the Natural Resources Building.

Figure 3-1. Cantilevered Top Two Stories, East Exterior.

Figure 3-2. Northwest Exterior.

Figure 3-3. South Exterior.

Figure 3-4. Northeast Exterior and Parking Deck.

#### *3.1.1.1 Building Use*

The Natural Resources Building currently houses multiple divisions of the state of Washington's Department of Natural Resources. The building is approximately 549,500-square-feet in size and has three levels of parking available to both building occupants and visitors. The remaining floors of the building consist primarily of office space with a cafeteria and lobby at the first floor.

#### *3.1.1.2 Structural System*

The primary lateral force-resisting systems (LFRS) in the Natural Resources Building are concrete shear walls and welded steel moment frames (WSMF). The two partial sub-grade levels are composed of concrete walls and a precast hollow core plank diaphragm. Concrete shear walls are present at the east, center, and west cores of the building and extend from the basement to the roof at each location. Welded steel moment frames are located at the open, circular bay, known as the rotunda, and also at the end bays of the curved walls at the first and fifth levels. The foundations consist of concrete pile caps supported by concrete filled steel pipe piles beneath the columns and concrete walls. Building elevations are shown in Figures 3.5 to 3.8.

Table 3-1. Structural System Description of Natural Resources Building.

| Structural System | Description |
|---|---|
| Roof | Wide-flange steel girders and beams support a metal roof deck. Steel columns and concrete shear walls provide gravity support for the roof framing. |
| Floor | Wide-flange steel girders and beams support composite concrete-on-metal deck on all floors except the parking levels. Steel columns and concrete shear walls provide gravity support for the floor framing at the composite deck locations. Precast hollow core planks with concrete topping support the two parking levels. Steel columns encased in concrete and concrete shear walls provide gravity support for the parking levels. |
| Foundations | Steel columns and concrete shear walls are supported by piles. A 6-inch slab on grade is used at the sub-grade parking level to support floor loads. Retaining walls are present on the north and west lower level walls. |
| Lateral | WSMF are present at the rotunda, at the second and fifth floors, to resists lateral loads. Moment frames are also present at both ends of the curved wall. Concrete shear wall cores are located at the east, center, and west ends of the building. Concrete shear walls are also in the north/south and east/west directions and extend to the roof at each location. |

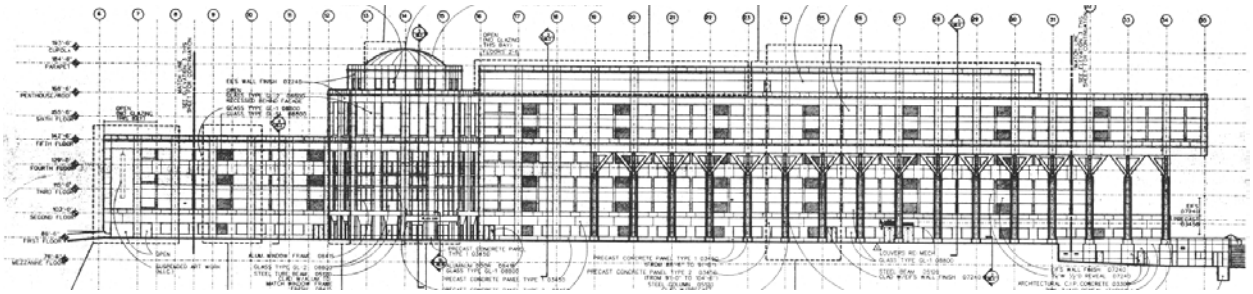


Figure 3-5. Record Drawings, South Elevation.[1]

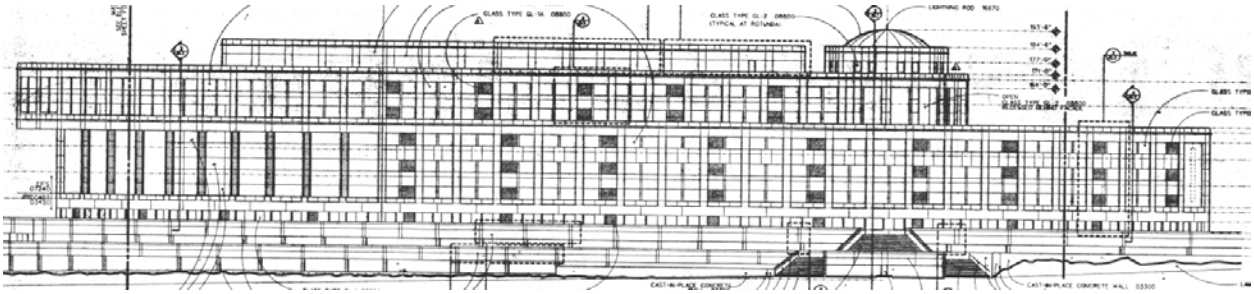


Figure 3-6. Record Drawings, North Elevation.[1]

[1]C.W.Fentress J.H.Bradburn and Associates, P.C. Architecture, Interiors, Planning, drawings titled Natural Resources Building, Project #90-003B, Olympia Washington, 1990.

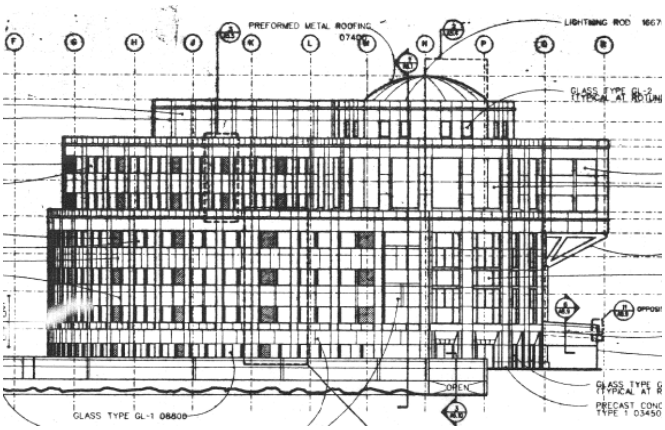



Figure 3-7. Record Drawings, West Elevation.[1]

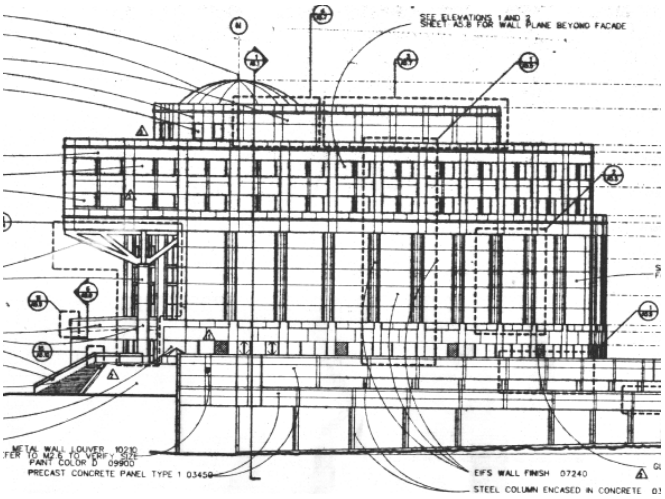



Figure 3-8. Record Drawings, East Elevation.[1]

### 3.1.2 Seismic Evaluation Findings

Table 3.3, Table 3.4, and Table 3.5 summarize the building's expected seismic performance for both a 10%/50-year event and a 2%/50-year event based on output from the linear dynamic model.

The building's expected seismic performance for both a 10%/50-year event and a 2%/50-year event based on output from the linear dynamic model are presented in the following tables. Isometric views of the three-dimensional finite-element model used to analyze the Natural Resources Building are shown in Figures 3.9 to 3.12. Modal accelerations were calculated using the ASCE 41-06 response spectrum.

[1] C.W.Fentress J.H.Bradburn and Associates, P.C. Architecture, Interiors, Planning, drawings titled Natural Resources Building, Project #90-003B, Olympia Washington, 1990.

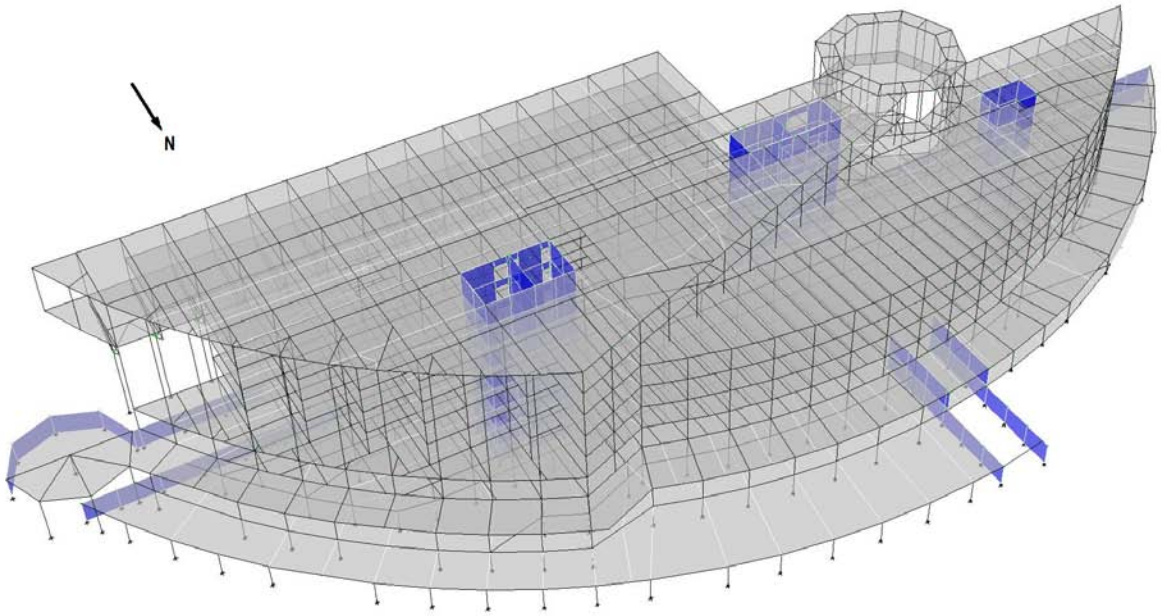



Figure 3-9. Northeast Isometric View of the Natural Resources Building from 3D Finite Element Model.

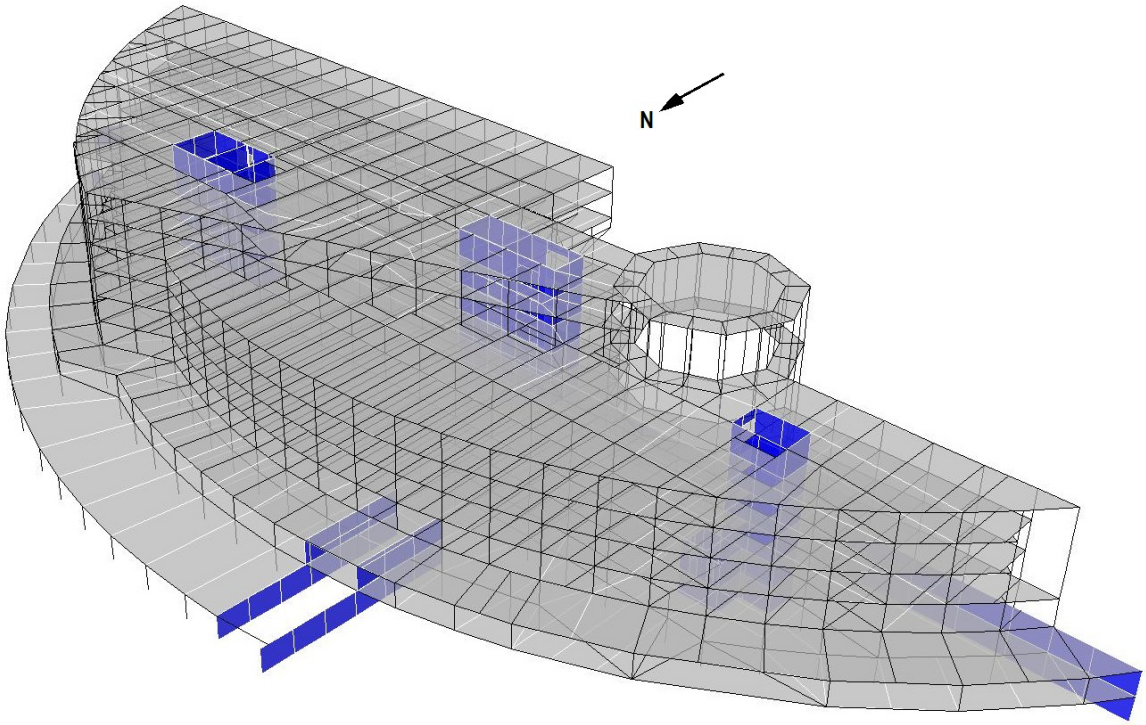



Figure 3-10. Northwest Isometric View of the Natural Resources Building from 3D Finite Element Model.

The approximate fundamental building periods and the associated principle directions are summarized in Table 3.2.

Table 3-2. Modal Analysis Period Results.

| Principal Direction | Period |
|---|---|
| Transverse (North-South) | 0.44 seconds |
| Longitudinal (East-West) | 0.35 seconds |
| Torsion | 0.07 seconds |

The calculated building displacements, inter-story drift ratios, and performance targets for the Life Safety Performance Level of each floor are summarized in Table 3.3. The values are based on output from the linear dynamic model. Table 3.4 displays the same information for the Collapse Prevention Performance Level. The drifts for Life Safety (LS) and Collapse Prevention (CP) Performance Objectives exceed the target values recommended by ASCE 41-06. The target drift ratios indicated in ASCE 41-06 and shown in the tables below are recommendations based on components of the lateral system. The "NO" indicated that the maximum drift ratios exceed the target Drift Ratio, meaning the building is likely overstressed on those levels.

Table 3-3. Building Displacements, 10/50 Event (BSE-1).

| Level | Max Transverse Displacement | Max Transverse Drift Ratio | Max Longitudinal Displacement | Max Longitudinal Drift Ratio | Target Drift Ratio *LS* | Meets Target Drift Ratio |
|---|---|---|---|---|---|---|
| Roof | 7.0 in. | 1.2% | 4.1 in. | 0.7% | 1% | NO |
| 6th Floor | 5.1 in. | 1.3% | 1.6 in. | 0.6% | 1% | NO |
| 5th Floor | 4.1 in. | 0.7% | 1.4 in. | 0.3% | 1% | YES |
| 4th Floor | 4.9 in. | 0.9% | 1.4 in. | 0.3% | 1% | YES |
| 3rd Floor | 4.8 in. | 1.2% | 1.6 in. | 0.4% | 1% | NO |
| 2nd Floor | 2.9 in. | 1.4% | 0.9 in. | 0.5% | 1% | NO |
| 1st Floor | 4.1 in. | 2.8% | 1.4 in. | 1.0% | 1% | NO |
| P-1 Floor | 2.7 in. | 1.9% | 0.3 in. | 0.2% | 1% | NO |

Table 3-4. Building Displacements, 2/50 Event (BSE-2).

| Level | Max Transverse Displacement | Max Transverse Drift Ratio | Max Longitudinal Displacement | Max Longitudinal Drift Ratio | Target Drift Ratio *CP* | Meets Target Drift Ratio |
|---|---|---|---|---|---|---|
| Roof | 10.8 in. | 1.9% | 6.3 in. | 1.1% | 2% | YES |
| 6th Floor | 7.9 in. | 2.0% | 2.5 in. | 1.0% | 2% | YES |
| 5th Floor | 9.4 in. | 1.1% | 2.8 in. | 0.5% | 2% | YES |
| 4th Floor | 7.7 in. | 1.3% | 2.2 in. | 0.4% | 2% | YES |
| 3rd Floor | 7.4 in. | 1.9% | 2.4 in. | 0.7% | 2% | YES |
| 2nd Floor | 4.5 in. | 2.2% | 1.4 in. | 0.8% | 2% | NO |
| 1st Floor | 6.3 in. | 4.4% | 2.2 in. | 1.5% | 2% | NO |
| P-1 Floor | 4.1 in. | 2.8% | 0.5 in. | 0.4% | 2% | NO |

### *3.1.2.1 Earthquake Record Correlation*

Earthquake time history records from the 2001 Nisqually earthquake, obtained from the U.S. Geological Survey (USGS) were used in the finite model to determine the distribution of expected seismic forces and displacements during the event. The expected values were then compared to the recorded outputs from sensors installed in the building. To get the analytical model to resemble the building response obtained from the sensors, various adjustments and iterations of the model were made. Unfortunately, both the measured period and the magnitude of the accelerations and displacements could not be marginally aligned at this time. Due to the complexity of the building and assumptions made for this evaluation at the time, it would require more effort to obtain this level of accuracy in the analytical model. However, this does not affect the results of the evaluation in accordance with the code as discussed in the Evaluation Criteria section.

Figures 3.10 to 3.12 show a comparison of the building accelerations and displacements at the fifth floor. The two traces compare the response of the finite (Etabs) model with the 2001 Nisqually earthquake record and the actual recorded accelerations and displacements from sensors located at the fifth floor of the building.

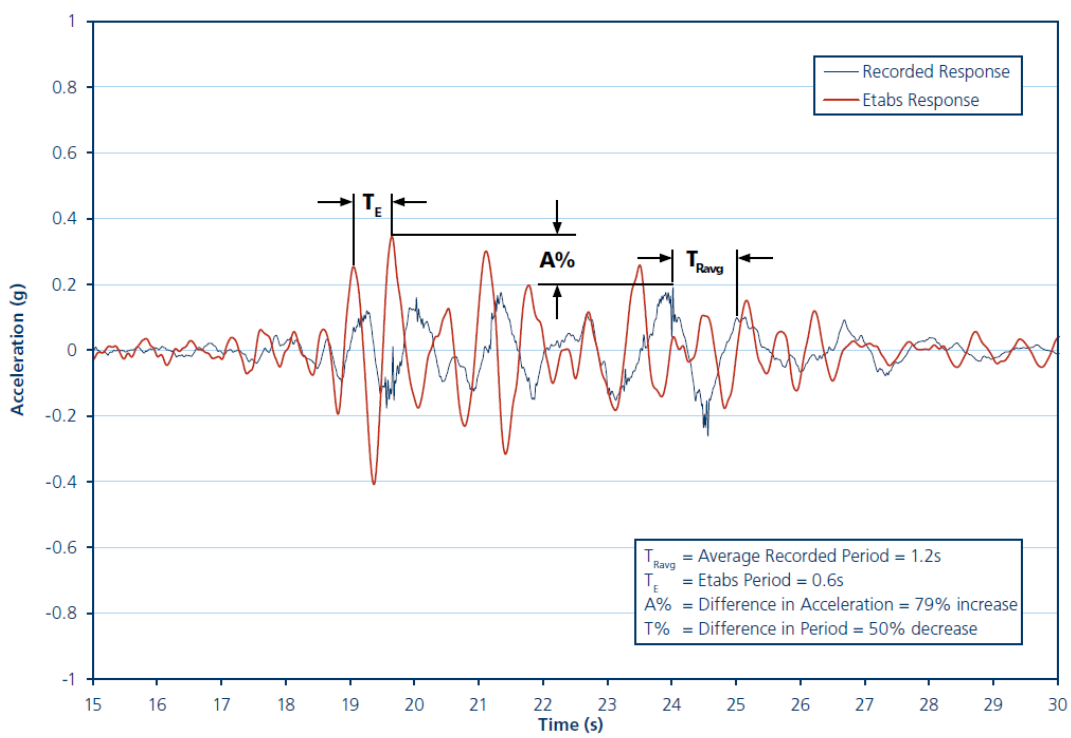



Figure 3-11. 5th Floor Acceleration Comparison in the Transverse Direction.

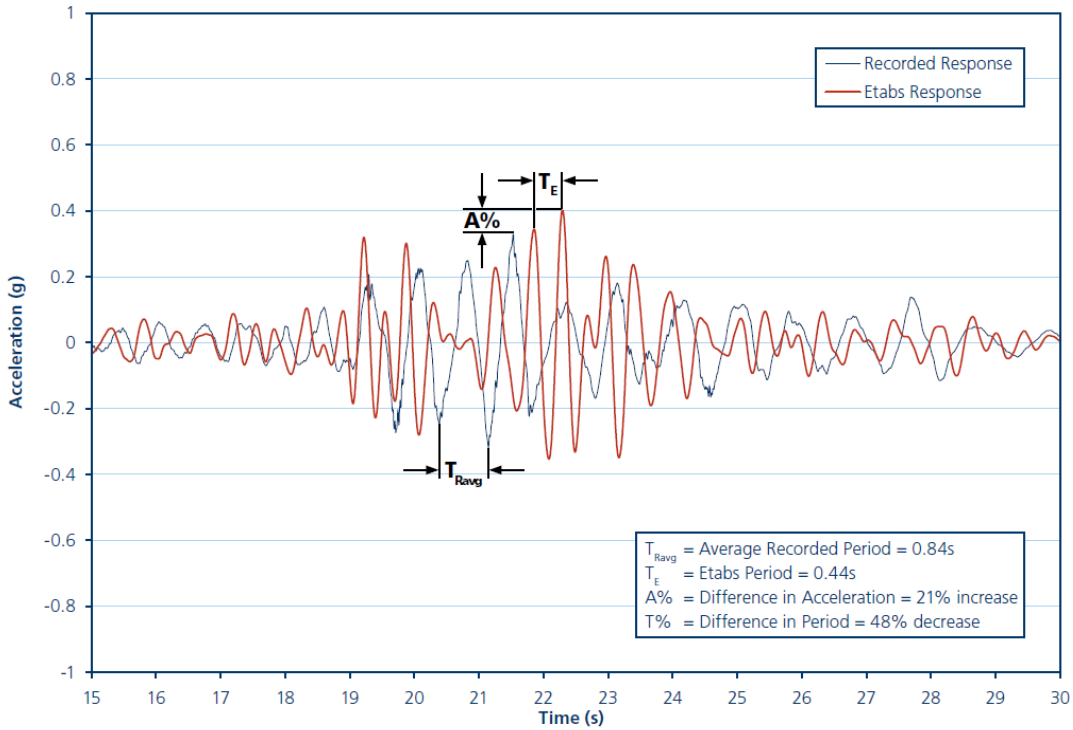



Figure 3-12. 5th Floor Acceleration Comparison in the Longitudinal Direction.

