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Independent Review of Seismic Performance Assessments for the Plutonium Facility PF-4

Los Alamos National Laboratory, Los Alamos, NM

Rev 1
October 2, 2015

Independent Review of Seismic Performance Assessments for the Plutonium Facility PF-4

Los Alamos National Laboratory, Los Alamos, NM

Prepared for

Los Alamos National Laboratory
National Nuclear Security Administration
Department of Energy

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Abbreviations

ACI 349	Code Requirements for Nuclear Safety Related Concrete Structures
AFUP	Annual Frequency of Unacceptable Performance
ASCE 4	American Society of Civil Engineers *Seismic Analysis of Safety-Related Nuclear Structures and Commentary,* Standard 4-98 (ASCE 1998)
ASCE 43	American Society of Civil Engineers *Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities,* Standard 43-05 (ASCE 2005)
CJC	Carl J. Costantino and Associates
DBE	Design Basis Earthquake
DNFSB	Defense Nuclear Facilities Safety Board
DOE	U.S. Department of Energy
DOE Standard 1020	Department of Energy, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*
LANL	Los Alamos National Laboratory
MMPA	Multi-Mode Pushover Analysis
NEP	Non-Exceedance Probability
NNSA	National Nuclear Security Administration
PF-4	Plutonium Facility 4 at LANL
SEP	Seismic Expert Panel
SGH	Simpson, Gumpertz and Heger
SSC	Structures, Systems and Components
SSI	Soil-Structure Interaction

Variables

ΣV	total shear demand
$\phi V_{n,ACI}$	code design capacity per ACI 349
C_{ACI}	code capacity per ACI 349

C_{MCFT}	AASHTO modified compression field theory capacity
d	depth of member, also distance from face of support at which girder shear demands are calculated
D/C	demand-capacity ratio; $D/C \leq 1.0$ indicates acceptable response
$D_{50\%}$	median seismic demands
$D_{80\%}$	80% NEP seismic demands
D_{MMPA}	seismic demands calculated using MMPA
f'_c	concrete compressive strength
$F_{CJC,E}$	adjustment to CJC-computed shears at Line E
F_{SGH}	adjustment to SGH-computed demands from MMPA
$F_{SGH,H}$	adjustment to SGH-computed shears at Line H
$f_{\Sigma Vs}$	adjustment to SGH-computed demands from MMPA = 1 / F_{SGH}
M_G	gravity moment
M_S	seismic moment
$V_{A,E}$	adjusted SGH shear demand at Line E
$V_{A,H}$	adjusted SGH shear demand at Line H
v_c	shear stress
V_G	gravity shear
V_{North}	shear demand on north end of interior girder
V_S	seismic shear
$V_{S,50}$	median shear demand
$V_{S,80}$	80% NEP shear demand
V_{South}	shear demand on south end of interior girder
$\Delta_{damping}$	SGH demand adjustment due to damping
Δ_{soil_spring}	SGH demand adjustment due to soil springs
Δ_{SSI}	SGH demand adjustment due to soil-structure interaction
ρ_w	reinforcement ratio based on web area

Executive Summary

The Plutonium Facility, designated PF-4, is located in Technical Area 55 at the Los Alamos National Laboratory (LANL). The facility is a one-story rectangular structure above a complete basement; the building was constructed of cast-in-place reinforced concrete, with small interior frames of structural steel. The plan dimensions of the building are 265’×284’. The overall height of the building varies between 39’-0” at the north and south ends, and 40’-6” at the center ridge. The programmatic work performed in the building is vital to our national security and its functions and storage purposes are not replicated elsewhere in the United States Department of Energy (DOE).

LANL is located in a region of moderate-to-high seismic hazard. Facility PF-4 was designed and constructed in the early 1970s using the appropriate design standards for that time. Earthquake shaking was considered explicitly in its design, and the seismic hazard studies performed in support of the original design can be described as state-of-the-art for the early 1970s.

The 40+ years since the design and construction of PF-4 has seen dramatic improvements in earthquake science and earthquake engineering. These improvements were triggered by earthquakes in the United States and abroad, and research supported by our federal government agencies, including the DOE, Nuclear Regulatory Commission, National Science Foundation, Federal Emergency Management Agency, and the US Geological Survey. The 2007 and 2009 updates to the probabilistic seismic hazards assessment at LANL resulted in an improved understanding of the potential for large ground motions. This new assessment substantially increased the design basis shaking levels with respect to the 1972 calculations by factors of approximately 2.5 in the horizontal direction and 4.5 in the vertical direction. In addition, design standards have evolved to be more onerous and to require the provisions of ductile detailing.

The re-assessments of seismic hazard at LANL in 2007 and 2009, and the associated increases in seismic hazard, prompted LANL, DOE and the National Nuclear Security Administration (NNSA) to evaluate the seismic performance of PF-4, in recognition of its importance to our nation’s nuclear security. The initial assessment by Carl J. Costantino and Associates (CJC), funded by LANL, led to a series of recommendations for facility upgrades, that were by-and-large implemented immediately. The method of analysis used by CJC for their assessment of PF-

4 utilized a loading profile that was based on results of a linear soil-structure-interaction analysis. The Defense Nuclear Facilities Safety Board (DNFSB) identified their concerns with this method of analysis and communicated them to the Deputy Secretary of Energy. In response, the Deputy Secretary instructed NNSA to immediately initiate an action to evaluate PF-4 using a second modeling approach, termed a modal loading analysis, and to consult with DNFSB staff to ensure that the premises underpinning the modal loading analysis took the DNFSB technical perspective into account. Simpson, Gumpertz and Heger (SGH) were retained by NNSA to develop an appropriate analysis methodology and use it to re-assess the performance of PF-4.

The scopes of work assigned to CJC and SGH, by LANL and NNSA, respectively, were different. Given these distinct scopes, it is not surprising that the recommendations from the two studies in terms of the seismic vulnerability of PF-4 are different. To better understand the differences in the two sets of recommendations, to reconcile the differences wherever possible, and to provide DOE and LANL with a plan of action, LANL and NNSA formed a Seismic Expert Panel (SEP), comprised of engineers with expertise in seismic analysis, design, risk assessment and earthquake hazard calculations. The report that follows presents the findings of the SEP.