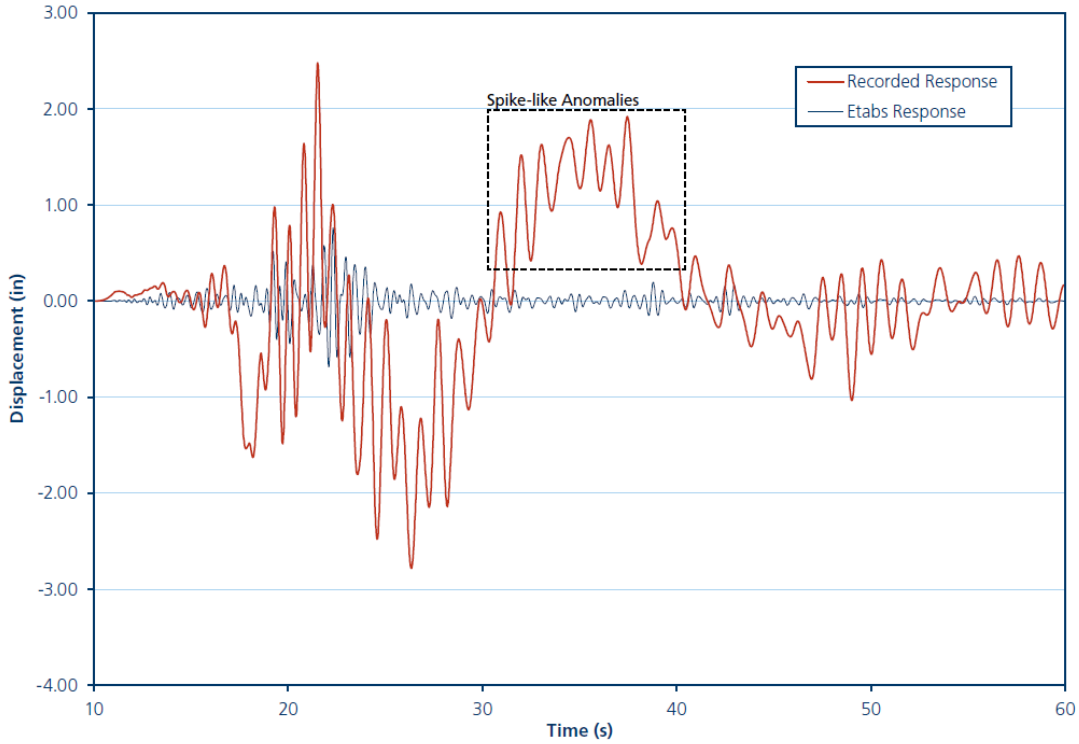



Figure 3-13. 5th Floor Displacement Comparison in the Longitudinal Direction.

When comparing acceleration output from the analytical model and the sensor measurement, ideally the two lines would lay exactly over each other. However, the traces show lower accelerations for the recorded earthquake than for what was modeled. After reviewing the displacement corrected records obtained from the USGS, there appears to be data that make comparison difficult. As seen in Figure 3-12 above, there are many small spike-like anomalies before curves go through an entire cycle. This type of curve resembles vibration or some disruption at the sensor, where the sensor may have been dislodged or was not secured in place during the event. The obtained record is questionable and was only used as the baseline threshold to validate output obtained from the analytical model, which provides a more conservative result.

Due to the uncertainty of the recorded earthquake data, it is difficult to prove that there could be a possible loss of stiffness over the years based solely on the measured output. In a subsequent phase of this project, investigation of some existing members for loss of section due to deterioration will be performed. Information obtained from the investigation, along with the recorded output could indicate a potential loss in building stiffness. However, the results will not change the recommendations discussed in Section 5.0.

#### *3.1.2.2 Seismic Deficiencies*

Multiple deficiencies were found during the Tier 2 evaluation. The deficiencies can be found in Appendix A, ASCE 31-03 Checklists. The Tier 2 deficiencies were evaluated further during the Tier 3 evaluation. The deficiencies that remained after the Tier 3 analysis are listed in Table 3.6.

Table 3-5. Identified Tier 3 Seismic Deficiencies.

| Deficiency | Description |
|---|---|
| Concrete Shear Walls | The walls have insufficient in-plane shear strength, inadequate out-of-plane bending capacity, and insufficient flexural capacity at multiple core locations in both the BSE-1 and BSE-2 events. |
| Moment Frames | The beams of east moment frames have insufficient strength for the 2%/50 year event and some moment frame beams in the Rotunda have insufficient strength in both the BSE-1 and BSE-2 events. |
| Moment-Resisting Connections | The steel moment frames consist of welded connections. Welded Steel Moment Frame connections used in the building may exhibit unacceptable seismic performance, as shown by the poor seismic performance of other WSMF buildings during the Northridge earthquake. |
| Lateral System Compatibility | Concrete shear walls are a stiff lateral force-resisting system, while steel moment frames are flexible. Having stiff and flexible lateral systems causes the building diaphragm to distribute more load to the stiffer elements, and the diaphragm must be stronger at the stiff elements than at the flexible elements. |
| Geometry | The horizontal dimensions increase in the project south direction by 42 feet at the top two levels. Horizontal irregularities can lead to fracture at columns and other lateral systems. |
| Drift Check | The drift ratios of the steel moment frames exceed the target ratio for both BSE-1 and BSE-2 as shown in Tables 3.3 and 3.4. |
| Deterioration of Steel | Rusted Steel is visible in the parking levels. |
| Diaphragm | The diaphragm has inadequate shear capacity to resist BSE-1 and BSE-2 diaphragm forces. This is driven by the stiff shear walls dragging more load through the diaphragm than the flexible moment-frames. |

#### *3.1.2.3 Demand-Capacity Ratios*

The Demand-Capacity Ratio (DCR) relates the element's force demand to the capacity (i.e., Demand/Capacity). Elements with DCR values higher than 1.0 are those that are subjected to forces exceeding their capacity. The concrete shear walls and moment frames, which create the lateral force-resisting system of the building, are labeled per Figure 3-13. The maximum DCRs for each element of each system are shown in Figures 3-14 to 3-22.

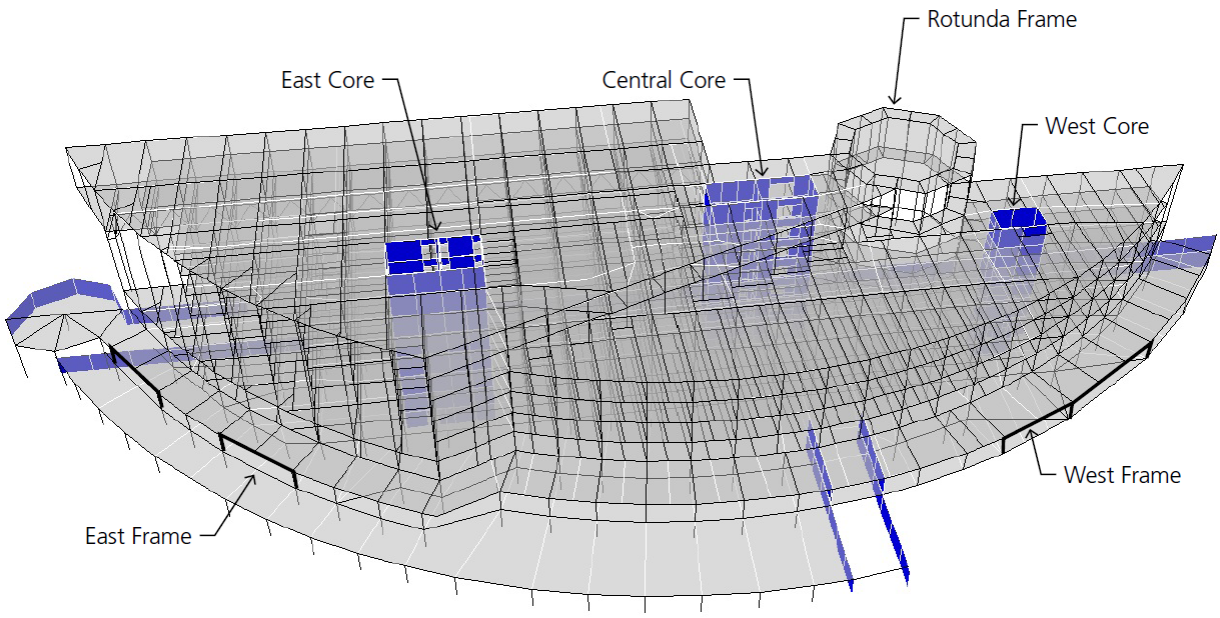



Figure 3-14. Building Lateral Force-Resisting Systems per DCR Labels.

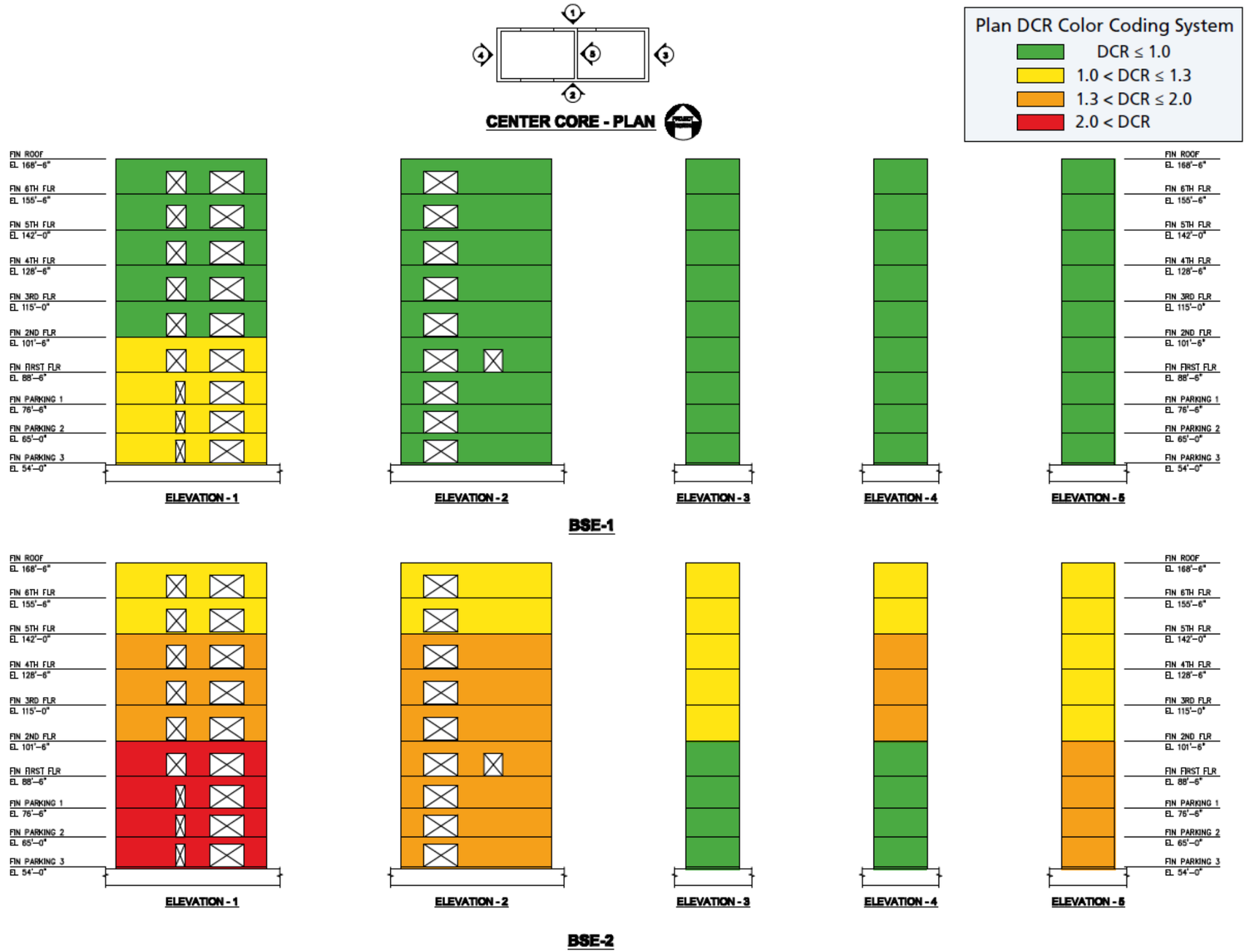



Figure 3-15. Central Core Shear Walls Demand-Capacity Ratios.

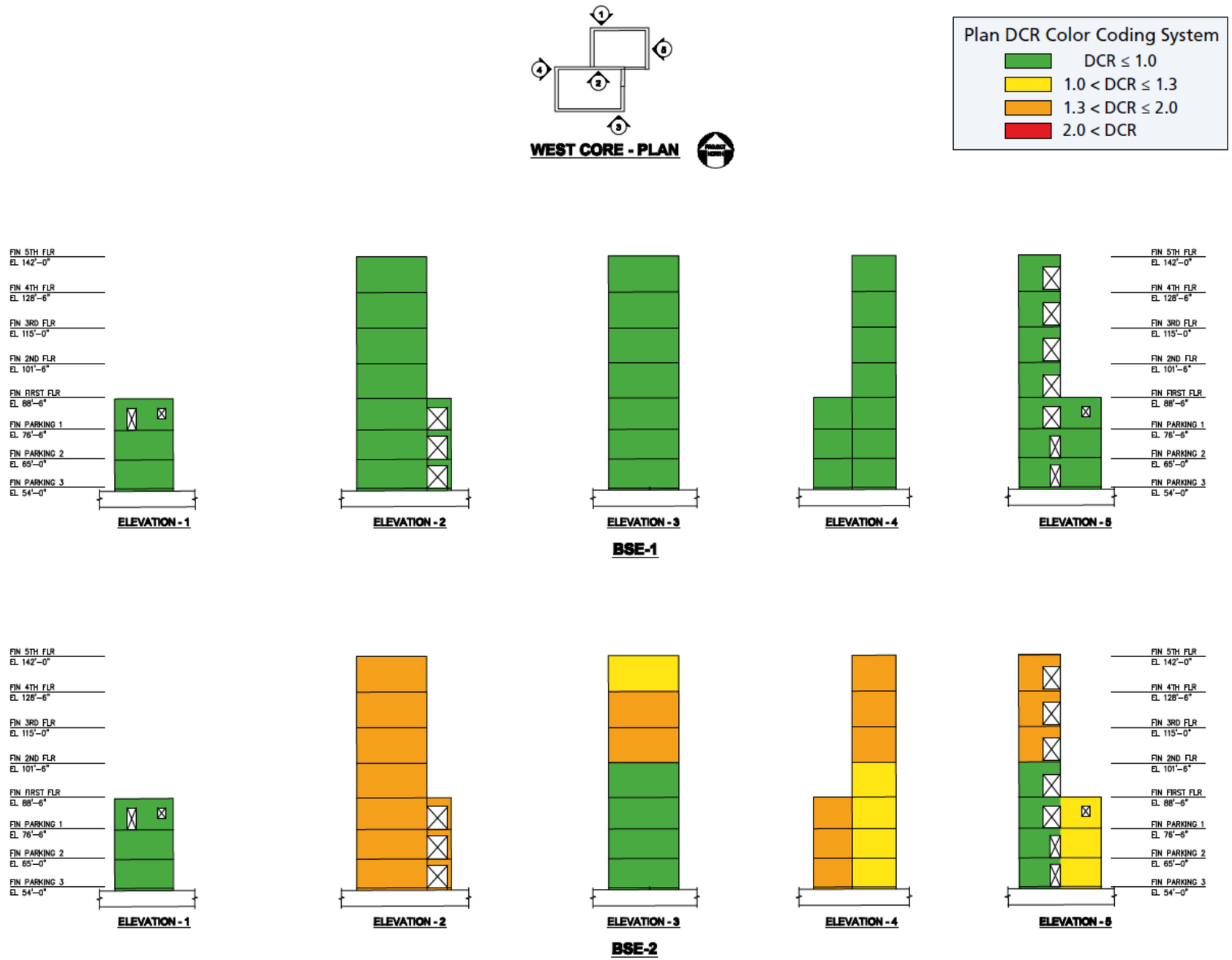



Figure 3-16. West Core Shear Walls Demand-Capacity Ratios.

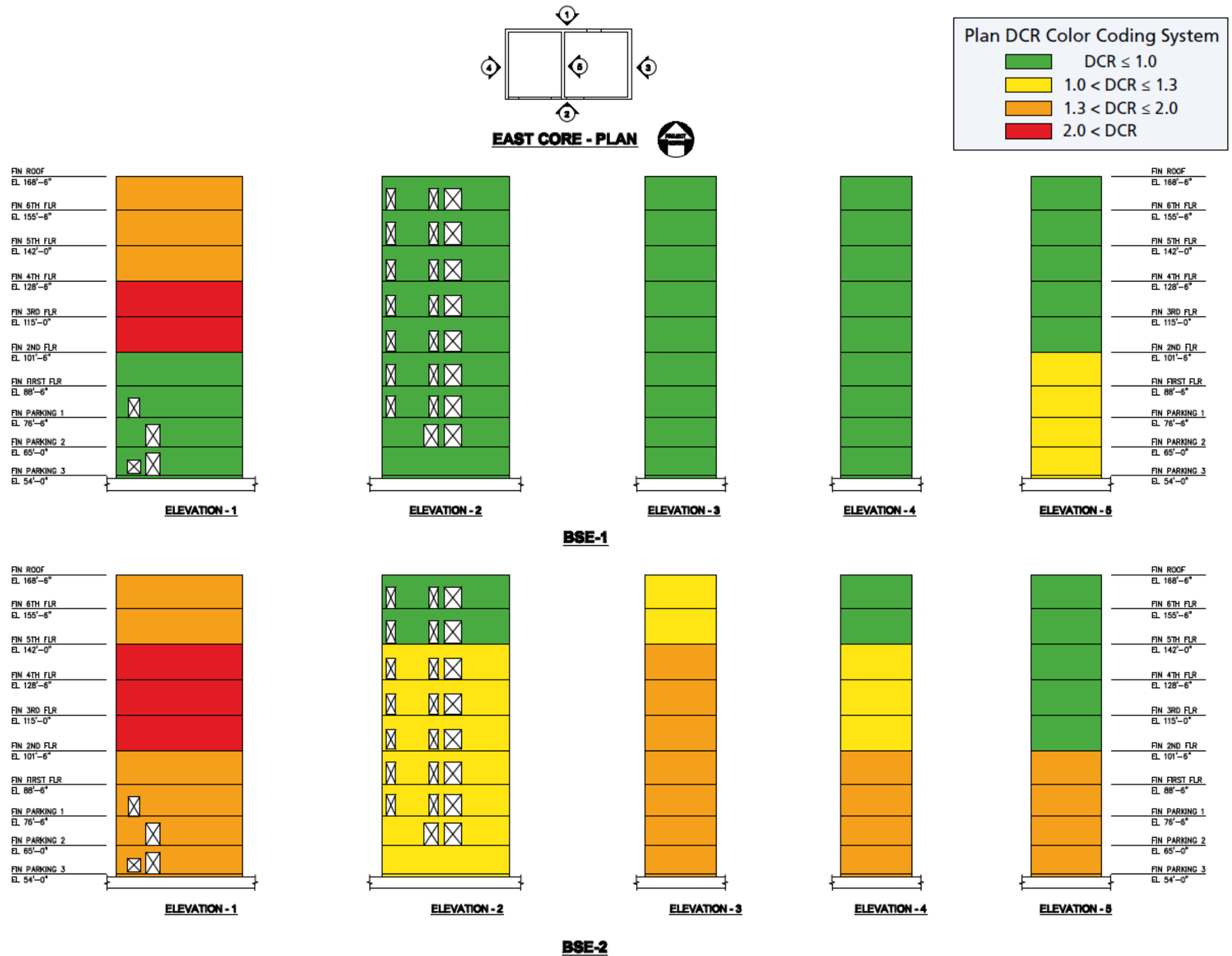



Figure 3-17. East Core Shear Walls Demand-Capacity Ratios.

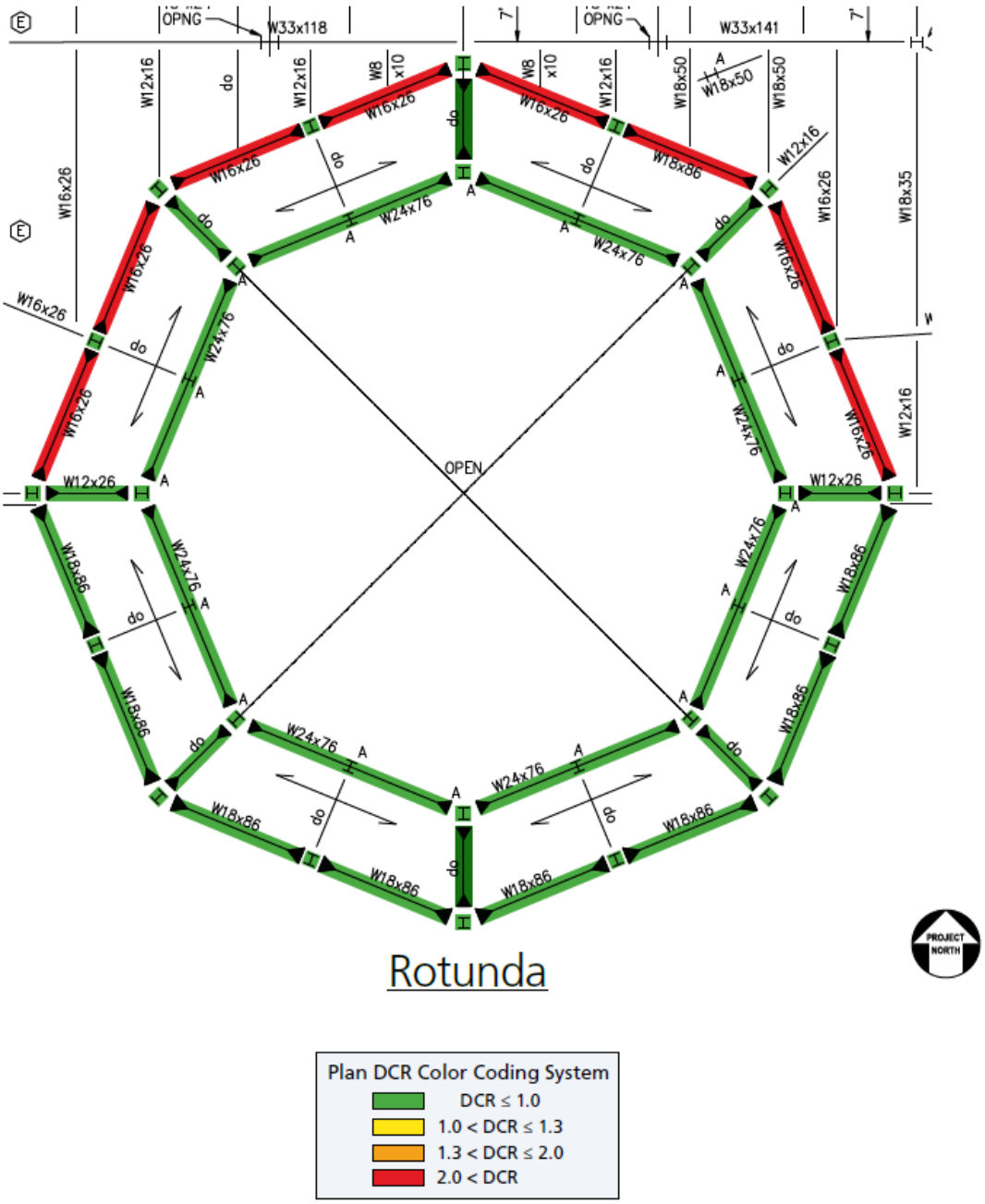



Figure 3-18. Rotunda Moment Frame BSE-1 Maximum Demand-Capacity Ratios.

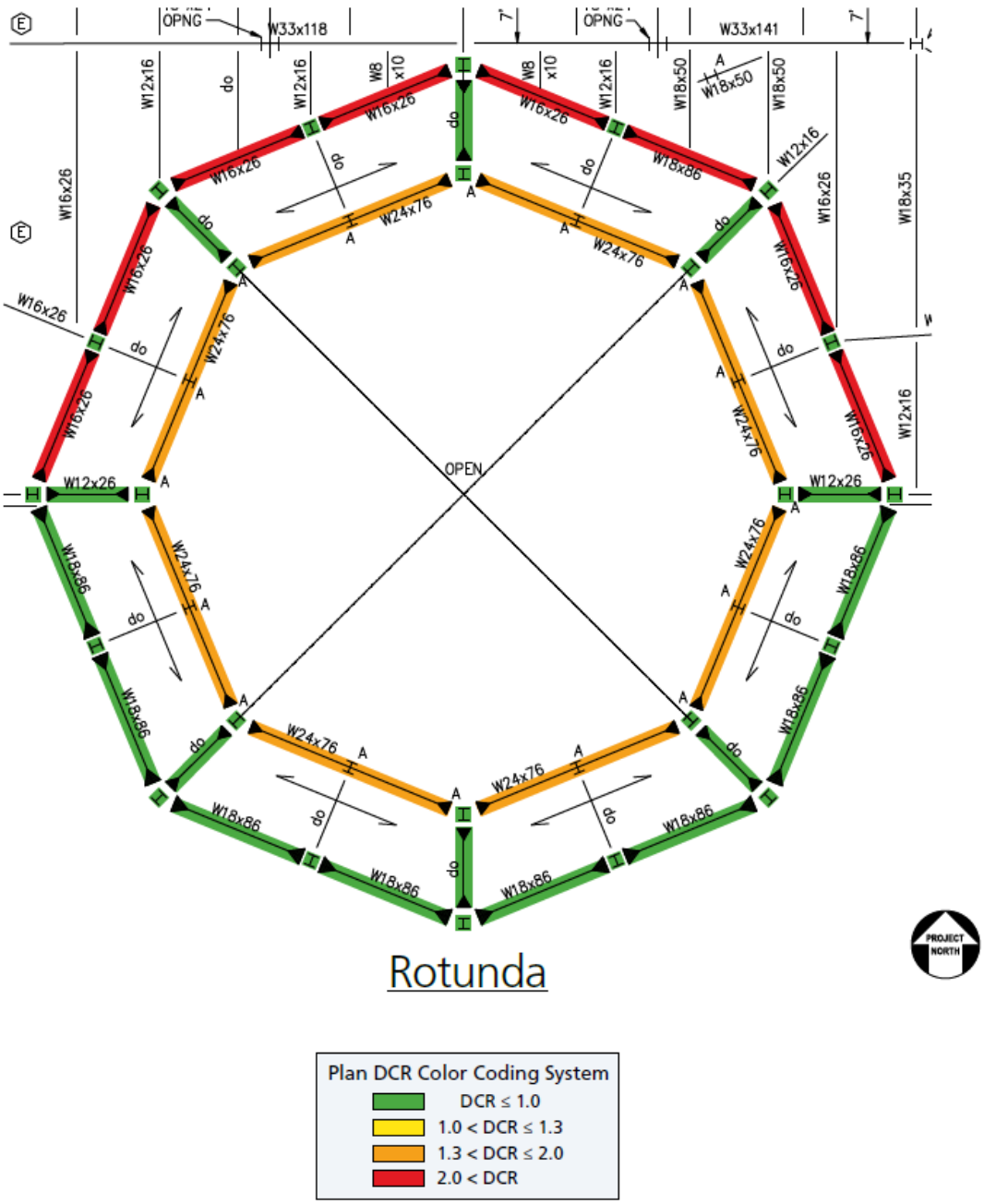



Figure 3-19. Rotunda Moment Frame BSE-2 Maximum Demand-Capacity Ratios.

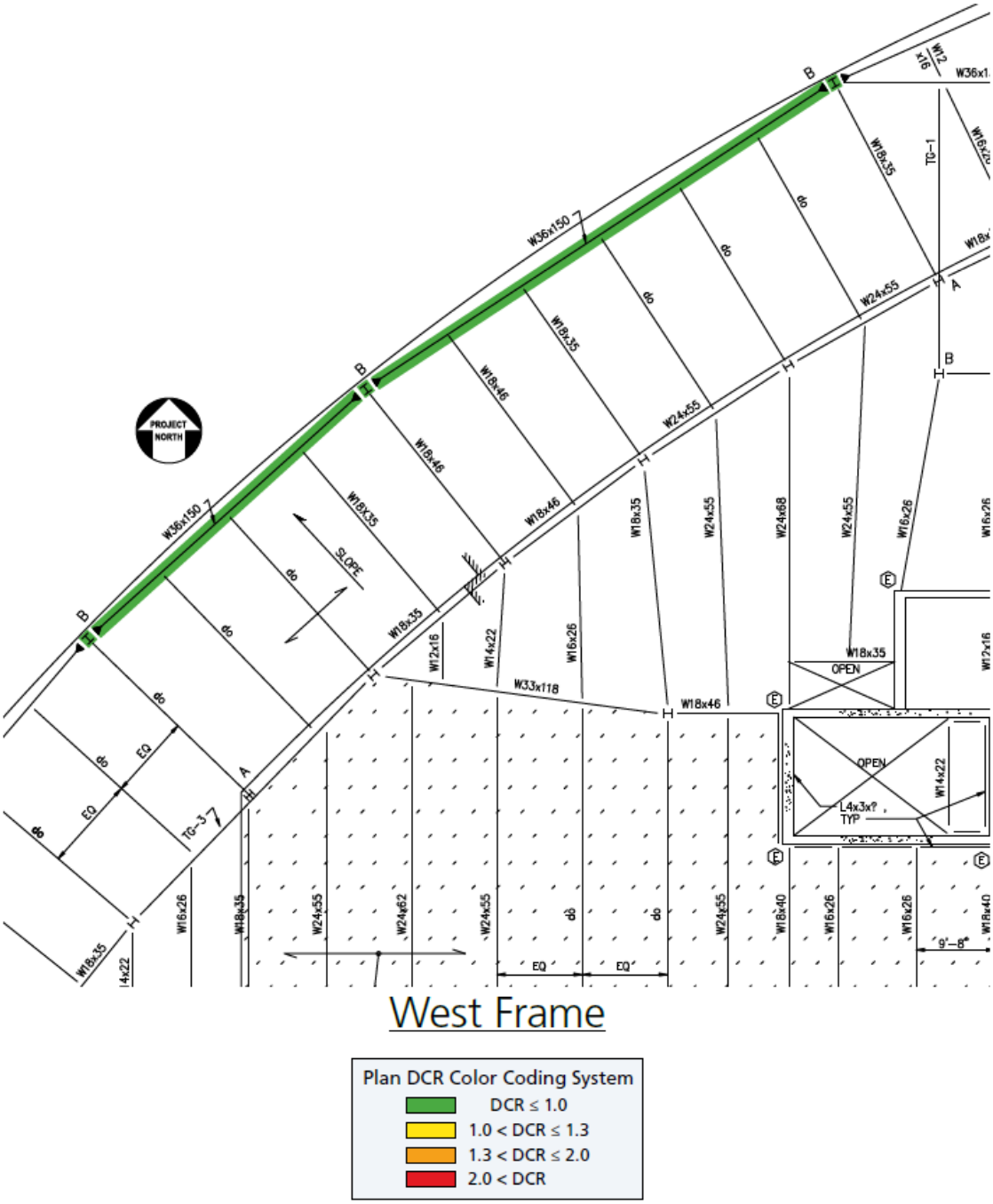
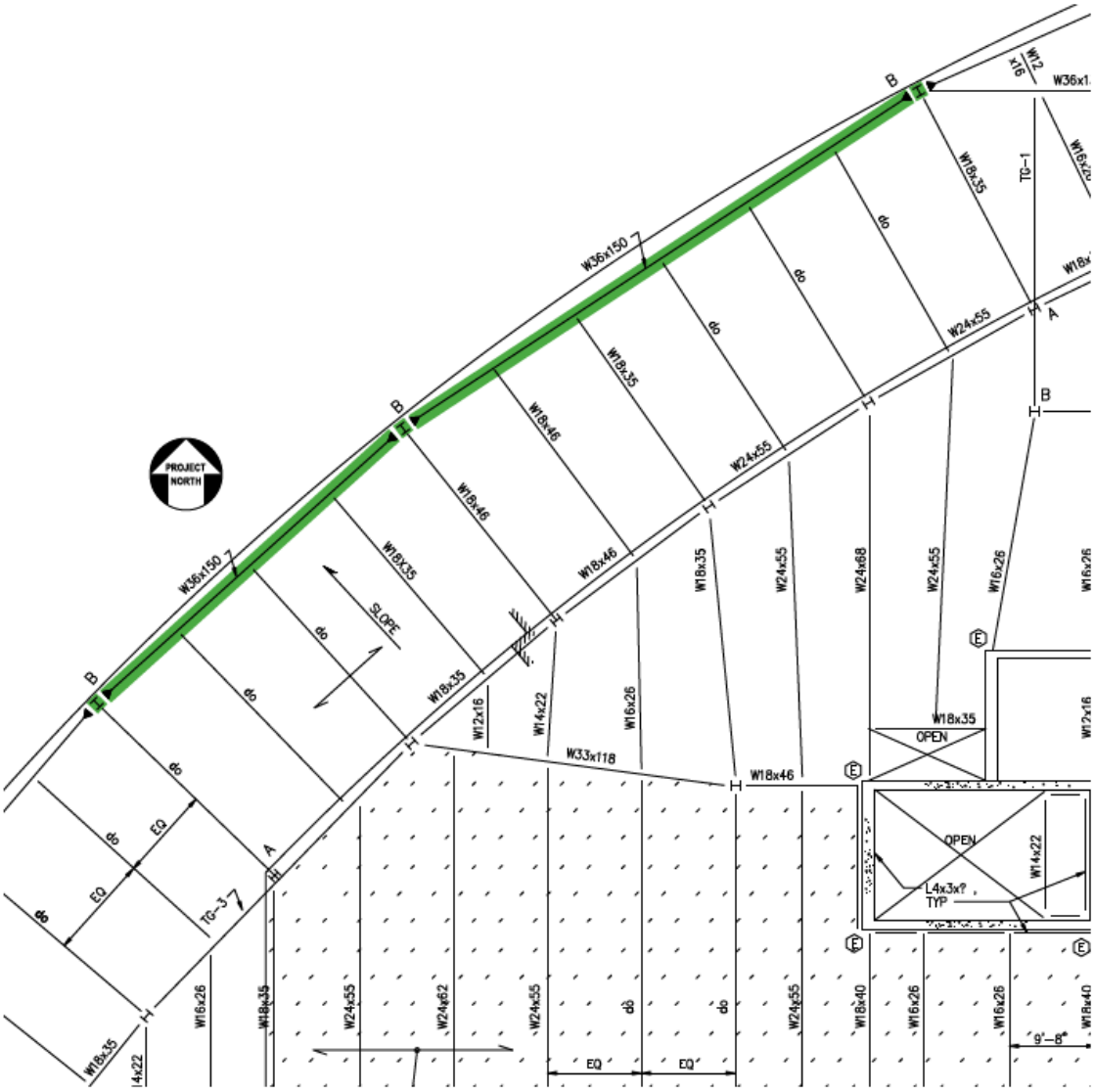



Figure 3-20. West Moment Frame BSE-1 Demand-Capacity Ratios.




West Frame

| Plan DCR Color Coding System |
|---|
| DCR ≤ 1.0 |
| 1.0 < DCR ≤ 1.3 |
| 1.3 < DCR ≤ 2.0 |
| 2.0 < DCR |

Figure 3-21. West Moment Frame BSE-2 Demand-Capacity Ratios.

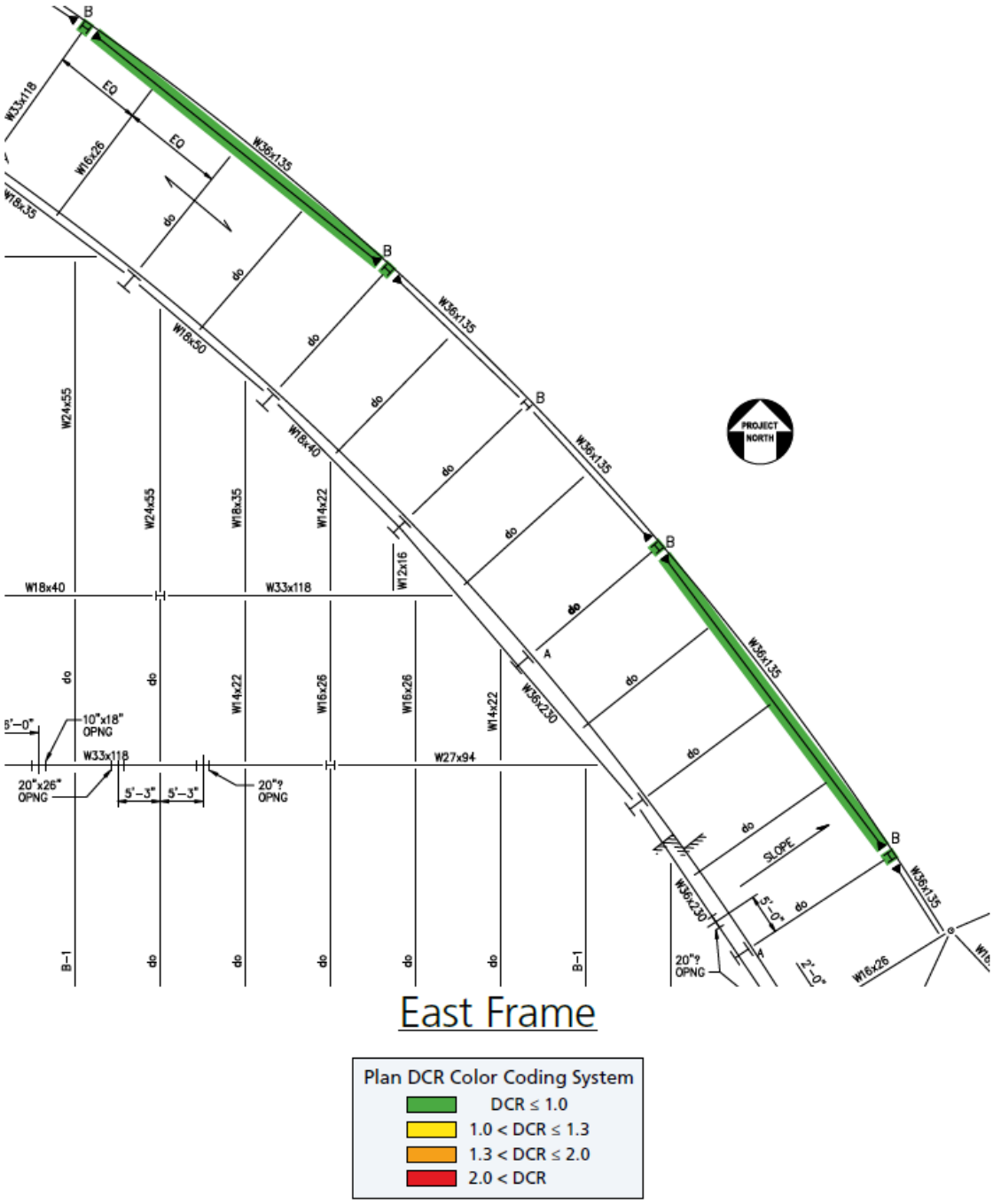



Figure 3-22. East Moment Frame BSE-1 Demand-Capacity Ratios.