Revision 0 of the SEP report was submitted on March 31, 2015. SGH and CJC were invited to review the report and provide comments to NNSA and LANL, respectively. These letters and the SEP responses are provided in Appendix E, which is new in this Revision 1 report. Revision 1 of the SEP report includes two changes of note: a) the text of Section 3.4 has been replaced in its entirety to better explain the SEP's opinion on the utility of the SGH-developed multi-modal pushover analysis (MMPA) method, and b) Section 4.3.2 has been updated to correct a misstatement in the Revision 0 report regarding the calculation of roof girder shear capacity on Line 8. These changes are explained in detail in the SEP letters of Appendix E.

We summarize our findings here under the headings of observations, required actions, and prudent actions.

Observations

1. The work of CJC and SGH was exemplary. They delivered to their clients, LANL and NNSA, respectively, the products identified in their scopes of work.

2. DOE Standard 1020-2002 enables seismic assessment of existing facilities, with an assumed short service life, for smaller seismic demands than for a new building with a comparable function. This relief should be set aside for PF-4 because of the building's enduring mission to our nation and its expected service life of at least another two or three decades.
3. The multimode pushover analysis advanced by SGH at the request of NNSA and DNFSB has not been verified to an extent that is consistent with nuclear standards. The method appears to provide conservatively biased estimates of demand for PF-4, which may be acceptable for code-type (Standards ASCE 4 and ASCE 43) analysis but not for a fragility or risk assessment. Nonlinear response-history analysis should be used for future fragility or risk assessments for complex structures such as PF-4.
4. We see no need to perform another vulnerability assessment of PF-4 at this time. The cost and effort associated with another vulnerability assessment should be directed towards the retrofit measures (interior roof girders on Line H and exterior girder on Line 8 between A and D) and the physical testing of the column capitals.

<u>Required Actions</u>

1. Inspect the tops of the columns, above the service chase floor slab and below the underside of the roof beams, on Lines D, E, K and L, to gage whether the columns have cracked in shear under gravity-induced loadings, and analyze the potential for shear failure of these columns and the resultant vulnerability of PF-4.

2. Inspect the sliding joint detail at a number of locations along each service chase to determine the current condition of the flexible material.

3. Understand and evaluate the effect of rotational restraint provided by the service chase corridor on the relative distribution of shearing force between the north and south ends of the exterior girders if the flexible material cast in the slab is found compressed. (If considerable restraint is identified, upgrades for shear resistance of the exterior girders near the service chases may be prudent.)

4. The seismic demands calculated by SGH and CJC are based on live loads significantly less than the design values. Those areas for which reduced live load has been assumed

must be posted and administered by LANL. The posted live loads must be no greater than the live load assumed by CJC and SGH for their analysis.

Prudent Actions

1. Increase the shear capacity of all interior roof girders on both sides of Line H by 120 kips using carbon fiber-reinforced polymer (CFRP) sheets, anchored to develop the capacity of the sheets as close as possible to the underside of the roof slab.

2. Increase the shear capacity of the exterior roof girder at Line 8 at Lines A and D by a minimum of 60 kips in a similar manner.

3. Conduct limited physical tests of representative slab-capital-column systems to simulate gravity and severe earthquake effects on the laboratory floor construction. As a minimum, a sufficient number of tests should be performed, at or near full scale, to characterize the performance of the Type V slab-capital-column assemblies. The testing program should provide the raw data and metadata needed to validate numerical models to the level of rigor that is standard in the nuclear industry. Earthquake shaking effects of at least 200% DBE shaking should be imposed on the test specimens to enable development of fragility functions for possible later use in a probabilistic risk assessment. If the specimens are badly damaged for the effects of 200% DBE shaking or less, retrofit strategies should be developed and implemented on virgin specimens to help guide LANL decision-making.

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We, the Seismic Expert Panel, could not have completed our work without the assistance of a number of expert engineers: Messrs. Lawrence Goen and Michael Salmon of LANL; Dr. Chuck Keilers of NNSA; Dr. Greg Mertz of CJC; and Drs. Said Bolourchi, Andrew Sarawit and Ben Deaton of SGH. We are extremely grateful to each and all.

Respectfully submitted,

Robert Kennedy; RPK Structural Mechanics
Brian McDonald; Exponent
Troy Morgan; Exponent
Loring Wyllie; Degenkolb Engineers
Andrew Whittaker; MCEER, University at Buffalo, State University of New York (Chair)

October 2, 2015

1 Introduction

1.1 Description of the Plutonium Facility, PF-4

1.1.1 Construction

Fluor Engineers and Constructors, Inc. designed the Plutonium Facility (PF-4) at the Los Alamos National Laboratory (LANL) in 1972-1973. The building is a one-story rectangular structure above a complete basement; the building was constructed of cast-in-place reinforced concrete, with small interior frames of structural steel. The plan dimensions of the building are 265’×284’. The overall height of the building varies between 39’-0” at the north and south ends, and 40’-6” at the center ridge. The south, east and western faces of the building are embedded to within one foot of the first suspended floor; grade on the north face of the building is at the basement level. The walls, foundation, floors and roof are made of cast-in-place reinforced concrete. Aerial photographs of PF-4 are presented in Figure 1.1.

Fluor Engineers and Constructors, Inc. constructed the building under contract to the U.S. Atomic Energy Commission. The design standard of record was the U.S. Atomic Energy Commission, *Minimum Design Criteria for New Plutonium Facilities*, dated June 1972. Structural design, load combinations, and construction of critical safety and fire protection features were prepared in accordance with the 1970 *Uniform Building Code* (ICBO 1970). Steelwork was designed in accordance with the 7th edition of the AISC *Specification for Design, Fabrication and Erection of Structural Steel for Buildings*. Reinforced concrete was designed in accordance with the 1971 ACI 318 *Building Code Requirements for Reinforced Concrete* (ACI 1971).

1.1.2 Gravity load path

A typical cross section through the building is presented in Figure 1.2. The roof consists of a 10” thick reinforced concrete slab supported on 6’-9” deep by 2’-0” wide roof girders. The roof slabs act one-way in the east-west direction between the north-south roof girders, with spans ranging between 17’-6” and 23’-2”. The roof girders span between 56’-6” and 61’-0” to 24”×26” columns, a 14” thick perimeter wall with 24”×26” integral pilasters, and a 12” thick interior wall.