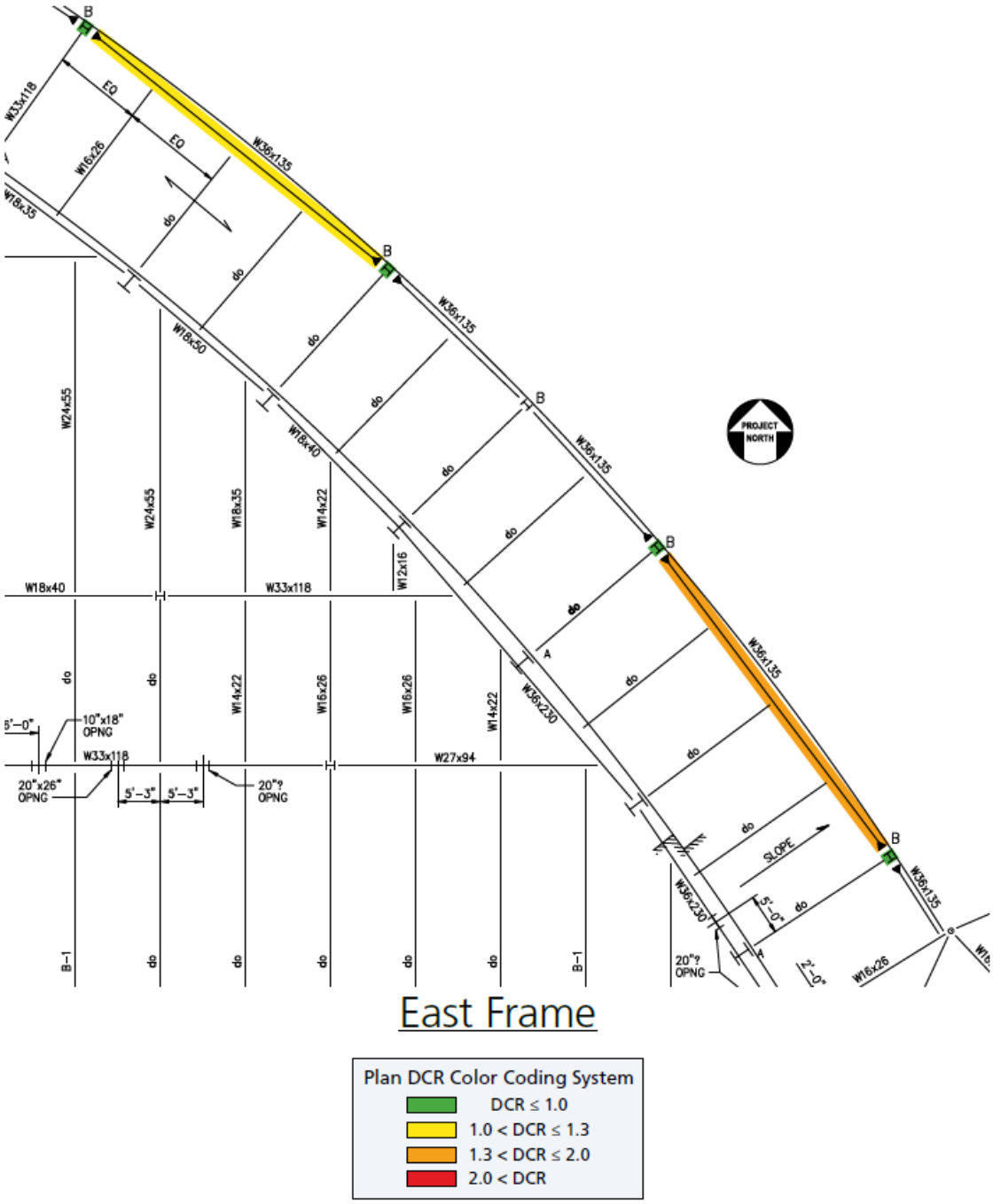



Figure 3-23. East Moment Frame BSE-2 Demand-Capacity Ratios.

# 4.0 Conclusion

## 4.1 Overview

A seismic screening and a structural evaluation were performed for the state of Washington Natural Resources Building (NRB). The evaluation reviewed the expected seismic performance of the NRB to identify potential structural deficiencies that may affect the building's performance during an earthquake. This report documents the results of the structural evaluation that was performed for the Natural Resources Building using ASCE 31-03 and ASCE 41-06 design standards. The building was evaluated with respect to Life Safety (LS) and Collapse Prevention (CP) Performance Levels.

The Natural Resources Building is a 549,500-square-foot facility located in Olympia, Washington. Built in 1993, the state office building houses offices and laboratories for employees of multiple divisions of the state of Washington's Department of Natural Resources. It also provides three levels of parking for employees and visitors.

Evaluation efforts documented in the report included preliminary performance-based seismic evaluations and detail further investigation and comprehensive evaluation of the Natural Resources Building to better understand of seismic program opportunities, goals, and objectives. The following items have been performed and furnished as part of this seismic evaluation project and form the basis for the conclusions and recommendations presented in this report.

1. Reviewed record drawings provided by the state of Washington to determine the extent and configuration of the structural system. Field-examined existing conditions and conducted a walk-through and to examine and obtain photographic documentation of building framing, finishes, and layout.

2. Completed ASCE 31-03 Tier 2 seismic evaluation for the Natural Resources Building to gain an understanding of expected seismic performance. The building was evaluated for the Life Safety Performance Objective.

3. Created a three-dimensional linear dynamic time-history finite-element analysis model to determine the distribution of expected seismic force and displacement from the 1999 and 2001 earthquakes.

4. An ASCE 31-03 Tier 3 detailed seismic evaluation of the Natural Resources Building was completed in accordance with the provisions of ASCE 41-06 component-based procedures.

5. Evaluated the building for Basic Safety Objectives; Collapse Prevention in 2%/50-year earthquake and Life Safety in 10%/50-year earthquake.

The Natural Resources Building was found to have seismic deficiencies. A summary of the detailed seismic evaluation describes the results and explains the extent of structural work that would be required for the buildings to achieve the Life Safety (LS) Performance Objective.

## 4.2 Seismic Evaluations

The following summarizes the results of the detailed seismic evaluation performed on the Natural Resources Building. The evaluation indicates that building upgrades are recommended to achieve the desired levels of seismic performance.

A Tier 3 detailed seismic evaluation was performed in accordance with ASCE 31 for the Natural Resources Building. The ASCE 41-06 standard was used to complete the Tier 3 evaluation. A three-dimensional finite element model used to analyze the structure indicated that the transverse direction inter-story drift at some levels is more than the limit suggested for the Life Safety (LS) and Collapse Prevention (CP) Performance Objectives in ASCE 41.

The lateral system of the NRB consists primarily of concrete shear walls and pre-Northridge type WSMFs. Typical moment-frame beam-column connections of this vintage have a tendency to crack and rupture at force and deformation levels below that for which the connections were typically designed. After detailed analysis of the Natural Resources Building, it was found that select moment-frame beams have limited strength and ductility due to the high forces and displacements for both the 10%/50-year earthquake and the 2%/50-year earthquake. The detailed analysis also showed most shear walls had enough strength in both shear and flexure to resist loads in a 10%/50-year earthquake. However, the shear walls showed unfavorable results in shear strength, with DCRs larger than 1.0, for the 2%/50-year earthquake.

Analysis also indicated that the diaphragm has inadequate shear capacity to resist diaphragm forces that result from the 10%/50-year and the 2%/50-year earthquakes. Having stiff (shear walls) and flexible (moment-frame) lateral systems causes the building diaphragm to unevenly distribute load. More load is dragged to the stiffer elements, and the diaphragm must, therefore, be stronger at the stiff elements than at the flexible elements.

Upgrades are required to reach the desired level of performance for this building. For additional discussion on the seismic evaluation, see Section 3.0 of this report. For additional discussion on recommended upgrades, see Section 5.0, Recommendations.

# 5.0 Recommendations

## 5.1 General

Seismic upgrade is recommended for Washington State's Natural Resources Building, based on the results of the ASCE 31-03 Tier 3 evaluations described in previous sections of this report. The objectives of the proposed upgrades are to:

1. Alleviate specific structural deficiencies identified in the ASCE 31-03 Tier 3 Structural Evaluation.
2. Improve the overall structural reliability and seismic performance level of the building.

Accomplishing these objectives will bring the building to a level of performance consistent with applicable codes and standards.

## 5.2 Potential Seismic Upgrade Strategies for NRB

The following is a discussion of how several seismic upgrade options accomplish the upgrade objectives, thereby reducing the overall NRB seismic risk. Inadequate diaphragm strength and shear transfer to the concrete walls is indentified in the evaluation as a main deficiency. One upgrade strategy is to enhance the existing concrete walls, install a concrete topping slab overlay, and strengthen collectors or install additional collectors. However, this strategy tends to be a disruptive approach.

A second seismic upgrade design strategy that can be used to improve the global strength and stiffness of the building, that is less disruptive, is to use a combination of installing additional steel braced frames and stiffening and strengthening the existing Welded Steel Moment-Frame (WSMF) members and connections. Particular frame bays could be chosen as preferred areas where the installation of braces could reside and minimize disruption.

### 5.2.1 Steel Braced Frames

Adding steel braced frames to an existing building is a common seismic upgrade technique. For a building such as the NRB, the addition of braced frames would be effective in reducing building displacements and inter-story drifts. The detriment, however, is that building stiffness is increased, causing an increase in the seismic base shear. As a result, the frame forces, overturning demands, and foundation uplift have the potential to increase as well. The entire structure – from diaphragm connections to foundations – could require strengthening to support the increase in seismic forces.

Examples of possible configurations and connection are shown in Figures 5-1 and 5-2. These configurations can be used to accommodate doorways, corridors, ducts, showers, and other miscellaneous nonstructural elements.

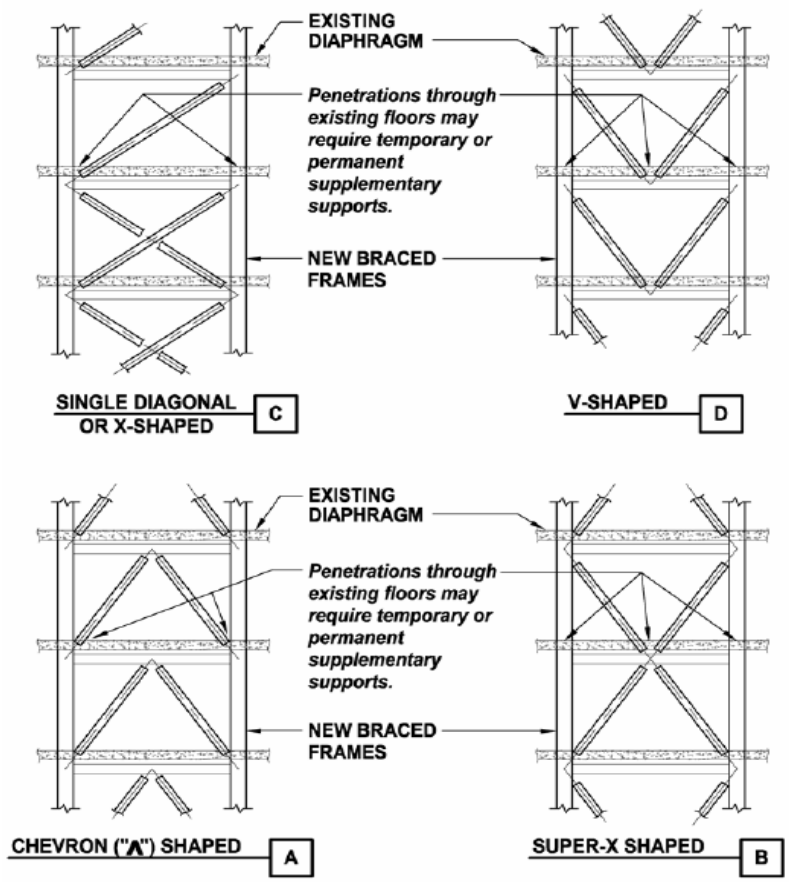



Figure 5-1. Typical Braced Frame Configurations[5].

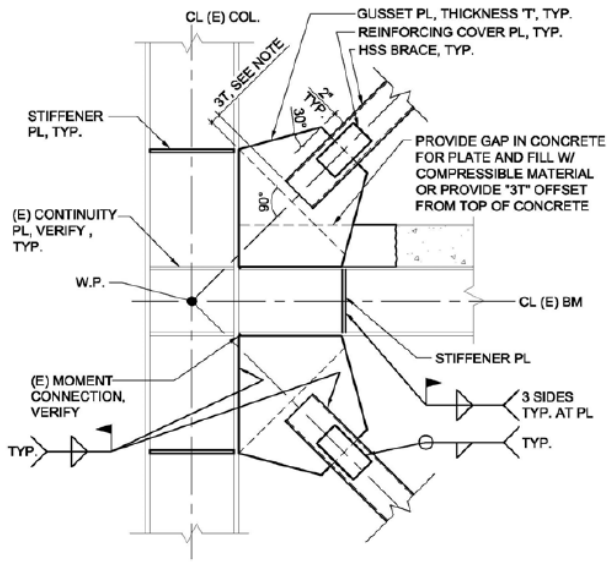



Figure 5-2. HSS Brace to Existing Beam-Column Connection[5].

[5] National Institute of Standards and Technology, "*Techniques for the Seismic Rehabilitation of Existing Buildings*," FEMA-547, 2006 Edition, Interagency Committee on Seismic Safety in Construction Subcommittee 1, Gaithersburg, MD, October 2006.

## *5.2.2* Welded Steel Moment Frame Upgrades

The addition of new lateral elements, as discussed in the previous Steel Braced Frames section, could reduce the demand and displacement on the Welded Steel Moment Frames (WSMFs). Enhancement of the WSMF connections and frame members may not be necessary.

However, if WSMFs are still lacking in both moment-frame connection strength and global stiffness, the upgrades would need to address both deficiencies. One way to improve the reliability of the moment-resisting connections is to weld a tapered haunch to the bottom flange of the beam and enhance the groove weld at the top beam flange. See Figure 5-3 for an example of an upgrade detail. FEMA 547 indicates sufficient connection rotation capacity can be obtained using this method.

Enhancing the moment frame connections improves the strength, ductility, and reliability of the connection but does little to mitigate the global lack of stiffness. For this reason, the stiffness of some moment-frame beams and columns would need to be increased. Typically, this is accomplished by welding a cover plate to the underside of the beam flanges and boxing in columns by welding plates along the flange edges. While this work requires extensive welding, it is not likely that the beam and column upgrades would be as extensive as required for the moment-frame connections. Extensive welding can also cause warping of the existing structure. Figure 5-4 shows examples of frame stiffening details.

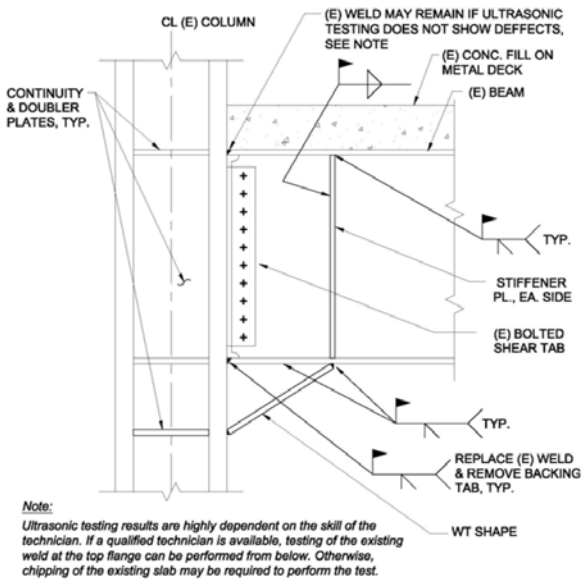



Figure 5-3. Typical Welded Beam Haunch Upgrade Detail[5].

[5] National Institute of Standards and Technology, "*Techniques for the Seismic Rehabilitation of Existing Buildings*," FEMA-547, Interagency Committee on Seismic Safety in Construction Subcommittee 1, Gaithersburg, MD,, October 2006.

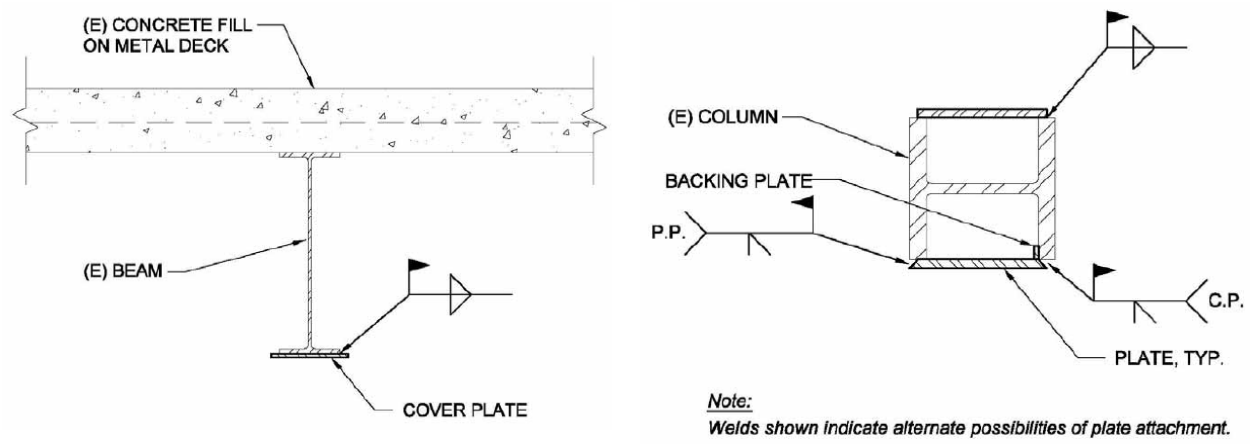



Figure 5-4. Typical Beam Stiffening/Strengthening Details for Beams and Columns[5].

Note that an effect of stiffening the existing beams and columns is an increase in the seismic forces, which could require additional upgrades in the lateral-force-resisting system, similar to those required for the steel-braced frame upgrade strategy.

### 5.2.3 Recommended Seismic Upgrade Strategy

Considering the aforementioned objectives of implementing the seismic upgrades for these buildings, the following general performance criteria were used in determining the appropriate upgrade strategy:

1. Reduce force demands on existing concrete walls at the three cores and diaphragms.
2. Reduce overall building displacements and inter-story drift.
3. Reduce force demands on existing moment frame connections.
4. Reduce rotational demands on existing moment frame connections.
5. Minimize interruption of NRB procedures.
6. Place upgrades in opportune locations (as much as is practicable).
7. Employ a cost-effective upgrade strategy.

Based on these criteria and information provided in previous sections of this report, it is recommended the Seismic Rehabilitation Program incorporate and implement a voluntary seismic upgrade to the Natural Resources Building to reduce overall seismic risk.

[5] National Institute of Standards and Technology, "*Techniques for the Seismic Rehabilitation of Existing Buildings*," FEMA-547, Interagency Committee on Seismic Safety in Construction Subcommittee 1, Gaithersburg, MD, October 2006.

## 5.3 Planning & Budgeting Recommendations

Since deficiencies have been identified in this evaluation, it is recommended that the state of Washington continue with the remaining Planning, Budgeting, Funding, and Implementation stages of the Seismic Rehabilitation Program, as shown in the Figure 5-5.

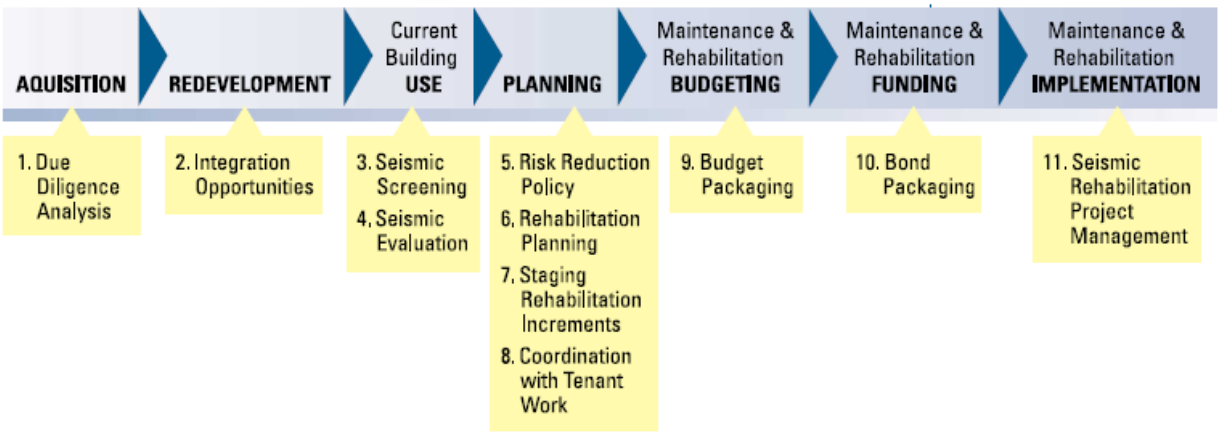



Figure 5-5. Integration of Rehabilitation Elements within the Typical Sequential Facility Management Process[6].

A seismic upgrade design and a detailed construction cost estimate have not been prepared at this time. However, based on the deficiencies, building age, occupancy, and the assumption that the building must remain in operation while under construction, budgetary upgrade costs could range from $9 to $27 per square foot. For the 549,500 square foot Natural Resources Building, construction cost could range from $5 M to $15 M.

### Limitations

The professional services described in this report were performed based on available record drawing information and limited visual observation of the structures. No destructive testing was performed to qualify as-built conditions or verify the quality of materials and workmanship. No other warranty is made as to the professional advice included in this report. This report provides an overview of the seismic evaluation results and does not address the building's programming and planning issues. This report has been prepared for the exclusive use of the state of Washington and is not intended for use by other parties, as it may not contain sufficient information for purposes of other parties or their uses.

[6]Frederick Krimgold et. al., "*Incremental Seismic Rehabilitation of Office Buildings: Providing Protection to People and Buildings*," Risk Management Series, FEMA-397, Federal Emergency Management Agency, Washington, D.C., December 2003.

# APPENDIX A

# ASCE 31-03 Checklists

# Basic Structural Checklist For Building Type C2: Concrete Shear Walls With Stiff Diaphragms

This Basic Structural Checklist shall be completed where required by Table 3-2.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), or Not Applicable (N/A) for a Tier 1 evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

## Building System

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | LOAD PATH: The structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1) | |
| X | | | MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3) | |
| X | | | WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story, above or below, for Life-Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1) | |
| X | | | SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80 percent of the average lateral-force-resisting system stiffness of the three stories above or below for Life-Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2) | |
| | X | | GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3) | The horizontal dimension increases in the project south direction by 42 feet at the top two levels. |
| X | | | VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4) | |
| X | | | MASS: There shall be no change in effective mass more than 50 percent from one story to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses, and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5) | |
| X | | | TORSION: The distance between the story center of mass and the story center of rigidity shall be less than 20 percent of the building width in either plan dimension for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.6) | |
| X | | | DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements. (Tier 2: Sec. 4.3.3.4) | |

| | | | | |
|---|---|---|---|---|
| | | X | POST-TENIONING ANCHORS: There shall be no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used. (Tier 2: Sec. 4.3.3.5) | |
| X | | | CONCRETE WALL CRACKS: All existing diagonal cracks in wall elements shall be less than 1/8 inch for Life Safety and 1/16 inch for Immediate Occupancy, shall not be concentrated in one location, and shall not form an X pattern. (Tier 2: Sec. 4.3.3.9) | |

## Lateral Force Resisting System

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | COMPLETE FRAMES: Steel or concrete frames classified as secondary components shall form a complete-vertical-load carrying system. (Tier 2: Sec. 4.4.1.6.1) | |
| X | | | REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.1.1) | |
| | X | | SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than the greater of 100 psi or $2f'_c{}^{1/2}$ for Life Safety and for Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.1) | The shear stress in the walls exceeds their capacity for Life Safety at some core walls. |
| X | | | REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be not less than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforced steel shall be equal to or less than 18 inches for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.2) | The minimum ratios in the vertical and horizontal directions are 0.0033 and 0.0028, respectively. |

## Connections

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | TRANSFER TO SHEAR WALLS: Diaphragms shall be connected for transfer of loads to the shear walls for Life Safety and the connections shall be able to develop the lesser of the shear strength of the walls or diaphragms for Immediate Occupancy. (Tier 2: Sec. 4.6.2.1) | Connection strength was not calculated because the building is not being evaluated for IO. |
| X | | | FOUNDATION DOWELS: Wall reinforcement shall be doweled into the foundation for Life Safety, and the dowels shall be able to develop the lesser of the strength of the walls or the uplift capacity of the foundation for Immediate Occupancy. (Tier 2: Sec. 4.6.3.5) | Dowel strength was not calculated because the building is not being evaluated for IO. |

# Supplemental Structural Checklist For Building Type C2: Concrete Shear Walls With Stiff Diaphragms

This Supplemental Structural Checklist shall be completed where required by Table 3-2. The Basic Structural Checklist shall be completed prior to completing this Supplemental Structural Checklist.

## Lateral Force Resisting System

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| | | X | DEFLECTION COMPATIBILITY: Secondary components shall have the shear capacity to develop the flexural strength of the components for Life Safety and shall meet the requirements of Sections 4.4.1.4.9, 4.4.1.4.10, 4.4.1.4.11, 4.4.1.4.12, and 4.4.1.4.15 for Immediate Occupancy. (Tier 2: Sec. 4.4.1.6.2) | There are no concrete secondary components. |
| | | X | FLAT SLABS: Flat slabs/plates not part of lateral-force-resisting system shall have continuous bottom steel through the column joints for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.1.6.3) | |
| | | X | COUPLING BEAMS: The stirrups in all coupling beams over means of egress shall be spaced at or less than d/2 and shall be anchored into the confined core of the beam with hooks of $135^o$ or more for Life Safety. All coupling beams shall comply with the requirements above and shall have the capacity in shear to develop the uplift capacity of the adjacent wall for Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.3) | |
| | | X | OVERTURNING: All shear walls shall have aspect ratios less than 4-to-1. Wall piers need not be considered. This statement shall apply to Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.4) | The building is not being evaluated for IO. |
| | | X | CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements shall be confined with spirals or ties with spacing less than 8 $d_b$. This statement shall apply to Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.5) | The building is not being evaluated for IO. |
| | | X | REINFORCING AT OPENINGS: There shall be added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall. This statement shall apply to Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.6) | The building is not being evaluated for IO. |
| | | X | WALL THICKNESS: Thickness of bearing walls shall not be less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 inches. This statement shall apply to Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.7) | The building is not being evaluated for IO. |

# Supplemental Structural Checklist For Building Type C2: Concrete Shear Walls With Stiff Diaphragms

## Diaphragms

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors and shall not have expansion joints. (Tier 2: Sec. 4.5.1.1) | |
| X | | | OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25 percent of the wall length for Life Safety and 15 percent of the wall length for Immediate Occupancy. (Tier 2: Sec. 4.5.1.4) | Diaphragm opening adjacent to the shear wall extend more than 25 % of the wall length in multiple locations, however, the openings are framed so load can transfer through the diaphragm to lateral-force-resisting elements. |
| | | X | PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7) | The building is not being evaluated for IO. |
| | | X | DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8) | The building is not being evaluated for IO. |

## Connections

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | UPLIFT AT PILE CAPS: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps for Life Safety, and the pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Immediate Occupancy. (Tier 2: Sec. 4.6.3.10) | Pile cap strength was not calculated because the building is not being evaluated for IO. |

# Basic Structural Checklist For Building Type S1: Steel Moment Frames With Stiff Diaphragms

This Basic Structural Checklist shall be completed where required by Table 3-2.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), or Not Applicable (N/A) for a Tier 1 evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

## Building System

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | LOAD PATH: The structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1) | |
| X | | | ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.1.2) | The closet building is 62 feet from the NRB. |
| X | | | MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3) | |
| X | | | WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story, above or below, for Life-Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1) | |
| X | | | SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the lateral-force-system stiffness in an adjacent story above or below, or less than 80 percent of the average lateral-resisting-system stiffness of the three stories above or below for Life-Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2) | |
| | X | | GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3) | The horizontal dimensions increases in the project south direction by 42 feet at the top two levels. |
| X | | | VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4) | |
| X | | | MASS: There shall be no change in effective mass more than 50 percent from one story to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses, and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5) | |

| | | | | |
|---|---|---|---|---|
| X | | | TORSION: The estimated distance between the story center of mass and the story center of rigidity shall be less than 20 percent of the building width in either plan dimension for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.6) | |
| | X | | DETERIORATION OF STEEL: There shall be no visible rusting, corrosion, cracking or other deterioration in any of the steel elements or connections in the vertical- or lateral-force-resisting systems. (Tier 2: Sec. 4.3.3.3) | Steel rusting is visible in the parking levels. |
| X | | | DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting system elements. (Tier 2: Sec. 4.3.3.4) | |

## Lateral Force Resisting System

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | REDUNDANCY: The number of lines of moment frames in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. The number of bays of moment frames in each line shall be greater than or equal to 2 for Life Safety and 3 for Immediate Occupancy. (Tier 2: Sec. 4.4.1.1.1) | |
| | | X | INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames shall be isolated from structural elements. (Tier 2: Sec. 4.4.1.2.1) | |
| | X | | DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 3.5.3.1, shall be less than 0.025 for Life Safety and 0.015 for Immediate Occupancy. (Tier 2: Sec. 4.4.1.3.1) | The drift ratio exceeds the maximum value for both BSE-1 and BSE-2 at multiple levels. |
| | X | | AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10F_y$ for Life Safety and Immediate Occupancy. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than $0.30F_y$ for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.1.3.2) | Axial stress in the first floor moment frame exceed $0.10F_{y.}$ |

## Connections

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| X | | | TRANSFER TO STEEL FRAMES: Diaphragms shall be connected for transfer of loads to the steel frames for Life Safety and the connections shall be able to develop the lesser of the strength of the frames or diaphragms for Immediate Occupancy. (Tier 2: Sec. 4.6.2.2) | Connection strength was not calculated because the building is not being evaluated for IO. |
| X | | | STEEL COLUMNS: The columns in the lateral-force-resisting frames shall be anchored to the building foundation for Life Safety, and the anchorage shall be able to develop the lesser of the tensile capacity of the column, the tensile capacity of the lowest level column splice (if any), or the uplift capacity of the foundation, for Immediate Occupancy. (Tier 2: Sec. 4.6.3.1) | Anchor capacity was not calculated because the building is not being evaluated for IO. |

# Supplemental Structural Checklist For Building Type S1: Steel Moment Frames With Stiff Diaphragms

This Supplemental Structural Checklist shall be completed where required by Table 3-2. The Basic Structural Checklist shall be completed prior to completing this Supplemental Structural Checklist.

## Lateral Force Resisting System

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| | X | | MOMENT-RESISTING CONNECTIONS: All moment connections shall be able to develop the strength of the adjoining members or panel zones. (Tier 2: Sec. 4.4.1.3.3) | The steel moment frames consist of welded connections. Welded Steel Moment Frame connections used in the building and may exhibit unacceptable seismic performance as shown in the poor seismic performance of other WSMF buildings during the Northridge Earthquake. |
| | X | | PANEL ZONES: All panel zones shall have the shear capacity to resist the shear demand required to develop 0.8 times the sum of flexural strengths of the girders framing in at the face of the column. (Tier 2: Sec. 4.4.1.3.4) | The steel moment frames consist of welded connections. Welded Steel Moment Frame connections are used in the building and may exhibit unacceptable seismic performance, as shown in the poor seismic performance of other WSMF buildings during the Northridge Earthquake. |
| X | | | COLUMN SPLICES: All column splice details located in moment resisting frames shall include connection of both flanges and the web for Life Safety, and the splice shall develop the strength of the column for Immediate Occupancy. (Tier 2: Sec. 4.4.1.3.5) | Flanges are welded together and webs are connected with MC's and bolts. Splice strength was not calculated because the building is not being evaluated for IO. |
| X | | | STRONG COLUMN/WEAK BEAM: The percentage of strong column/weak beam joints in each story of each line of moment-resisting frames shall be greater than 50 percent for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.1.3.6) | |
| X | | | COMPACT MEMBERS: All moment frame elements shall meet section requirements set forth by *Seismic Provisions for Structural Steel Buildings* Table I-9-1 (AISC, 1997). (Tier 2: Sec. 4.4.1.3.7) | The minimum width-thickness ratio is 9.16. |

| | | | | |
|---|---|---|---|---|
| | | X | BEAM PENETRATIONS: All openings in frame-beam webs shall be less than 1/4 of the beam depth and shall be located in the center half of the beams. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.1.3.8) | The building is not being evaluated for IO. |
| | | X | GIRDER FLANGE CONTINUITY PLATES: There shall be girder flange continuity plates at all moment-resisting frame joints. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.1.3.9) | The building is not being evaluated for IO. |
| | | X | OUT-OF-PLANE BRACING: Beam-column joints shall be braced out-of-plane. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.1.3.10) | The building is not being evaluated for IO. |
| | | X | BOTTOM FLANGE BRACING: The bottom flange of beams shall be braced out-of-plane. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.1.3.11) | The building is not being evaluated for IO. |

## Diaphragms

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| | | X | PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7) | The building is not being evaluated for IO. |
| | | X | DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8) | The building is not being evaluated for IO. |

## Connections

| C | NC | N/A | EVALUATION STATEMENT | COMMENT |
|---|---|---|---|---|
| | | X | UPLIFT AT PILE CAPS: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps for Life Safety, and the pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Immediate Occupancy. (Tier 2: Sec. 4.6.3.10) | The building is not being evaluated for IO. |

# APPENDIX B

## Performance Based Earthquake Engineering (PBEE)

# B. Performance Based Earthquake Engineering

The seismic evaluation of the state of Washington Natural Resources Building is based on performance-based earthquake engineering (PBEE) guidelines presented in ASCE 31-03 *Seismic Evaluation of Existing Buildings* (American Society of Civil Engineers, 2002). A general background of PBEE and an overview of seismic rehabilitation objectives, building performance levels, and the seismic evaluation and rehabilitation procedures are included in this section.

## Background

Seismic analysis and design of buildings has traditionally focused on one performance level – reducing the risk of life loss in the design earthquake. The concept of designing essential facilities that are needed immediately after an earthquake evolved after hospitals and other critical facilities were damaged in the 1971 San Fernando, California earthquake. That concept is balanced by the recognition that the cost of rehabilitating existing buildings to higher levels of seismic performance may be onerous to both stakeholders and policy makers.

A comprehensive program was started in 1991, in cooperation with FEMA, to develop guidelines tailored to address this variation of performance levels. The first formal applications of performance-based evaluation and design guidelines were the FEMA 310 *Handbook for the Seismic Evaluation of Buildings – A Prestandard (1998)* and FEMA 273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings (1997).* ASCE 31-02 and FEMA 356 superseded these documents. The new PBEE documents reflect advancements in technology and incorporate case studies and lessons learned from recent earthquakes.

ASCE 31-03 and ASCE 41-06 provide criteria by which existing buildings can be seismically evaluated and rehabilitated to attain a wide range of different performance levels when subjected to earthquakes of varying severity. Relationships are established between structural response and performance-oriented descriptions, such as Operational, Immediate Occupancy, Life Safety, and Collapse Prevention. As illustrated in Figure B-1, each building performance description or objective is related directly to its expected post-earthquake damage state. Each damage state has readily identifiable consequences, including:

- Cost – economic feasibility of restoring the facility to pre-earthquake condition.
- Public Safety – number of critical injuries and casualties to building occupants.
- Downtime – length of time the building is removed from service to make repairs.

Efforts by the Applied Technology Council (ATC), the Pacific Earthquake Engineering Research Center (PEER), FEMA, and others are underway to quantify these estimated losses to allow stakeholders to make more informed decisions.

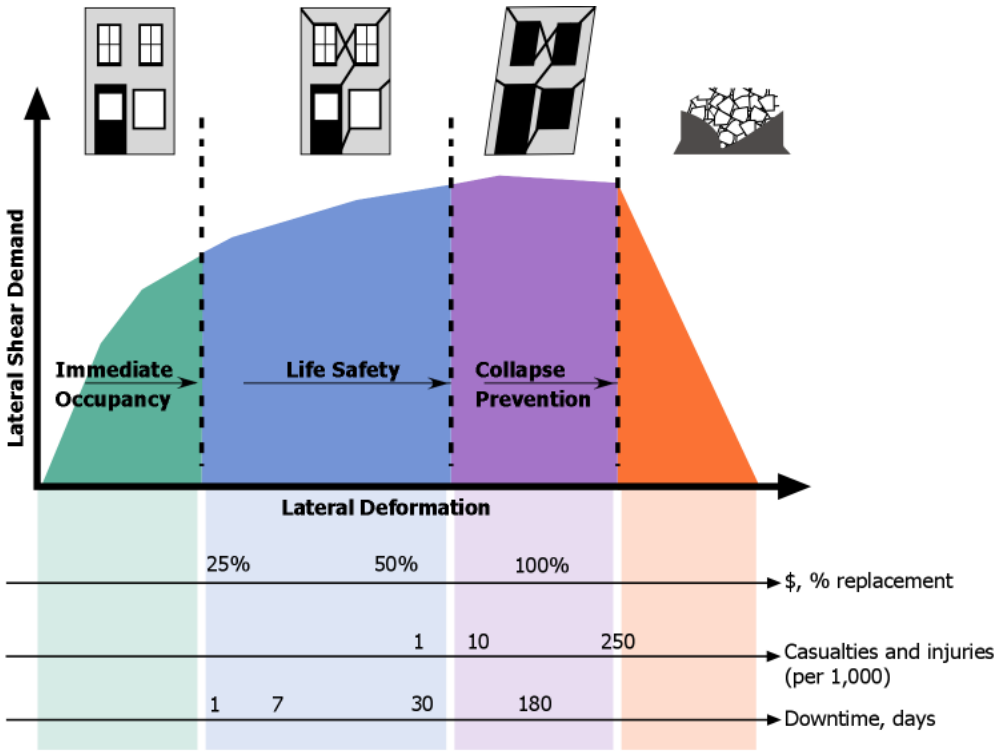



Figure B-1. Estimated Performance-Related Consequences.[2]

## Rehabilitation Objectives

The seismic rehabilitation objective expresses the desired building behavior during an earthquake of projected severity, a design-level event. The objective consists of one or more goals, each with a target building performance level and a corresponding earthquake hazard level. The four defined levels of building performance are Operational (OP), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). Common probabilistic earthquake hazard levels and their corresponding mean return periods are:

Table B-1. Probabilistic Earthquake Hazard Levels and Return Period.

| Earthquake Hazard Level | Probability of Exceedance in 50 Years | Mean Return Period (Years) |
|---|---|---|
| 50%/50-year | 50 percent | 72 |
| 20%/50-year | 20 percent | 225 |
| BSE-1 (10%/50-year) | 10 percent | 474 |
| BSE-2 (2%/50-year) | 2 percent | 2,475 |

A decision must be made for each building as to the acceptable behavior for different levels of seismic hazard, balanced with the cost of rehabilitating the structure to obtain that behavior. Figure B-2 presents the schematic relationship between different rehabilitation objectives and probable program cost.

[2] J. Moehle, "A Framework for Performance-Based Earthquake Engineering," *Proceedings from ATC 15-9, 10th US-Japan Workshop on the Improvement of Structural Design and Construction Practices*, Applied Technology Council, Makena, Hawaii, 2003.

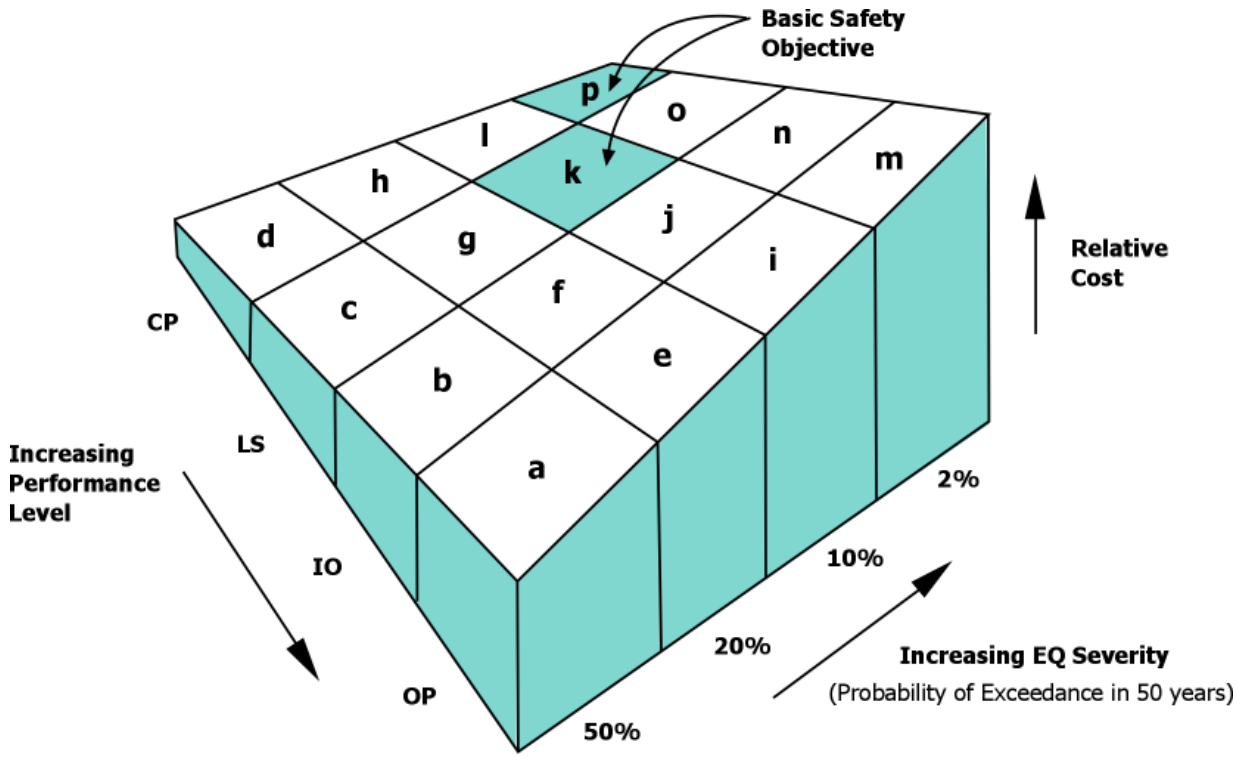



Figure B-2. Surface Matrix of Rehabilitation Objectives.[3]

The "baseline" performance level for a standard building is referred to as the Basic Safety Objective (BSO). The BSO is defined as providing Collapse Prevention performance at the 2%/50-year event and Life Safety performance at the 10 percent probability of exceedance in a 50-year event (10%/50). Higher (enhanced) or lower (limited) objectives may be selected based on the essential nature of the facility, the expected remaining life of the building, and the associated cost and feasibility. The rehabilitation objective selected as a basis for design will determine the benefit to be obtained in terms of improved safety, reduction in property damage, and interruption of use in the event of future earthquakes.

## Building Performance Levels

The terminology used for target building performance levels is intended to represent goals of design. The target levels are discrete damage states selected from among the infinite spectrum of possible damage states that a building could experience during an earthquake.

Since the actual ground motion is seldom comparable to that used for design, the selected damage state may only determine relative performance during most events. Even given a ground motion similar to that used in design, variations from stated performance objectives should be expected. Variations in actual performance could be associated with differences in the level of

[3] Applied Technology Council, "*NEHRP Commentary for the Seismic Rehabilitation of Buildings*," FEMA-274, Federal Emergency Management Agency, Washington, D.C., October 1997.

workmanship, variations in actual material strengths, deterioration of materials, unknown geometry and sizes of existing members, differences in assumed and actual live loads in the building at the time of the earthquake, influence of nonstructural components, and variations in response of soils beneath the building.