Two east-west service chase corridors run the length of the building. Figure 1.2 presents one half of a north-south section through the building and identifies key structural features. The 10’ wide service chase is located between the ends of the roof girders supported on Lines D and E. The walls of the service chase act as beams spanning 17’-6” to 23’-2” in the east-west direction, between the 24”×26” columns. The 6” thick service chase floor slab spans one way in the north-south direction between the service chase walls. The 10” thick roof slab over the service chase also spans one-way between the service chase walls. Figure 1.3 presents information related to the construction of the service chases.

The roof slab was cast in a checkerboard pattern, presumably to minimize shrinkage effects. The roof slabs over the service chases were cast after the adjacent roof slabs. An axial slip detail, of unknown utility, was introduced into the service chase floor slabs, presumably to minimize their interaction with the roof girders framing into the service chases (left panel of Figure 1.3d).

The suspended first floor is a 10.5” thick two-way flat slab supported by columns spaced between 12’-2” and 21’-0” on center in the north-south direction and between 17’-6” and 23’-2” on center in the east-west direction. The floor slab is 15” thick in the primary corridors between Lines D and E and Lines K and L.

The columns supporting only the first floor are typically 18” diameter with spiral reinforcing at a 3” pitch and a 4’-6” diameter column capital. The 24”×26” columns have tie sets at 18” on center and a 5’-0” diameter column capital. The 24”×26” pilasters have tie sets at 18” on center. The pilasters that are integral with the walls were not constructed with column capitals. Figure 1.4 presents information on two column types: Type III/IV and Type V.

The columns are supported on spread footings, which were cast integrally with the 10” thick slab-on-grade. The perimeter and interior walls are supported by strip footings, which were also cast integrally with the slab on grade.

1.1.3 Lateral load path

Perimeter and interior reinforced concrete shear walls provide the primary load path for horizontal earthquake (and wind) effects. Loads are transferred to the shear walls by reinforced concrete diaphragms at the roof and first floor. The locations of the shear walls are identified in Figure 1.5. The relatively short shear walls around the basement rooms and stairwells are not shown.

The perimeter shear walls are 14” thick; the interior walls are 12” thick. The east-west wall on Line H, which extends from the basement to the roof, is penetrated by openings for doors and mechanical equipment. The north-south wall on Line 8 between the basement and the first floor is divided into three parts by corridors. The southern half (between Lines A and H) of this wall terminates at the first floor.

Lateral loads in the shear walls are transferred to the supporting soil through their strip footings and the basement slab-on-grade.

1.2 Seismic Hazard at the Los Alamos National Laboratory

Seismic design principles and procedures have changed substantially since PF-4 was designed and constructed in the early 1970s. Less was known at the time of construction regarding the importance of ductile detailing of reinforced concrete: the provision of closely spaced transverse reinforcement that radically improves the deformation capacity of structural components (e.g., beams, columns and walls) and substantially reduces the likelihood of collapse. The Seismic Expert Panel (SEP) considers the detailing of the reinforced concrete in PF-4 to represent best practice at the time.

The methods used to estimate seismic hazard, or the expected intensity of shaking for a given return period, have also changed substantially in the past 40 years. The profession has moved from deterministic to probabilistic seismic hazard analysis; our understanding of earthquake mechanisms is orders of magnitude better, our ground motion prediction equations are more robust, and we now treat uncertainty and variability in a mathematically sound way. Importantly, new active faults have been identified near LANL, and existing faults have been recharacterized, both of which have contributed to the significant increase in seismic hazard.

The change in seismic hazard at the site of PF-4 can be explained using acceleration response spectra. Since earthquake rupture generates horizontal and vertical earthquake shaking, both horizontal and vertical spectra are used to characterize the effects of earthquake shaking. Figure 1.6 presents the spectra used for the design of PF-4 in the early 1970s and for the evaluations performed in the past five years. Noting that the first mode horizontal and vertical frequencies of PF-4 are approximately 5 and 7 Hz, respectively, the horizontal and vertical seismic hazard has increased by factors of about 2.5 and 4.5 respectively. For this reason, the seismic vulnerability of PF-4 has been carefully studied in the past five years.

1.3 Seismic Upgrades to PF-4

Building PF-4 has been the subject of several seismic analyses, which identified deficiencies in the original construction, including shear-critical columns and incomplete load paths in the roof slab and the mezzanine steel framing. These deficiencies were addressed through seismic upgrades prior to the SEP being formed. The upgrade measures included a) the addition of a reinforced concrete drag strut atop the roof slab on Line 8, b) cutting columns free from adjacent basement masonry walls, increasing their shear spans, c) wrapping selected columns with carbon fiber-reinforced polymer to provide additional shear strength and confinement, and d) the addition of steel bracing to the mezzanine floors to complete lateral load paths. These upgrades were included in the mathematical models prepared for the seismic analyses of PF-4 that were reviewed by the SEP. Accordingly, they are described hereafter as part of the as-built construction.

1.4 Seismic Evaluations of PF-4

The Los Alamos National Laboratory (LANL) and the National Nuclear Security Administration (NNSA) retained Carl J. Costantino and Associates (CJC) and Simpson, Gumpertz and Heger (SGH), respectively, to perform seismic analysis of PF-4. The scope and objective of each analysis activity are documented in Sections 2 and 3 of this report, respectively, and are not summarized here. The scopes and objectives of the two studies were different and it was therefore not surprising to the SEP that the conclusions and recommendations made by CJC and SGH also differed.

The SEP commends CJC and SGH for the high quality of their work and their willingness to engage in vigorous discussion about their findings and recommendations. The Plutonium Facility is a mission-critical building whose function contributes significantly to our national security. Both CJC and SGH placed the importance of this facility and the need to understand its seismic vulnerability far above their corporate investments in the project. Both companies deserve our nation's thanks for a job well done.

The initial evaluation of PF-4 by CJC included a nonlinear static (or pushover) analysis wherein translational loads, representing the effects of horizontal and vertical earthquake shaking, were incremented from zero to values greater than those associated with design basis earthquake shaking. Such an analysis, at the time it was undertaken, represented the state-of-the-art in the US Department of Energy (DOE) complex. Nonlinear static analysis, first documented in FEMA 273 and FEMA 274 (FEMA 1997a, 1997b), was developed for seismic analysis of buildings typically used for commercial construction, whose translational response along each horizontal axis is dominated by the first mode along that axis, and for which vertical shaking effects were assumed to be of secondary importance. CJC extended this method for their analysis of PF-4 and used a horizontal and vertical loading profile based on calculated accelerations from a soil-structure-interaction analysis.