Building performance is a combination of the performance of both structural and nonstructural components. Structural performance is related to the amount of lateral deformation or drift of the structure and the capacity or ability of the structure to deform. In the ASCE 31-03 and ASCE 41-06 documents, it is intended that structures meeting Life-Safety performance will be able to experience at least 33 percent greater lateral deformation before failure of primary elements. This equates to a safety factor of 1.33 against collapse. In the design of new buildings, somewhat better performance is expected, since structures are designed with an approximate 1.5 margin against collapse.

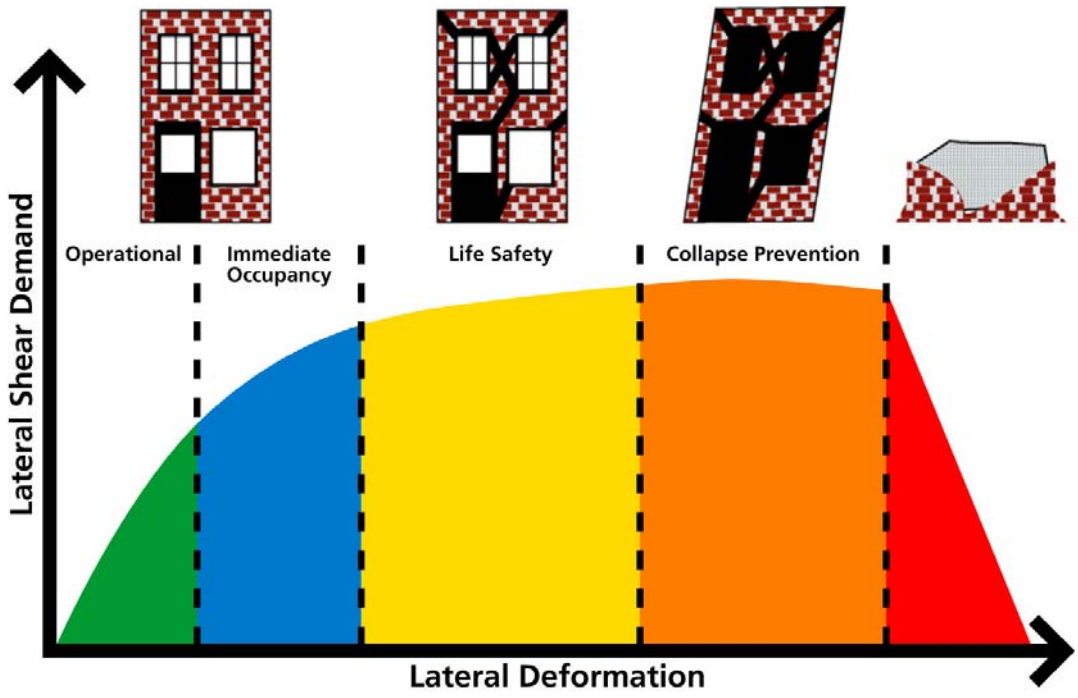



Figure B-3. Building Performance Levels.

Mitigation of nonstructural seismic hazards is a complex issue that is addressed independently in the evaluation and rehabilitation guidelines. Many nonstructural components, if adequately secured to the structure, are seismically rugged. However, mitigation of some nonstructural hazards (such as bracing for mechanical and electrical components within suspended ceiling systems or the improvement of ceiling systems themselves) can result in extensive disruption of occupancy and can also be costly to repair or replace post earthquake. Due to these complexities and the required coordination with other disciplines (i.e., architect, mechanical engineer, electrical engineer, hazardous materials engineer, etc.), nonstructural seismic performance has not been addressed in this initial evaluation. The owner, with assistance from the design team,

will select a nonstructural performance level during the rehabilitation design process that considers the cost-benefit of such mitigation.

Table B-2 summarizes the approximate levels of structural and nonstructural damage that may be expected of buildings rehabilitated to the defined levels.

Table B-2. Damage Control and Building Performance Levels.[4]

| | Building Performance Levels | | | |
|---|---|---|---|---|
| | Collapse Prevention | Life Safety | Immediate Occupancy | Operational |
| Overall Damage | Severe. | Moderate. | Light. | Very Light. |
| Permanent Drift | Large. 1% to 5%. | Some. 0.3% to 1%. | Negligible. | Same as Immediate Occupancy. |
| Remaining Strength and Stiffness After Earthquake | Little. Gravity system (columns and walls) functions, but building is near collapse. | Some. Gravity system functions, but building may be beyond economical repair. | Substantial. Minor cracking of structural elements. | Same as Immediate Occupancy. |
| Examples of Damage to Concrete Framing | Extensive cracking and spalling of concrete members. Crack widths greater than 1/4 inch. | Extensive cracking and spalling of concrete. Crack widths typically less than 1/4 inch and less than 1/8 inch in columns and joints. | Crack widths typically less than 1/8 inch and less than 1/16 inch in columns and joints. | Same as Immediate Occupancy. |
| Examples of Damage to Steel Framing | Extensive yielding and buckling of steel members. Significant connection failures. | Local buckling of steel beams and braces. Moderate amount of connection failures. | Minor deformation of steel members, no connection failures. | Same as Immediate Occupancy. |
| Other General Description | Structure likely not repairable and not safe for reoccupancy due to potential collapse in aftershock. | Repair may be possible, but may not be economically feasible. Repairs may be required prior to reoccupancy. | Minor repairs may be required, but building is safe to occupy. | Same as Immediate Occupancy. |
| Nonstructural Components | Extensive damage. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. | Falling hazards mitigated, but many architectural, mechanical, and electrical systems are damaged. | Minor cracking of facades, partitions, and ceilings. Equipment and contents are generally secure, but may not operate due to lack of utilities. | Negligible damage. All systems important to normal operation are functional. Power and other utilities are available, possibly from standby sources. |
| Comparison with New Building Design | Significantly more damage and greater risk. | Somewhat more damage and slightly higher risk. | Much less damage and lower risk. | Much less damage and lower risk. |

## Seismic Evaluation Procedure

ASCE 31-03 provides a three-tiered evaluation procedure using performance-based criteria. The process for seismic evaluation is depicted in Figure B-4. The evaluation process consists of the following three tiers: Screening Phase (Tier 1), Evaluation Phase (Tier 2), and Detailed Evaluation Phase (Tier 3). A summary of each phase is provided below.

[4] American Society of Civil Engineers, "*Prestandard and Commentary for the Seismic Rehabilitation of Buildings*," FEMA-356, Federal Emergency Management Agency, Washington, D.C., November 2000.

**TIER 1 – Screening Phase**

- Checklists of evaluation statements to quickly identify potential deficiencies
- Requires field investigation and/or review of record drawings
- Analysis limited to "Quick Checks" of global elements
- May proceed to Tier 2, Tier 3, or rehabilitation design if deficiencies are identified

**TIER 2 – Evaluation Phase**

- "Full Building" or "Deficiency Only" evaluation
- Address all Tier 1 seismic deficiencies
- Analysis more refined than Tier 1, but limited to simplified linear procedures
- Identify buildings not requiring rehabilitation

**TIER 3 – Detailed Evaluation Phase**

- Component-based evaluation of entire building using reduced ASCE 41 forces
- Advanced analytical procedures available if Tier 1 and/or Tier 2 evaluations are judged to be overly conservative
- Complex analysis procedures may result in construction savings equal to many times their cost

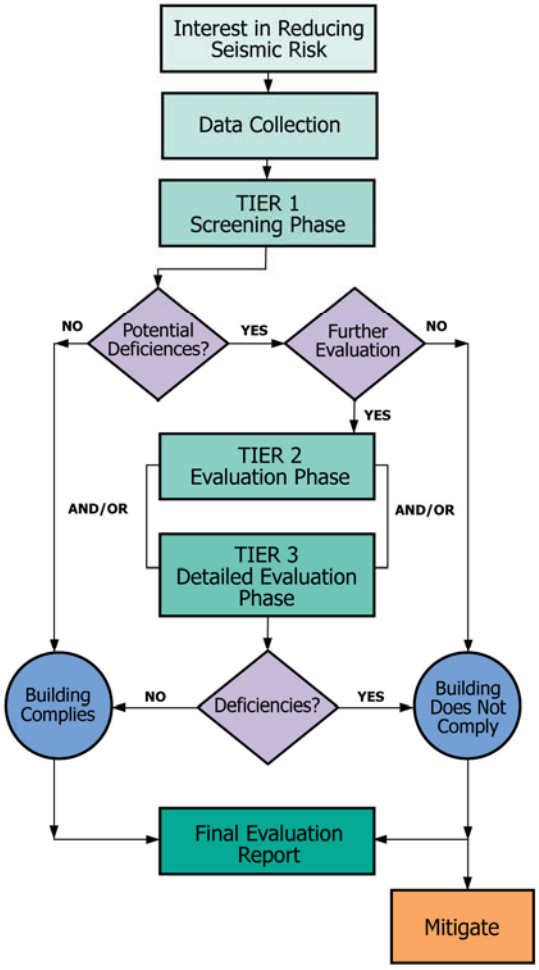



Figure B-4. Flow Chart and Description of ASCE 31 Seismic Evaluation Procedure.

The Tier 3 detailed evaluation references and uses rehabilitation design criteria, such as ASCE 41-06. Since ASCE 31-03 is an evaluation standard, it is written to accept greater levels of damage within each performance level than permitted by retrofit design standards. This is consistent with the historic practice of evaluating existing buildings for slightly lower criteria than those used for design. ASCE 31-03 quantifies this difference by using a 0.75 reduction factor on demands when performing a Tier 3 evaluation. This essentially lowers the reliability of achieving the selected performance level from about 90 percent to 60 percent. This practice generally minimizes the need to rehabilitate structures with relatively modest deficiencies relative to the desired performance level.

## Seismic Rehabilitation Procedure

If seismic deficiencies are identified in the evaluation process, the owner and design team should review all initial conditions before proceeding with the hazard mitigation. Many conditions may affect the rehabilitation design significantly – results of the seismic evaluation and seismic hazard study, building use and occupancy requirements, presence of hazardous materials, and other anticipated building remodeling. The basic process for performance-based rehabilitation design is illustrated in Figure B-5 below.

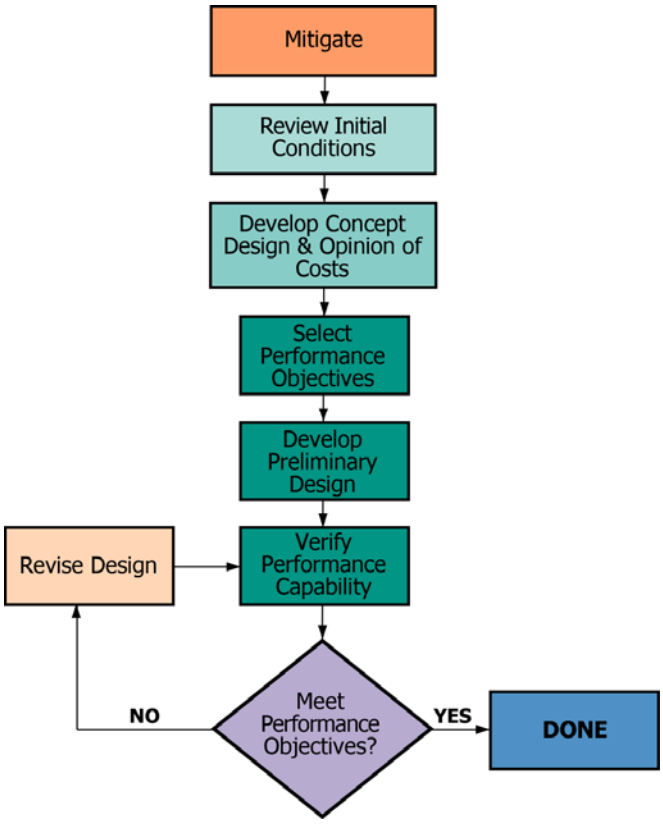



Figure B-5. Seismic Rehabilitation Flow Diagram.

Following the review of initial conditions, concept designs may be performed in order to develop rough opinions of probable construction costs for one or more performance objectives. The owner and design team can then develop a rehabilitation strategy considering the associated costs and feasibility. Schematic and final design can then proceed through an iterative process until verification of acceptable building performance is obtained.

# APPENDIX C

## State of Washington Study

# Transient and Long-term Changes in Seismic Response of the Natural Resources Building, Olympia, WA Due to Earthquake Shaking

**Paul Bodin,[a] John Vidale,[a] Timothy Walsh,[b] Mehmet Çelebi,[c] M.EERI, Recep Çakir[b]**

The Natural Resources Building in Olympia was shaken by the 1999 Mw5.8 and June 2001 M5.0 Satsop earthquakes and the February 2001 Mw6.8 Nisqually earthquake. 11-16 strong-motion channels in the building were recorded on instruments installed by the National Strong Motion Program in collaboration with the Washington Department of Natural Resources. This comprises one of the best dense digital recordings to date of repeated strong shaking in a building. The NRB building has an unusual asymmetric design such that fundamental mode N-S motions are the dominant mode of vibration. In the 1999 earthquake the frequency of the fundamental mode was revealed to be about 1.3Hz during motions of 10%g. The frequency dropped to 0.7Hz during the 2001 Nisqually strong motions, in which the strongest shaking showed remarkable high-frequency transients of up to 0.18 g, several of which are visible on widely spaced stations, and perhaps indicative of damage. The feeble 2001 Satsop earthquake motions showed the frequency remained depressed at less than 1Hz for the eastern side of the structure, although the western side had recovered to 1.3Hz. Finally, an ambient noise survey in 2008 showed the fundamental frequency of N/S vibrations is still about 1.0Hz for the eastern side of the building and 1.3Hz for the western side. These results suggest that the east side of the NRB suffered a permanent reduction in fundamental mode frequency of up to 37%, probably due to the Nisqually earthquake.

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## INTRODUCTION

It is well known that strong earthquake shaking can cause structural weakening or failure of structural systems, components and structural members. Seismic recordings in structures offer possibilities for monitoring their state of health. From seismograms it may be possible (1) to compare recordings before and after damage to identify significant changes in structural response that may serve as indicators of damage, (2) to directly record the transient signals from the damage (*e.g., Rodgers and Çelebi, 2005, 2006, Rodgers, Mahin and Çelebi, 2007*) and (3) to assess thresholds such as drift ratios as indicators of damage using the actual structural geometry, member cross-sections and material properties [*Çelebi and others, 2004*]. Each approach has challenges, but the real-time approaches are becoming increasingly practical with improvements in recording and telemetry efficiency [*Çelebi, 2007a, Çelebi and others, 2004, Porter et al., 2006]*. Buildings and bridges [*Çelebi, 2007, Çelebi and others, 2004, Çelebi*, 2006; *Masri et al.*, 2004; *Siringoringo and Fujino*, 2006] may be monitored by such techniques. Case studies such as this study of recordings capturing damage and non-damaging motions allow calibration of the usefulness of such monitoring.

Changes in structural response indicative of damage have been reported, albeit usually with sparse and sometimes only analog instrumentation. Examples include the Imperial Country Services Building severely damaged during the 1979 Imperial Valley earthquake *[Rojahn and Mork, 1981]*, and from buildings damaged by the 1994 Northridge earthquake *[e.g., Anderson and Filippou, 1995,* and *Naiem, 1998]*. The resonant frequency can be problematic to assess because the spectral response of the building itself must be separated from the frequency content of the seismic waves incident at the base of the building. In addition, individual channels of motion have spectral holes and peaks due to non-structural vibrations and difficult-to-model higher modes. At the building discussed in this paper, we are fortunate to have multiple digital recordings at several levels in the building, so we are able to average out much of the noise, as well as separate the incident energy from the earthquake, which dominates at lower building levels, from the structural response, which dominates at higher levels.

One view is that most increases in period due to shaking arise from damage to the ground around the building, and are temporary [*Trifunac et al.*, 2001b]. In contrast, there is one clear case of permanent period drop from 2.2 to 1Hz in the Imperial County Services Buildings, which suffered serious damage in the 1979 Imperial Valley earthquake [*Rojahn and Mork,*

*1981, Bradford*, 2006; *Todorovska and Trifunac*, 2007]. Changes in period up to 40% in a survey of strongly shaken buildings using analog records have been reported, although without checking back later to see whether the changes are permanent [*Li and Mau*, 1997]. A temporary drop of 60% has been inferred from a single-station analog record in the case of a badly damaged building, and the mode is seen to return to its pristine period just weeks later [*Trifunac et al.*, 2001a]. Detection of permanent changes in response during digitally-recorded earthquakes has been elusive [*Rodgers and Çelebi*, 2006].

Changes in resonant period lasting only minutes to days of 10-20% due to mild shaking, wind, rainfall, and temperature changes without significant damage have also been demonstrated [*Clinton et al.*, 2006; *Kohler et al.*, 2005; *Kohler et al.*, 2007; *Luco et al.*, 1987; *Todorovska and Al Rjoub*, 2006]. Repeated measurements widely separated in time in another study from analog (and digital) records showed 20% variations in estimates without a clear evolution or relation to shaking [*Rodgers and Çelebi*, 2006].

Direct recording of high-frequency transients within strong shaking also presents challenges, as the high-frequency nature of some damage signals can be problematic to recover from analog recording, and interpretation from single instruments is fraught with ambiguity. Only a few examples are of sufficient recording quality to examine [*Rodgers and Mahin*, 2006]. The extent of damage that crackling and response changes might reveal has proven difficult to ascertain [*Krishnan et al.*, 2006; *Kunnath et al.*, 2004; *Lee and Foutch*, 2002; *Righiniotis and Imam*, 2004; *Rodgers and Mahin*, 2006].

The largest challenge to interpretation of transients is the lack of digital recordings of past strong shaking to establish a baseline [*Çelebi,* 2007b, *Rodgers and Çelebi*, 2005]. We ameliorate this shortcoming by describing a unique dataset of four recordings from a moderately well-instrumented building with digital dataloggers, the DNR Building in Olympia, Washington.. Many buildings, mostly in California, now are well-instrumented. However, no strategically-placed strong earthquakes have followed the conversion to digital instrumentation in the late 1990s [*Dunand et al.*, 2004].

## THE STRUCTURE AND DATASET

The NRB is a moment-resisting welded-steel-frame structure, erected in 1993. The building has an unusual design, with 5 above-grade stories and 3 sub-grade parking floors (Figure 1). The 5$^{th}$ and 6$^{th}$ story are cantilevered, and extend beyond the lower floors on the

eastern end of the building, supported by long columns. In overall design the building is elongated in the E-W direction, but is asymmetric, and the deformation modes, which we did not compute for this study, may be expected to be complicated.

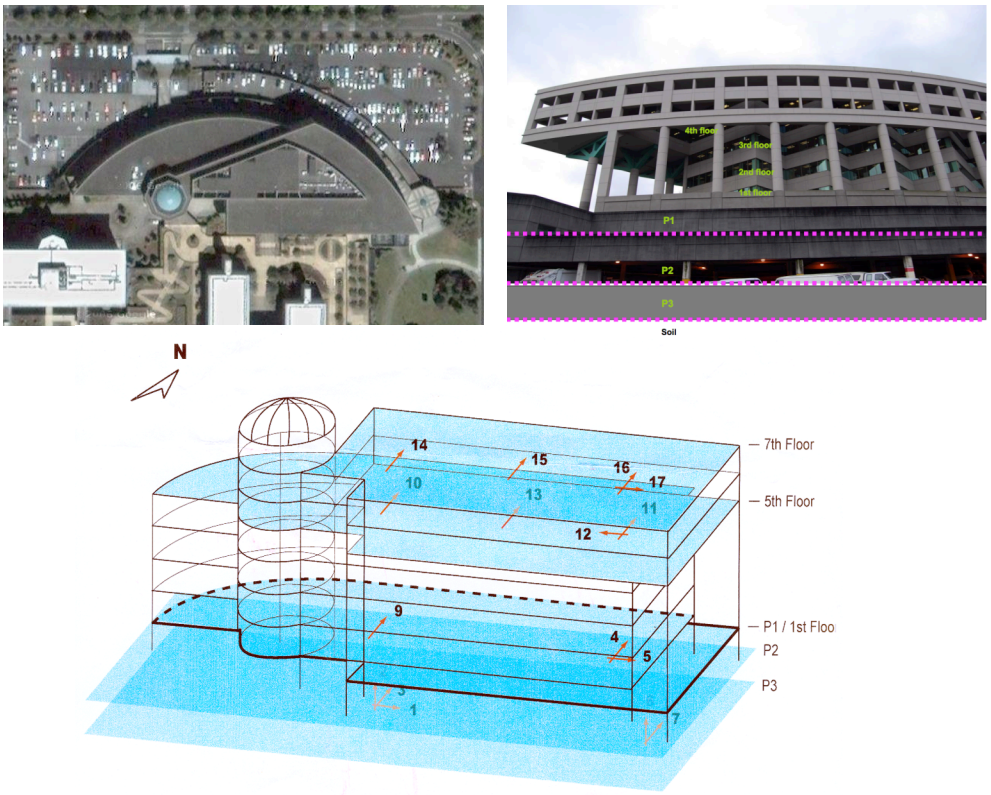


**Figure 1.** (A) Aerial view of the NRB; North is up. (B) View of the east end of the NRB from the NE, showing the cantilevered upper stories. (C) Schematic diagram of sensor locations and orientations in NRB during collection of datasets 2-4. Each component may be assigned a 3-character location-orientation code where the first and second characters designates its vertical and horizontal position in the building, respectively, and the third its orientation (N, E, Z). The sensor numbers correspond to: 1 = B4_E, 2 = B4_Z, 3 = B4_N, 4 = P2_N, 5 = P2_E, 6 = unused, 7 = B1_N, 8 = B1_Z, 9 = P4_N, 10 = 44_N, 11 = 42_N, 12 = 42_E*, 13 = 53_N†, 14 = R4_N†, 15 = R3_N†, 16 = R2_N†, and 17 = R2_E†. [ * indicates that orientation is reversed, and † indicates that the component failed to record the Nisqually earthquake].