The Defense Nuclear Facilities Safety Board (DNFSB) questioned the use of a single loading profile because it could not reflect changing structural response associated with damage and softening with increasing intensity of loading. To identify the effects of changing loading profiles on the seismic response and vulnerability of PF-4, NNSA commissioned a follow-on study by SGH. The Deputy Secretary of Energy instructed NNSA to use a second modeling approach in the follow-on study, termed a modal loading analysis. The Deputy Secretary also instructed NNSA to consult with DNFSB to ensure that the technical basis for the analysis method, termed a multi-mode pushover analysis (MMPA) hereafter in this report, addressed the DNFSB perspectives. This analysis methodology had been proposed in the literature and studied as an academic exercise, but had never been applied in practice to a commercial building. Further, PF-4 is a low-rise building constructed with thick shear walls, making it (and its dynamic properties) much different from typical commercial buildings. SGH went to considerable effort to attempt to demonstrate that this method provided reasonable demands for a code-like evaluation of PF-4. It is the opinion of the SEP that SGH substantially advanced

MMPA for application to PF-4. We congratulate SGH for accepting the challenge provided by NNSA and DNFSB and advancing the analysis methodology to a stage well beyond that of a theoretical framework. However, we note that MMPA has not been formally verified for use on PF-4 or other building structures as would normally be required for nuclear facilities.

1.5 Formation of Seismic Expert Panel and Charter

Los Alamos National Laboratory (LANL) established the Seismic Expert Panel (SEP) to review and comment upon the seismic analyses of the LANL Plutonium Facility, PF-4. The charter is reproduced below and it was used as the basis for the SEP's deliberations.

> **Charter – Seismic Expert Panel Review of Plutonium Facility (PF-4) Seismic Analyses**
>
> **Purpose:** Establish a Seismic Expert Panel to review and comment upon recent analyses completed for the Los Alamos National Laboratory (LANL) Plutonium Facility (PF-4).
>
> **Background:** PF-4 is about 1 km from the nearest residential area. It met its 1970's design requirements; however, later seismic hazard studies have resulted in increased seismic loads at the site. PF-4 also does not have the redundant load-carrying pathways or the ductility expected in design of a new nuclear facility.
>
> In May 2011, a linear dynamic analysis determined that an earthquake with likelihood of once in hundreds of years could release plutonium from PF-4. By February 2012, LANL completed upgrades to reduce risk and began a nonlinear analysis to investigate marginal structural members, particularly columns. In September 2012, that analysis identified weaknesses from an earthquake with a likelihood of about one in eight thousand years. LANL updated its nonlinear analysis in 2013. The little margin exists between an earthquake that would cause loss of confinement and a slightly larger earthquake that could induce collapse. By March 2013, LANL began pursuing further upgrades to increase that margin.
>
> Also in September 2012, DOE/NNSA committed to the Defense Nuclear Facilities Safety Board (DNFSB) to perform an alternate "modal loading" analysis of PF-4 to confirm results of the original LANL analysis. Simpson, Gumpertz, and Heger (SGH) has been working with NNSA and the DNFSB staff on this alternate analysis. SGH issued a final report on Phase I of their analysis in September 2014.
>
> **Objective:** The Seismic Expert Panel will investigate the similarities and the differences in the SGH and LANL analyses and conclusions. The Panel's deliverables are a letter-report with comments, observations and recommendations, and a briefing to NNSA and LANL senior management.
>
> **Technical Sponsors:** The federal and LANL leads are C. H. Keilers (505-845-4280) and L. Goen (505-665-6847). They will coordinate with the Field Office (J. Krepps) and NNSA Headquarters (J. Serra).

General Work Instructions:

1. The Panel will internally assign a Chair, who will be primarily responsible for:
   a. Establishing the Panel's timeline,
   b. Coordinating with the technical sponsors, and
   c. Developing and issuing the letter report
2. The Panel will internally coordinate development of comments and observations, defined as follows:
   a. Comment: A point of concern that, if not addressed, has a direct impact on conclusions involving the adequacy of PF-4 to meet DOE STD-1020 expectations for an existing facility.
   b. Observation: A point of perspective that is unlikely to directly impact those conclusions.
3. Formal Panel deliberations will be coordinated with the technical sponsors. It is also anticipated that the Panel will need further briefings by SGH and LANL (including CJC) and that these will be limited efforts. Such briefings will also be coordinated through the technical sponsors above.
4. The Panel's deliverable will be a briefing to NNSA and LANL senior management and a consolidated letter-report. The report, comments, and observations should reflect the Panel's consensus; however, the report may include any minority opinions of Panel Members as the Panel deems appropriate.

The SEP was formed by LANL in October 2014 and comprised the following individuals: Dr. Robert Kennedy, RPK Structural Mechanics; Dr. Brian McDonald, Principal, Exponent; Dr. Troy Morgan, Managing Engineer, Exponent; Dr. Andrew Whittaker, Professor, MCEER, State University of New York, Buffalo; and Mr. Loring Wyllie, Senior Principal, Degenkolb Engineers. Dr. Whittaker chaired the SEP.

1.6 Performance Expectations for PF-4

The two performance-oriented standards used for the seismic evaluation of PF-4 are DOE Standard 1020 *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities* (DOE 2002), and ASCE Standard 43 *Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities* (ASCE 2005). Each document establishes target performance goals. DOE defined PF-4 to be a Performance Goal PC3 facility per DOE 1020-2002. In this Standard, the target performance goal is an Annual Frequency of Unacceptable Performance (AFUP) of 0.0001 (1×10^{-4}), where unacceptable performance is defined as loss of confinement. Performance is assessed component by component and not for the building as a whole, in large part because structural acceptance criteria (e.g., maximum plastic rotation) are applied at the component level.

Standard ASCE 43-05 allows for a broader range of unacceptable performance categorizations by defining four Limit States ranging from A (short of collapse) to D (essentially elastic behavior, or no damage). Performance goal PC3 in DOE 1020-2002 is equivalent to Seismic Design Basis SDB-3C (limited permanent distortions) in ASCE 43-05.