At the time of the 1999 Satsop earthquake, the building was recorded by 5 separate dataloggers. The clocks of the various dataloggers were not sychronized, and one datalogger triggered very late, however, the motions of the rest of the channels were recorded. Subsequent to 1999 and prior to the 2001 Nisqually earthquake, the separate dataloggers were replaced by a single "Mt. Whitney" recording system. In the February 2001 Nisqually earthquake, several of the 16 channels failed to trigger and thus did not record; only 11 records were recovered. All 16 channels recorded the June 2001 Satsop earthquake. A schematic map of the sensor locations in the building since 2001 is also shown in Figure 1.

Most of the sensor locations remained the same during the instrument upgrade, but a few of the recording sites for the 1999 Satsop earthquake may not precisely duplicate current locations, and in this study we used information from the NSMP to reconstruct the 1999 locations *[Christopher Stevens, personal communication, 2008]*. Table 1 summarizes the data sets, event descriptions and peak accelerations at basement and roof of the DNR Building.

All three earthquakes we examine were deep, 40 to 60 km, and at hypocentral distances of about 60 km from the NRB. Motions in the three earthquakes differed greatly because of their magnitudes. Peak horizontal accelerations low down in the buildings were 0.02, 0.19, and 0.006 % g for the 1999 Satsop, 2001 Nisqually, and 2001 Satsop earthquakes, respectively. The highest peak accelerations were all on the upper floors, 0.20, 0.42, and 0.04 %g for the three earthquakes, respectively.

The strength of shaking in this building during the 2001 Nisqually earthquake is clear from anecdotes [*Lasmanis*, 2001], but only minor damage was found by physical inspection [*Smith*, 2001].

We also present results from a recording of ambient building motions made in early 2008 with the goal of identifying the NRB's unforced vibrational modes years after the studied earthquakes.

## OBSERVATIONS—TRANSIENTS & TILT

Potential seismic proxies for shaking-induced damage include bursts of high-frequency energy that we refer to as "pops", a long-term increase in the periods of the oscillation modes of the structure, and strong baseline offsets in acceleration that may reveal long-term structural tilts. We examine evidence for each below.

Figures 2 through 5 illustrate some general features observed in datasets 1-3. Figure 2 shows records of the North and East motions from a sub-ground level and the Roof level for the 1999 Satsop earthquake. The simple and small-amplitude ground motions observed at the lower levels become highly amplified, and the motions are dominated by an apparent resonant frequency that depends on the sensor orientation. These frequencies represent grave fundamental oscillation modes of the building in the two directions. Higher frequency resonance is observed in the East/West direction than in the North/South direction because

the building is relatively elongated—and therefore stiffer—in the East/West direction. There is some suggestion of beating between the two directions.

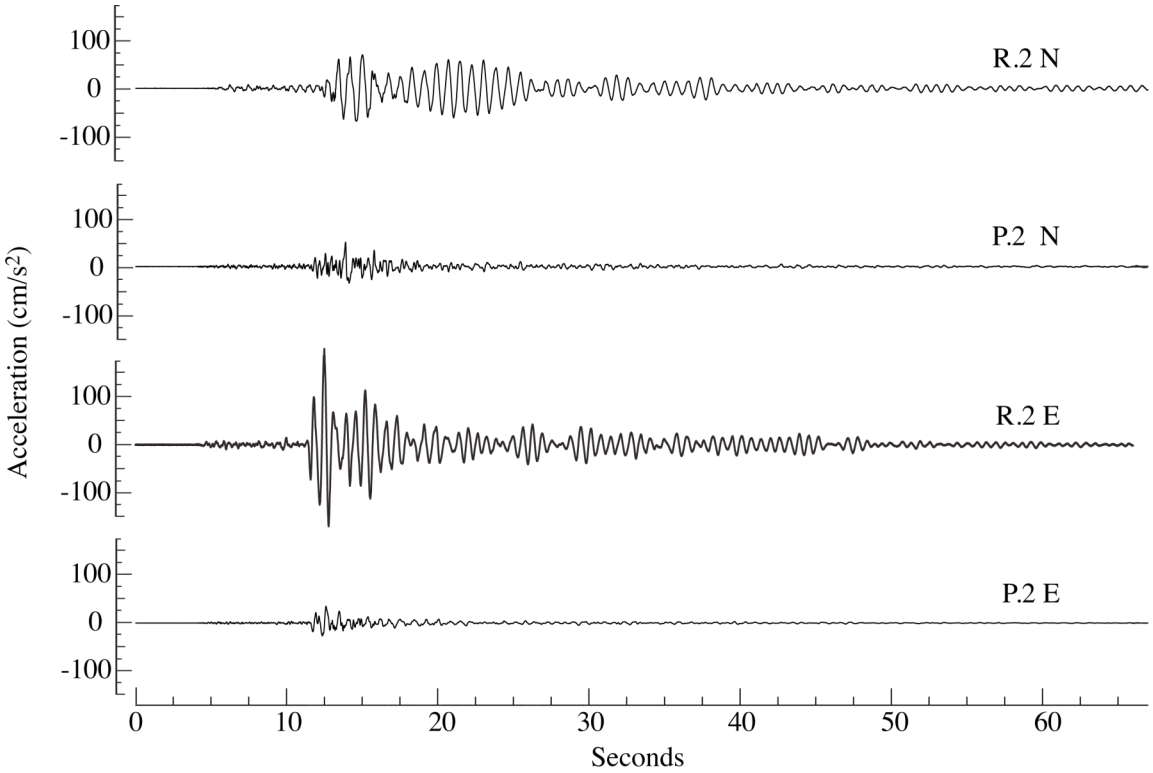


**Figure 2**. Comparison of motions high in building with input motions in the 1999 Satsop earthquake. P2 is a lower level station, R2 is at the roof level. There is a clear N-S resonance of the NRB building from 18-28s, and a similarly strong but briefer E-W resonance from 12-18s. Timing between P2 and R2 components is not synchronized. Amplitudes of P2 records have been halved from those distributed by NSMP, following review of field notes (Christopher Stephens, personal communication 2008). Perhaps there is beating between the E-W and N-S modes. The pulses visible on P2 are probably the direct S at 12s followed by the Love wave at 14s.

One seismogram from each earthquake (datasets 1-3) is shown in Figure 3. The raw acceleration records show that the duration of strong shaking in each event was about 10s, and the dominant period in the incoming wavefield increases with earthquake magnitude. Nisqually had stronger N-S excitation. In general, the Satsop 1999 input ground motion was dominated by signal very close to the building's fundamental resonances, Nisqually ground motions were strongest at periods longer than the building response, while Satsop 2001 input motions were at higher frequency than the building resonances.

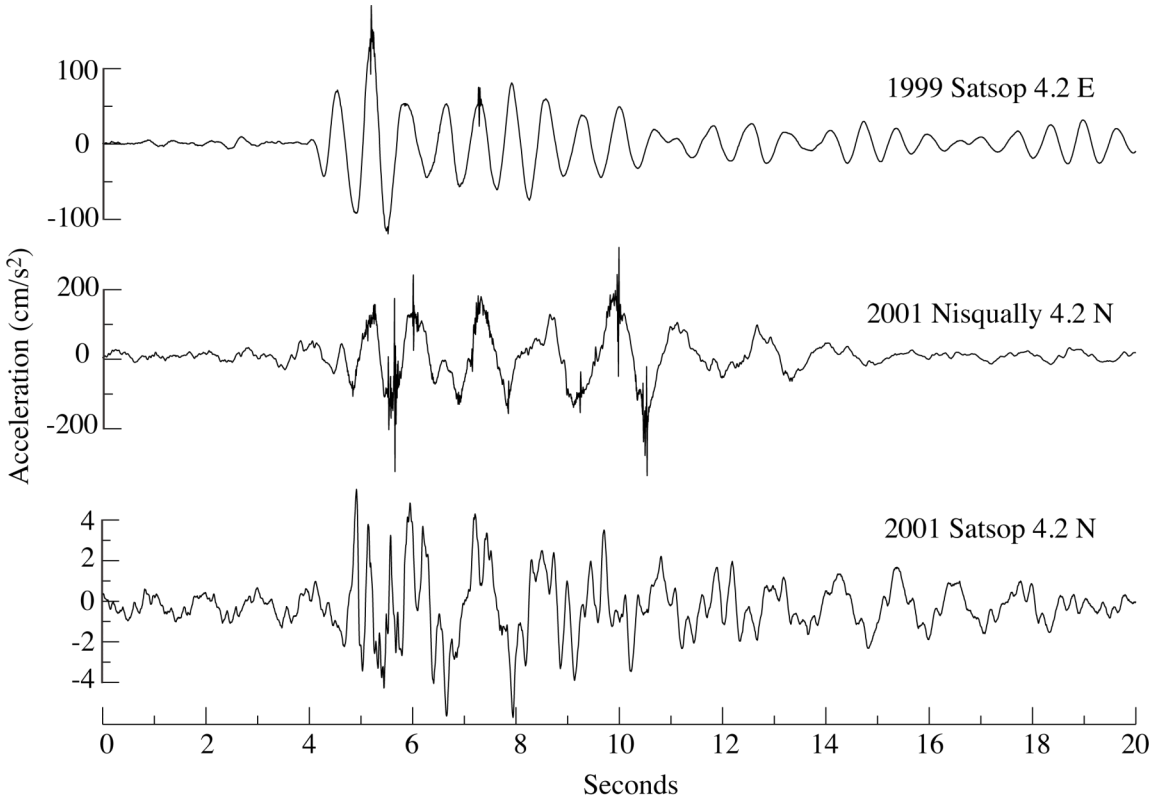


**Figure 3.** (top) Strong-motion record from 1999 Satsop earthquake. (middle) Record from 2001 Nisqually earthquake. (bottom) Record from the 2001 Satsop earthquake. Note the weak pops in 1999 and stronger pops in Nisqually, also note that the amplitude in the 2001 Satsop earthquake is much smaller than the previous two events. These records are from the upper stories of the building.

The component shown in these figures recorded the most numerous and strongest high-frequency bursts and generally the largest overall motions amongst the components. The same seismograms subject to a 20-Hz high-pass filter are shown in Figure 4. Two sharp pops are visible for the 1999 Satsop earthquake, a nearly continuous but staccato stream of pops shows up for the 2001 Nisqually earthquake, while the 2001 Satsop events shows no discernable pops. The bursts during the 2001 Nisqually earthquake were particularly energetic, exceeding 100 cm/s$^2$ at high frequencies, about 10 times stronger than those during the 1999 Satsop earthquake, which in turn exceeded the high-frequency energy during the 2001 Satsop event by up to a factor of 10.

For the 2001 Nisqually earthquake, high-frequency bursts were found on many components. However, perhaps because they were so numerous, we were unable to correlate many individual bursts at different sites in the building unambiguously. Also, we note that generally (but not always) bursts occurred simultaneously with longer-period peaks in the

accelerations. These times are maxima in both the force on the structure local to the sensor and displacement of the structure. We feel that most of the high frequency bursts observed are the result of impulsive sources close to the sites that recorded them, but a few bursts appear to have traveled some distance through the building.

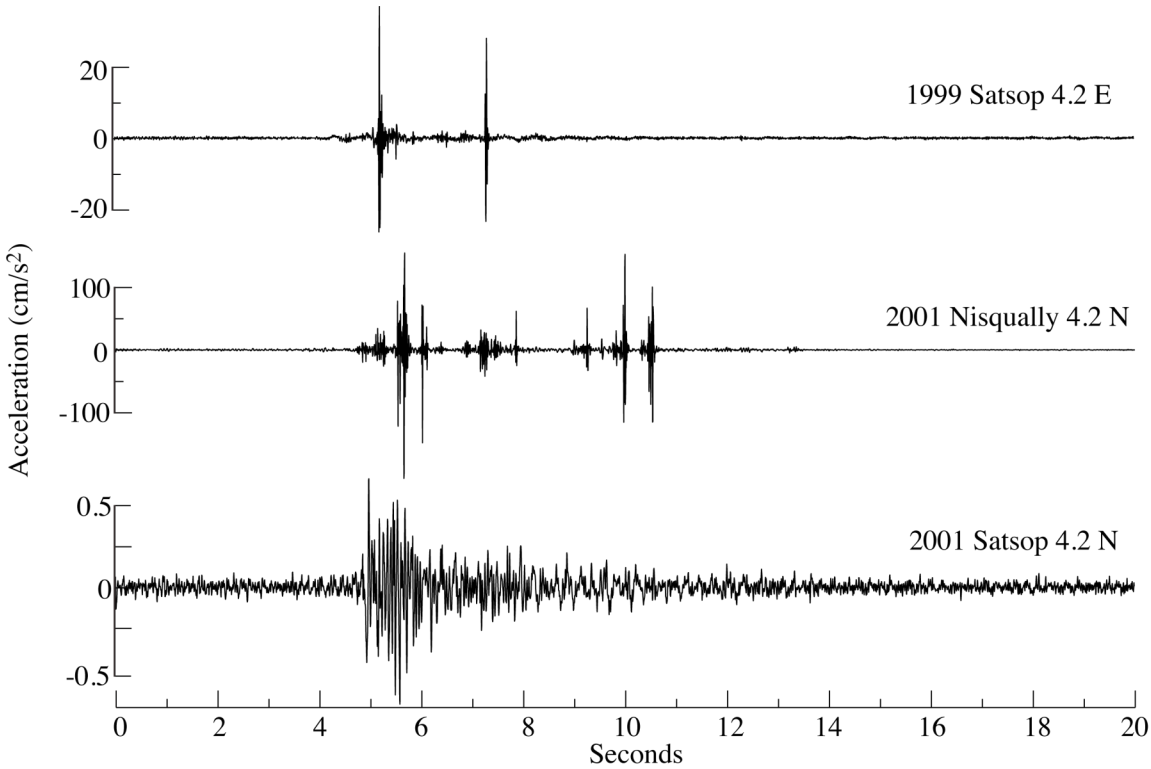


**Figure 4**. The same records as in Figure 3, high-passed at 20Hz. Note the shorter time window and the great variation in amplitude.

We next investigated whether strong shaking in the Nisqually earthquake caused long-lasting changes in the NRB by studying the accelerometer recordings for permanent offsets. Static offsets in acceleration time histories (baseline shifts) are not possible, and instead are due to either instrumental problems or to local rotations (*e.g., Pillet and Vireux, 2007* and references therein; *Grazier, 2007* and references therein). Figure 5 shows how we can reveal baseline shifts by low-pass filtering the accelerogram. The baseline shift begins at about the time of the most energetic pops seen in the high-pass filtered accelerations, which are the bottom time series of each pair. The largest offset, in the 5.2.E component shown in Figure 5, is a static offset of about -1.79 cm/s$^2$ and would correspond to a tilt of about 1.6˚ to the west. *Grazier [2007]* estimates tilt resolution with inertial accelerometers is better than 0.5˚, so this observation represents a significant rotation. Only a handful of apparent tilts were larger than 0.5˚. Moreover, the pattern of apparent tilts we observed, even given the incomplete coverage

of sensors, is not consistent with a uniform rotation of the entire building. Rather there seemed to be generally stronger tilts at the top floor, above the cantilevered portion of the building. Also, tilts in the N/S directions at the deepest parking level were toward the north but toward the south in the high floor. While the strongest shaking was in the N/S direction, the strongest static tilt was in an E/W component on the 5th floor. We conclude that the observed tilt pattern suggests localized quasi-static deformations near the affected sensor.

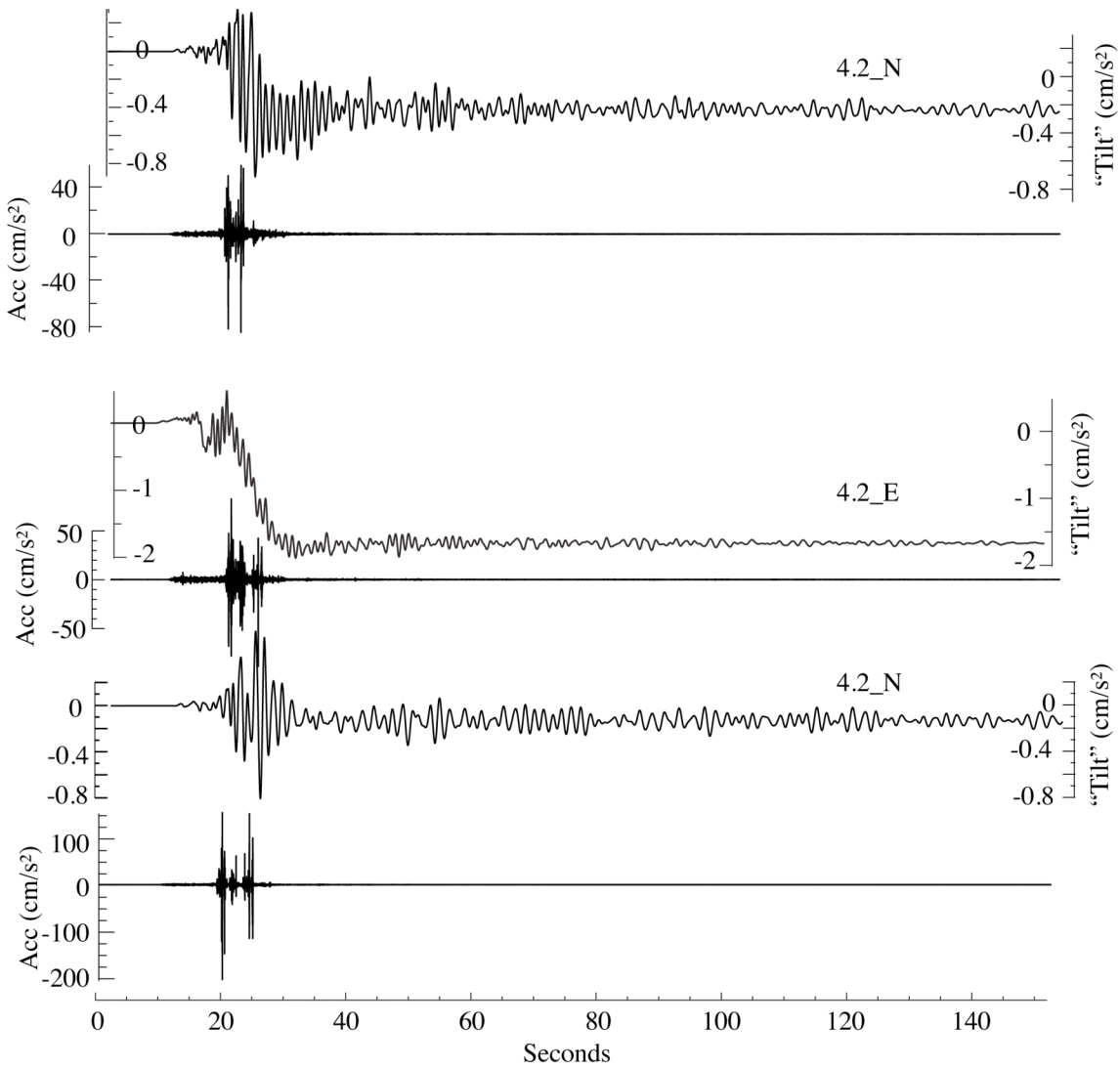


**Figure 5.** NRB permanent tilts associated with Nisqually high-frequency transients. Nisqually accelerations from 3 sensors on the 5th floor of the NRB high passed at 20 Hz to reveal "pops", and low-passed at 0.05 Hz to show permanent baseline offset in acceleration. We feel the most plausible explanation for the baseline offsets are permanent tilts in structural member to which the sensors are attached. The onsets of the apparent tilt signals are closely tied to the large pops. The duration of the apparent tilt transient is controlled by the filter, and we attribute it no physical significance.

Given our interpretation of the offsets and high frequency pops, as well as the amplitude of shaking, we infer the greatest response changes may have been associated with the Nisqually earthquake, with perhaps some in the 1999 Satsop earthquake, and not much in the 2001 Satsop event.

## OBSERVATIONS—RESONANCE FREQUENCIES

The unforced vibrations of the NRB recorded in 2008 revealed that the fundamental modes of the structure are complicated, as might be expected for an asymmetric and unusual structure. The largest ambient motions as of 2008 were in the N/S direction at the eastern end of the structure at about 1 Hz (Figures 6), and were systematically largest in the upper floors of the building. The motions were also systematically much larger on stations in the east than the west.