In both DOE 1020-2002 and ASCE 43-05, performance is assessed at the component (element) level and not for the building as a whole. To assess the annual frequency of failure for a building, the fragilities for the independent failure modes would have to be convolved together in a systems model using Boolean algebra cutset equations to develop a fragility function for the facility. In practice, the combination of individual component fragilities to develop a building fragility is very complicated because component fragilities are seldom either perfectly independent or perfectly correlated. For example, the AFUP of the roof girder and of the exterior shear walls could individually satisfy DOE 1020-2002 whereas the AFUP of the building might exceed the target performance goal because these two fragilities are partially uncorrelated. Standards DOE 1020 and ASCE 43-05 cannot address building fragility because component correlations are building specific.

The SEP notes that the target performance goals are simply that, targets for which to aim. Even though the mathematical procedure of convolving fragility curves with seismic hazard curves is rigorous, both the fragility curves and the seismic hazard curves contain many sources of judgment, uncertainty, and variability. A 20% difference between a computed AFUP and a target performance goal cannot be the sole basis for decision-making. On the other hand, a 30% reduction in seismic risk by facility upgrades represents a very meaningful reduction. Such a reduction is often justified on the basis of cost of risk reduction, particularly for facilities such as PF-4 with a long remaining service life.

Section 1.3 of DOE Standard 1020-2002 permits a doubling of the hazard exceedance probabilities for an existing facility (or halving of the return period of the shaking, say from 2,500 years to 1,250 years) but limits the reduction to 20% (Appendix B-3 of DOE 1020-2002). It is the SEP’s opinion that this relief for existing facilities was intended to avoid having to make costly upgrades that could not be justified on the basis of cost versus risk reduction, generally because of a short remaining service life. We do not believe this relief is appropriate for PF-4, which DOE plans to operate for many years.

The PC3 target performance goal for PF-4 is an AFUP of 0.0001 against loss of confinement. The 2002 DOE standard does not provide a target performance goal against collapse. It has generally been expected that the AFUP against collapse would be substantially smaller than the AFUP against loss of confinement. This expectation does not hold for the brittle failure modes (e.g., roof girder shear) that dominate the AFUP for PF-4. The dominant failure modes for loss of confinement are those that would lead to either a partial or a complete collapse of PF-4.

The recommendations of the SEP, presented in Section 6 of this report, are influenced by the performance expectations discussed above.

1.7 Meetings of the Seismic Expert Panel

The SEP met in person on three occasions: 1) November 4 and 5, 2014 in the DOE complex on the Kirtland Air Base, in Albuquerque, New Mexico, 2) February 22 and 23, 2015 in the office of Degenkolb Engineers, San Francisco, California, and 3) March 17, 2015 in the office of Degenkolb Engineers, San Francisco, California. Representatives from LANL, NNSA, DNFSB, CJC and SGH were present at the first two meetings. A representative from LANL (Mr. Goen) was present at the third meeting.

The SEP met via telephone on five occasions: 1) October 3, 2014, 2) January 20, 2015, 3) January 27, 2015, 4) February 3, 2015, and 5) February 12, 2015. Dr. Keilers of NNSA arranged the conference calls.

1.8 Organization of Report

This report is composed of seven sections and four appendices. Section 2 summarizes the seismic evaluation of PF-4 undertaken by Carl J. Costantino and Associates (CJC) for LANL in the period 2011 to 2013. Section 3 summarizes the subsequent evaluation of PF-4 undertaken by Simpson Gumpertz and Heger (SGH) for the National Nuclear Security Administration in the period 2013 to 2014. Section 4 describes the work of the SEP. Section 5 identifies studies that

LANL may wish to pursue. The recommendations of the SEP are summarized in Section 6. A list of references is provided in Section 7.

Appendix A provides a list of the documents made available by LANL at the start of the SEP review. Appendix B provides a list of the additional documents provided by LANL, CJC and SGH at the request of the SEP. The documents listed in Appendix B are provided in a companion electronic file *SEP-Documents.zip*. Appendix C provides the formal information requests made by the SEP, and Appendix D presents the SEP meeting minutes. Appendix E presents SGH, CJC, and SEP correspondence on the Revision 0 report dated March 31, 2015.

1.9 Revision 1 Report

Revision 0 of the SEP report was submitted on March 31, 2015. SGH and CJC were invited to review the report and provide comments to NNSA and LANL, respectively. The SGH and CJC letters are presented in Appendix E, which is new in this Revision 1 report. The SEP responded to each letter and these are also included in Appendix E. The SEP thanks SGH and CJC for providing timely input on the Revision 0 report.

The body of this Revision 1 of the report is updated to reflect the answers provided to SGH and CJC in the SEP letters of Appendix E. Specifically, a) the text of Section 3.4 has been replaced in its entirety to better explain the SEP's opinion on the utility of the SGH-developed multi-modal pushover analysis (MMPA) method, and b) Section 4.3.2 has been updated to correct a misstatement in the Revision 0 report regarding the calculation of roof girder shear capacity on Line 8.

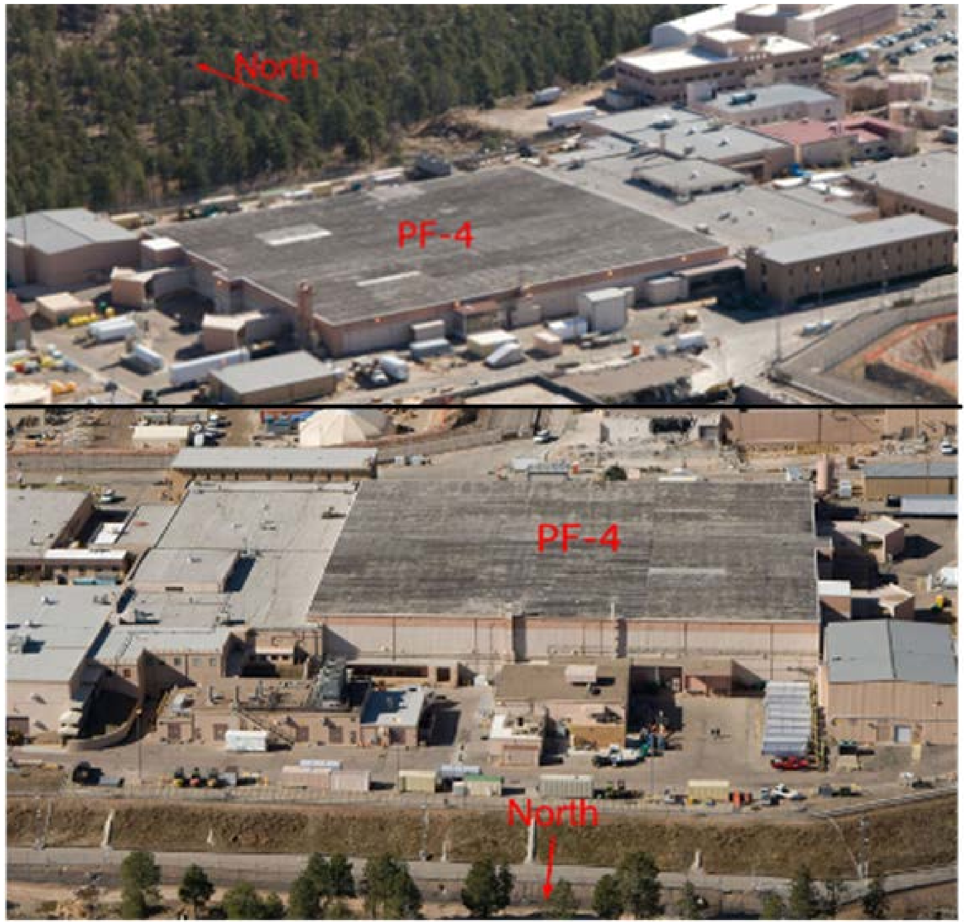


Figure 1.1. Aerial views of the Plutonium Facility, PF-4, circa 2010 (courtesy of LANL)

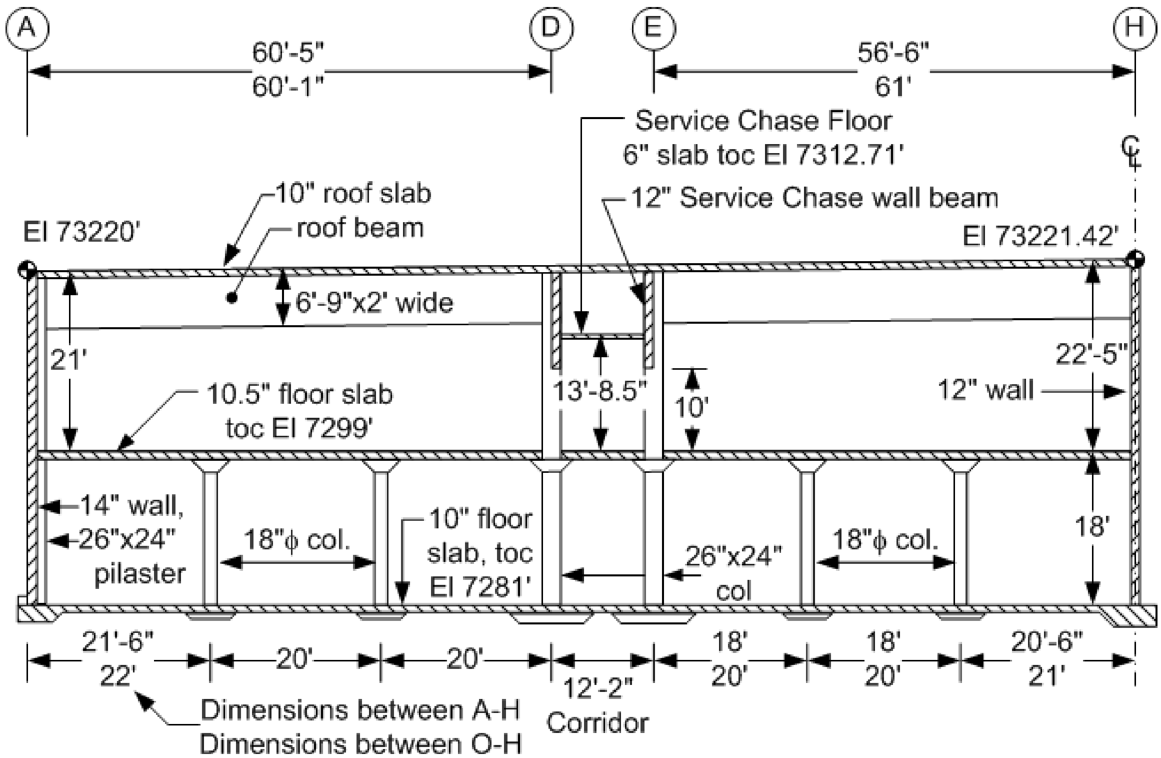
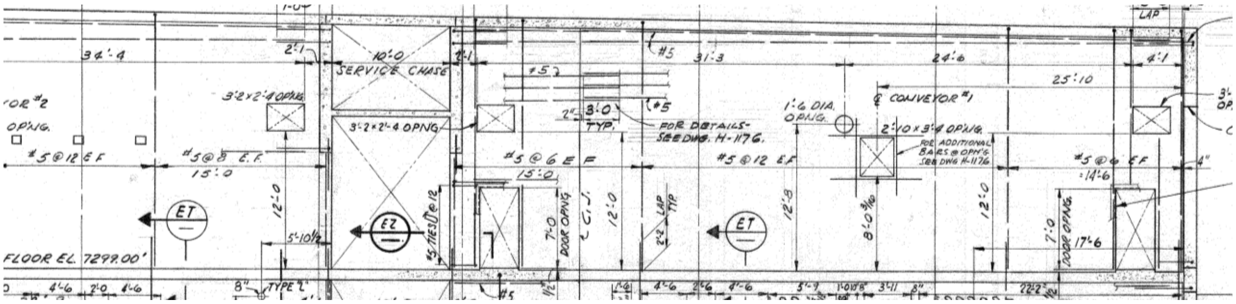
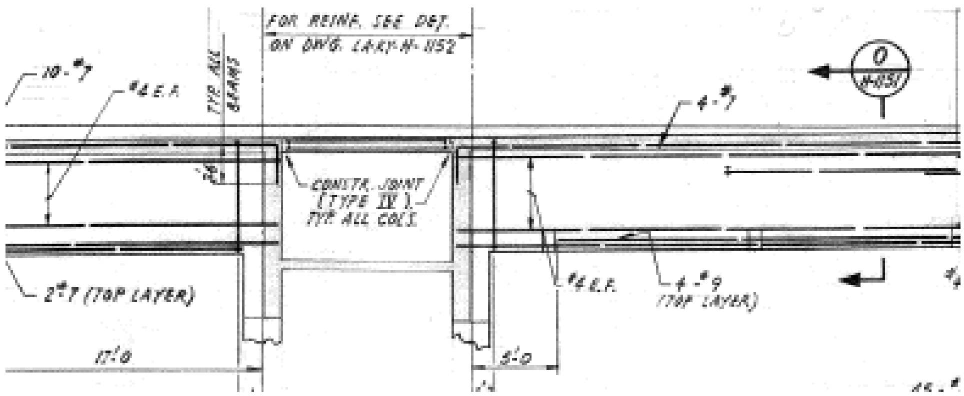