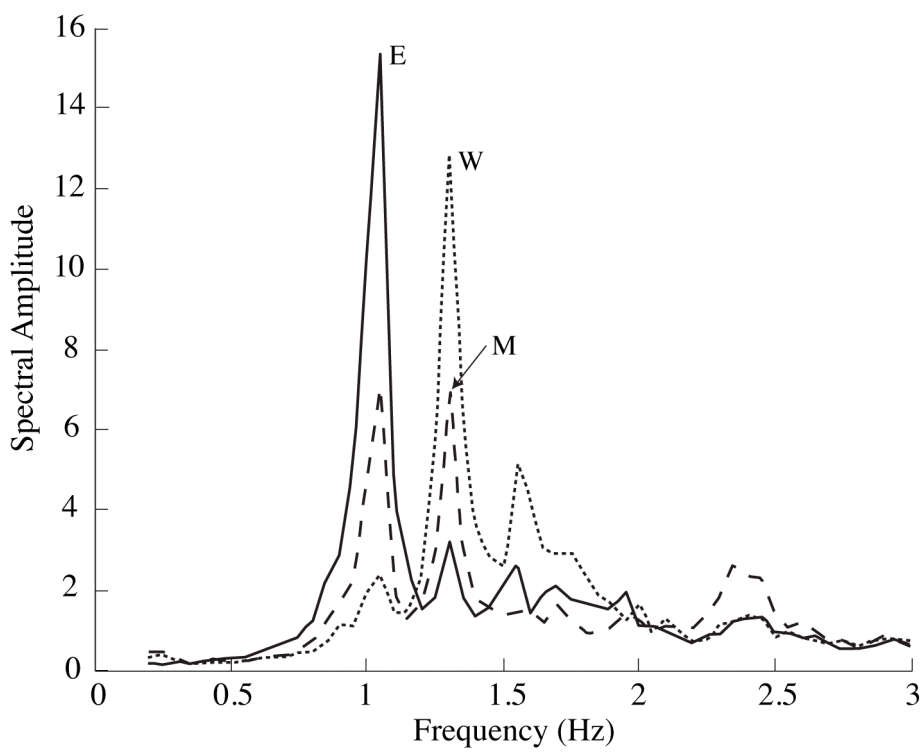


**Figure 6.** Spectra of ambient NS motions in 2008 from dataset 4. Shown are medians of the noise spectra computed for many small windows in the 5000 second recording, to avoid unduly emphasizing irrelevant noise bursts. The solid curve labeled "E" is for the easternmost stations, long dashed line (labeled "M") for the middle, and short dashed line ("W") for the westernmost.

The synchronous phasing as well as the spatial pattern of these motions in the time series of Figure 7 indicates that this is the fundamental N/S mode. At the west end of the building,

the largest N/S motions are at ~1.3 Hz. The E/W motions (not shown) are similarly amplified although peak at a higher frequency (~1.6 Hz), which is appears as a weak peak in the N/S motions, consistent with a stiffer structure in the building's widest dimension. In addition to variations of stiffness, the data are consistent with coupling between torsional and linear-motion modes. A similar analysis of the broad and much smaller peak in Fig. 6 at ~2.4 Hz., suggests that this is a first higher mode of N/S building motion.

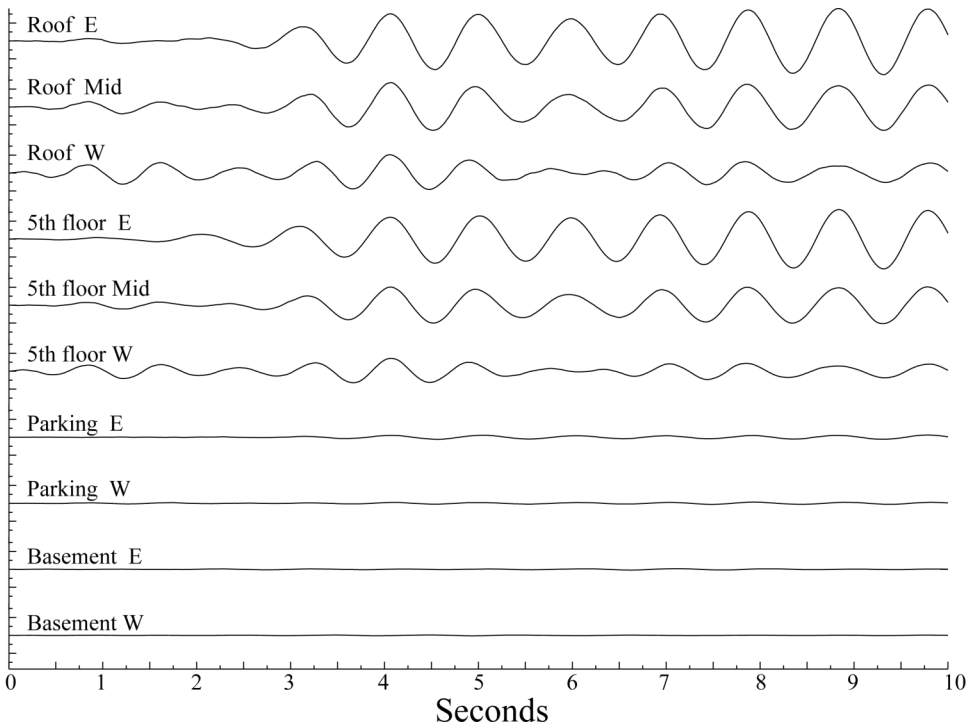


**Figure 7.** Time series of a typical 10-second segment of selected channels of 2008 ambient motion recordings (N/S components) from dataset 4. These have been band-passed between 0.8 and 1.2 Hz. Traces are labeled with the story (Roof, 5$^{th}$ Floor, Parking level #2 and Basement) and location (East, Middle, or West). Note the coherence and amplitude pattern of traces revealing that these motions are sampling a fundamental N/S resonance mode of the building.

In order to track the temporal evolution of modal frequencies during the earthquakes, we processed the recordings into a series of spectrograms, being cognizant of the pattern of the NRB modal response discussed above. In order to focus on the building response, as distinct from the ground input motions, we form spectral ratios between components high in the building (floors 4 and above) and those at the lowest sub-surface sites. Because at the resonance period of the dominant mode of the components were in phase, we sharpened the resolution, where possible, by summing the upper-story N/S components before forming the ratios. Due to changing station layout and failures of some channels to trigger during the

Nisqually earthquake, the ratio spectrograms shown in Figures 8 & 10-11 sometimes include different components.

The 1999 Satsop records from the east end of the NRB (Figure 8) start with a resonance peak at ~1.6 Hz. During the strongest shaking, the resonance peak frequency decreases ~22% to 1.25 Hz, recovering to ~1.4 Hz by the end of the record, for an overall observed drop of ~12%. We have no data to reveal if it recovered more in the hours to days afterward. The next record starts in 2001 with the Nisqually shaking. Early in the Nisqually record the N/S resonance peak is about 1.3 Hz, with the E/W peak at ~1.9 Hz. (Figure 9). Within 20 seconds, the N/S peak has reduced nearly 50% to about 0.7 Hz (Figure 10). By the end of the Nisqually recording, the peak in the N/S spectral ratio has recovered slightly to ~0.8 Hz. Then in June 2001, the recording of the second Satsop earthquake starts with an apparent N/S resonance peak at ~0.8 Hz (Figure 11), which stays at 0.8-0.9 Hz throughout the recording. In the ~6.5 years between the 2001 Satsop earthquake and the 2008 ambient motion samples illustrated in Figure 6 and 7 the fundamental period of N/S motions at the east end of the NRB has only partially recovered, to ~1 Hz. The NRB has apparently lost stiffness in this mode, leading to an overall drop in resonance frequency of ~37%.

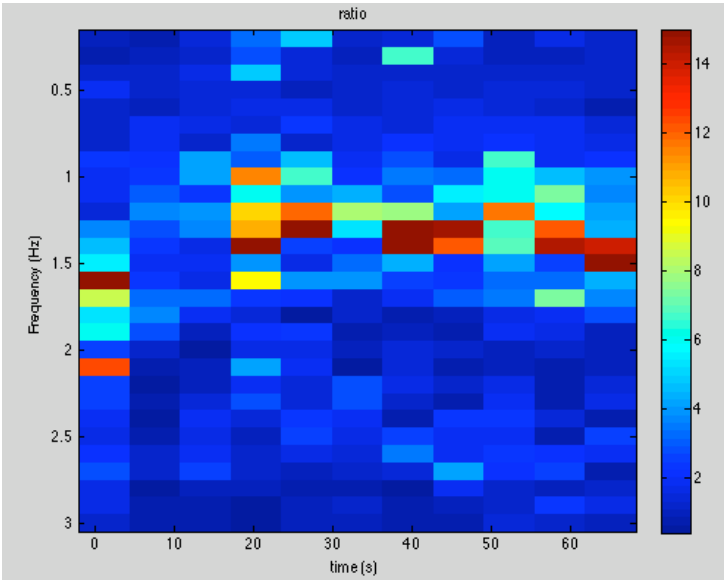


**Figure 8.** Spectral ratio of the sum of the three NS sensors that measure the fundamental mode of the east side of the NRB in the 1999 Satsop earthquake, divided by the two least amplified N channels, Satsop_1999_B1_HNN.sac and Satsop_1999_P4_HNN.sac. The plot has been clipped at a spectral ratio of 15. Notice the distinct peak about 1.6Hz before the strong motion. The resonance peak shifts to 1.25Hz during strong motion, which arrives from 10 to 25s, settling back to about 1.4Hz by 30 seconds later.

Analyses of both E/W motions, and N/S motions at the west end of the building (not shown) reveal large temporary reductions in resonance period during strong shaking that were nearly fully recovered during the recording. However comparing the clear 1.9 Hz peak in the pre-Nisqually E/W motion at the east end of the building (Figure 9), to the strong spectral peak at ~1.6 Hz at these stations in the ambient motions of 2008 (not shown) leads to the inference of a long-term drop in resonance period of ~16% in the E/W resonance there.

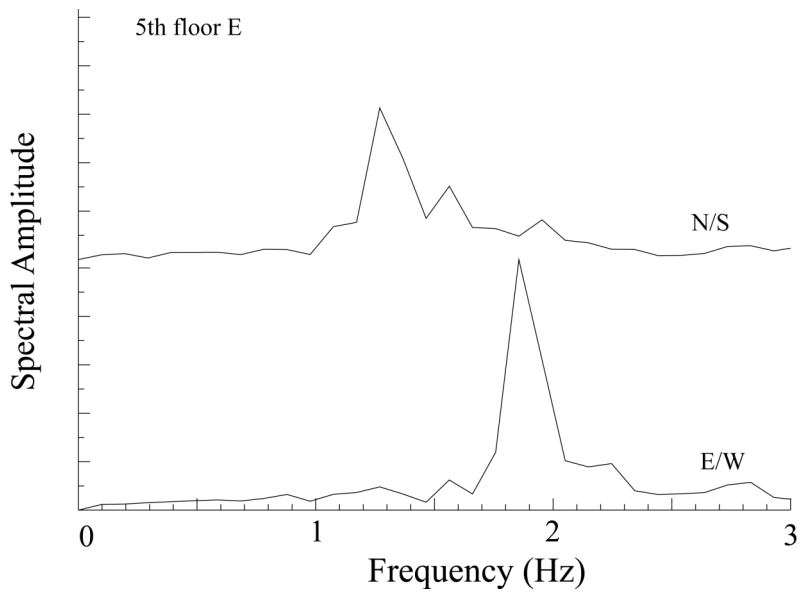


**Figure 9.** Spectra of the initial 9.5 s (pre-event) of Nisqually records (top=N/S, bottom = E/W) from the east end of the 5th floor of the NRB.

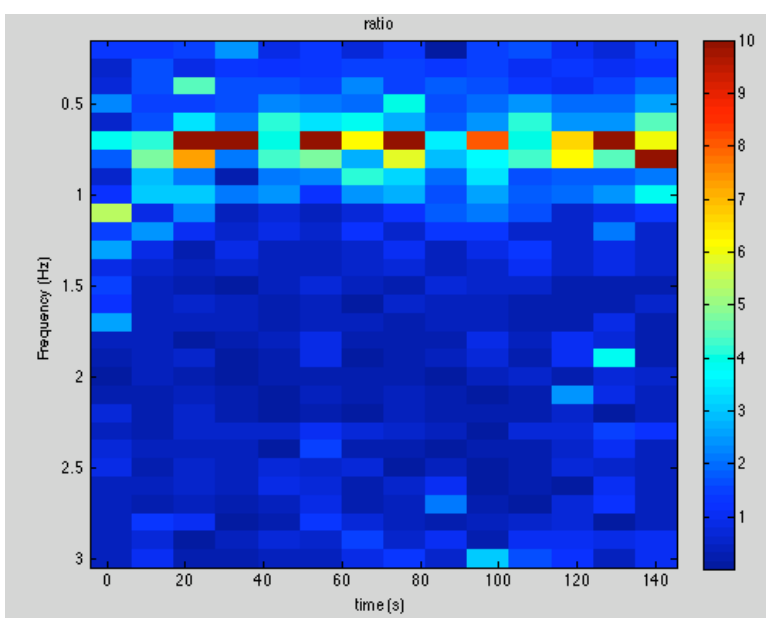

**Figure 10.** Spectral ratio of the one working NS sensor that measured the fundamental mode in the Nisqually earthquake divided by the average spectrum of the two basement NS channels. The plot has been clipped at a spectral ratio of 10. Also, the amplification is less than in the 1999 earthquake, probably due to increased damping

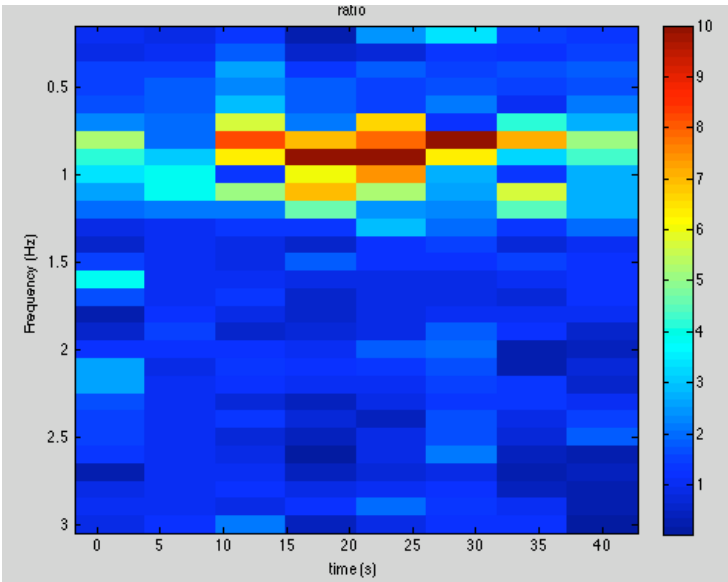


**Figure 11.** Spectral ratio of the sum of the four NS sensors that measure the fundamental mode from the 2001 Satsop earthquake, divided by two basement NS channels. The plot has been clipped at a spectral ratio of 10. Notice the distinct peak about 0.9Hz during the moderate shaking from about 10 to 15s.

## DISCUSSION

Figure 12 condenses the observations of the North-South building response frequencies presented above to more clearly reveal the history of this dominant deformation mode. During the 1999 Satsop earthquake, the NRB N/S resonant frequency temporarily dropped about 22%, and may have permanently dropped 12%. The larger Nisqually shaking reduced the frequency much further, and the 2008 ambient recordings reveal the changes to be a long term change. From early in the 1999 Satsop recordings to 2008 the N/S motions in the eastern side of the building dropped ~37%. An estimate of the long-term reduction in resonant frequency of E/W motions from early in the Nisqually records to the recordings of ambient motions made in 2008 is ~16%. While the short-term frequency shifts during strong shaking likely reflect the effect of structural ductility, the long-term decreases in modal

frequencies represent a reduction in stiffness that could factor into assessment of seismic response in future earthquakes.

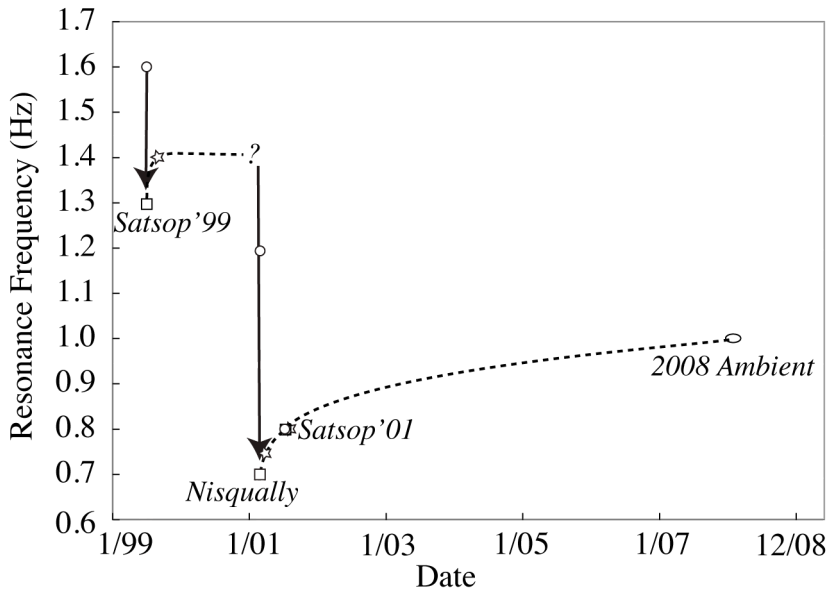


**Figure 12.** History of resonance frequencies of the gravest mode of N/S motions at the East end of the NRB. Circles are peak frequencies early in each dataset, squares in the middle of each recording, during strongest shaking, and stars at the end of each recording. Dashed lines are schematic trajectories connecting datasets. Arrows are frequency drops at the time of strong shaking. The query at the time of the Nisqually earthquake reflects the fact that our earliest time window included some shaking which may have already reduced the resonance peak somewhat.

The 28% drop and 12% recovery in frequency in the N/S motions of the fundamental mode during the first Satsop event is comparable to values observed in the Millikan library due to the San Fernando earthquake, albeit at lower acceleration levels [*Luco et al.*, 1987]. The additional 38% drop in the Nisqually earthquake is without precedent for undamaged buildings in the literature, to our knowledge. It is an order of magnitude greater than estimates of changing temperature, rainfall, and wind [*Clinton et al.*, 2006]. However, a direct mapping of frequency changes into stiffness reductions is fraught with uncertainties about how much loss of stiffness is generated by degeneration of the horizontal stiffness as opposed to loss of rocking stiffness controlled by soil-structure interactions [*e.g., Luco et al.*, 1987]. Such an interpretation would demand a more complete analysis of the building deformation modes than we have done.

The causal relation between pops and permanent tilts and the loss of stiffness is not well known, perhaps due to paucity of digital records, with a fairly complete review of results recently published [*Rodgers and Çelebi*, 2005]. A 10% reduction in resonant frequency in an Alhambra building from the Whittier Narrows quake was not accompanied by audible

cracking, however the limitations of analog recording prevents a sensitive analysis [*Rodgers and Çelebi*, 2006]. Inspection in that case revealed weld defects or small cracks in 30 of 52 welds inspected. The problems were attributed to construction, but further damage during shaking cannot be ruled out [*Rodgers and Çelebi*, 2006].

Monitoring changes in building translational and torsional modes can resolve significant reductions in stiffness in this case, and such techniques may prove of general usefulness in monitoring structural health. To understand fully the stability and resolution of modal frequency tracking, continuous—or at least regular and frequent sampling—of the ambient motions would be quite helpful. Similarly, the presence of high-frequency transients and permanent baseline offsets in digital accelerograph recordings of structural motions may help to locate and characterize the sources of localized failures. More experimental and theoretical work is needed to understand the forensic significance of the signals we observe. None of the analyses we present take a lot of time to carry out. All might be done within minutes after a large earthquake, potentially enabling rapid (and even automatic) seismic assessments of structural state-of-health very soon following a potentially damaging event. Combined with a model of the modal deformations calibrated by the ambient motions, it might be possible to determine loci of stiffness reduction.

**Table 1**. Datasets, Event Characteristics and Peak Accelerations Recorded at NRB Building (Δ= hypocentral distance, km (epicentral distance in parentheses))

| Dataset | Event | | | | Peak Accel (Component) | |
|---|---|---|---|---|---|---|
| | Name | Date | Mw | D | Base (P3 level) | Upper Story |
| 1 | Satsop99 | 07/03/1999 | 5.8 | 59.4 (43) | 42.4 (E) | 182.4 (E) |
| 2 | Nisqually | 02/28/2001 | 6.8 | 62 (19) | 374.4 (N) | 215.4 (N) |
| 3 | Satsop01 | 06/10/2001 | 5.0 (Ml) | 63.1 (48.4) | 15.4 (N) | 42.03 (N) |
| 4 | Ambient | 01/23/2008 | -- | -- | < .01 | < .01 |

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# APPENDIX D

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# D References


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