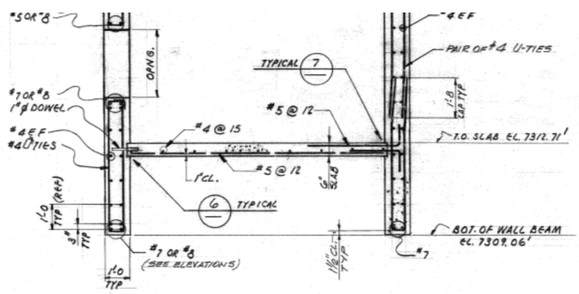
Figure 1.2. Part north-south cross section through PF-4 (courtesy of LANL)



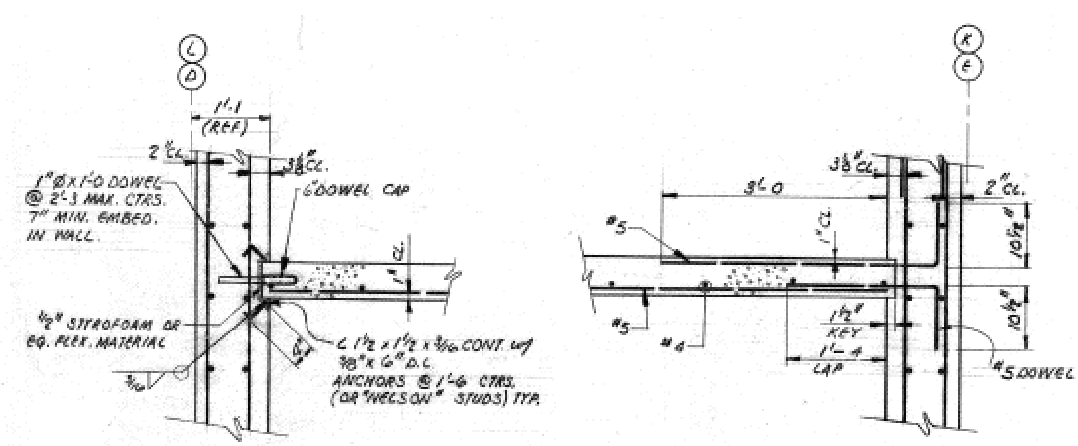
a. Part east elevation of interior wall on Line 8 (from drawing sheet 36)



b. Roof beam elevations on Line 8 (from drawing sheet 61)

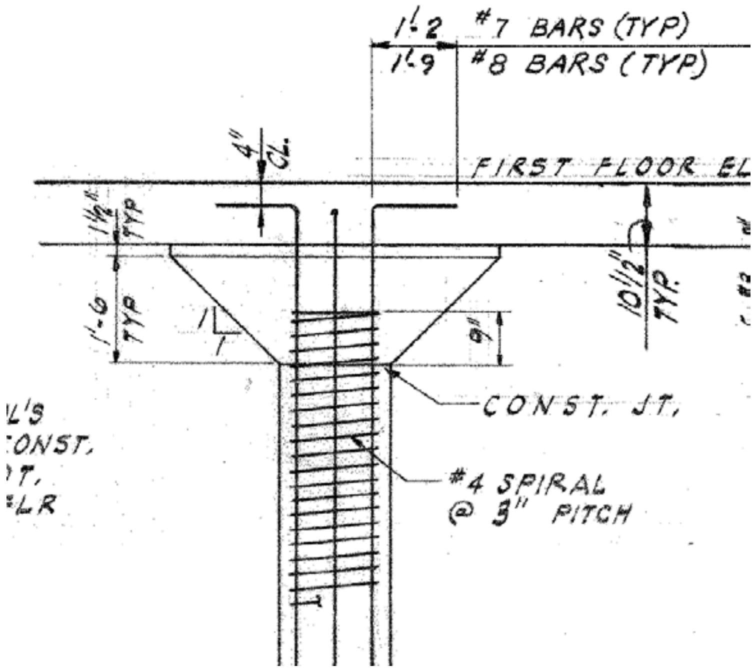


c. Typical section through the service chase (from drawing sheet 76)

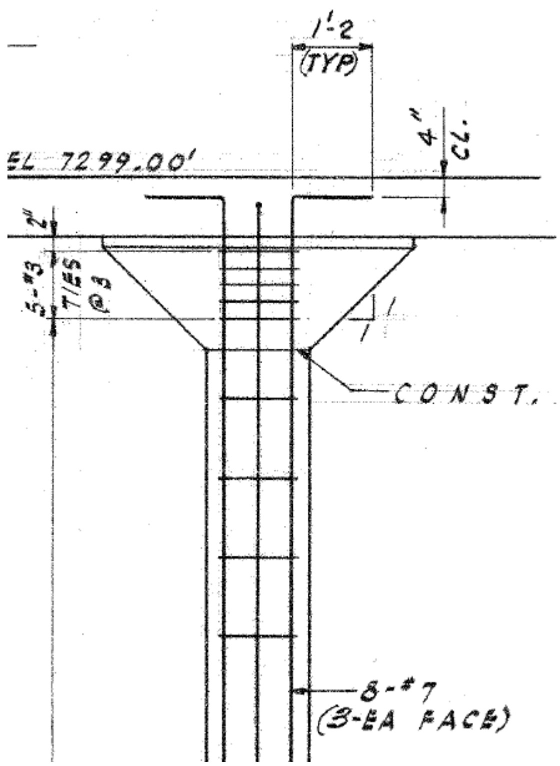


d. Floor slab to wall connection details in the service chase (from drawing sheet 76)

Figure 1.3. Chase floor slab connections (from Fluor 1977)

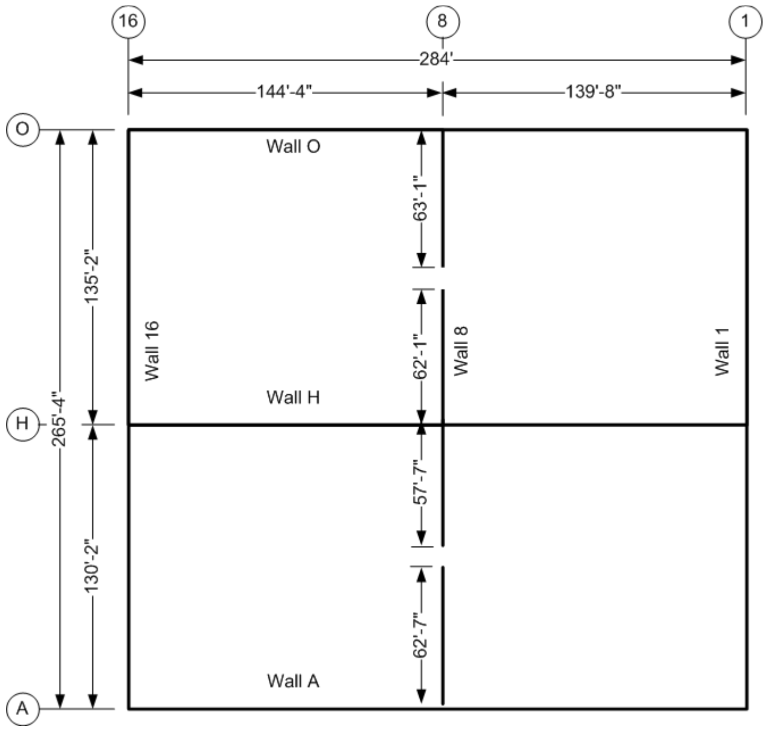


a. Column Type III/IV

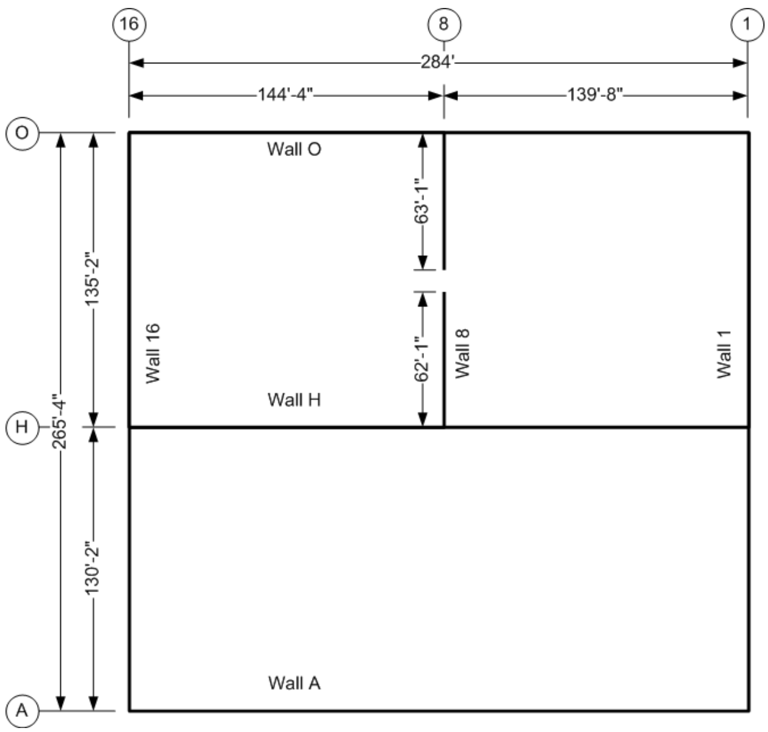


b. Column Type V

Figure 1.4. Typical PF-4 column details (from Fluor 1977)

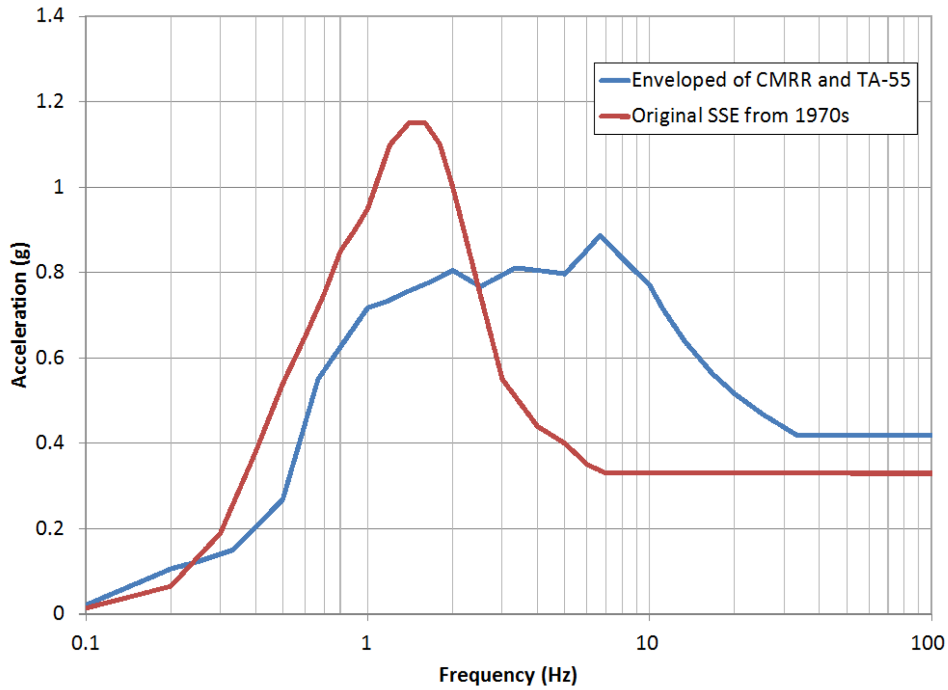


a. Plan view of shear walls between the basement and first floor slabs

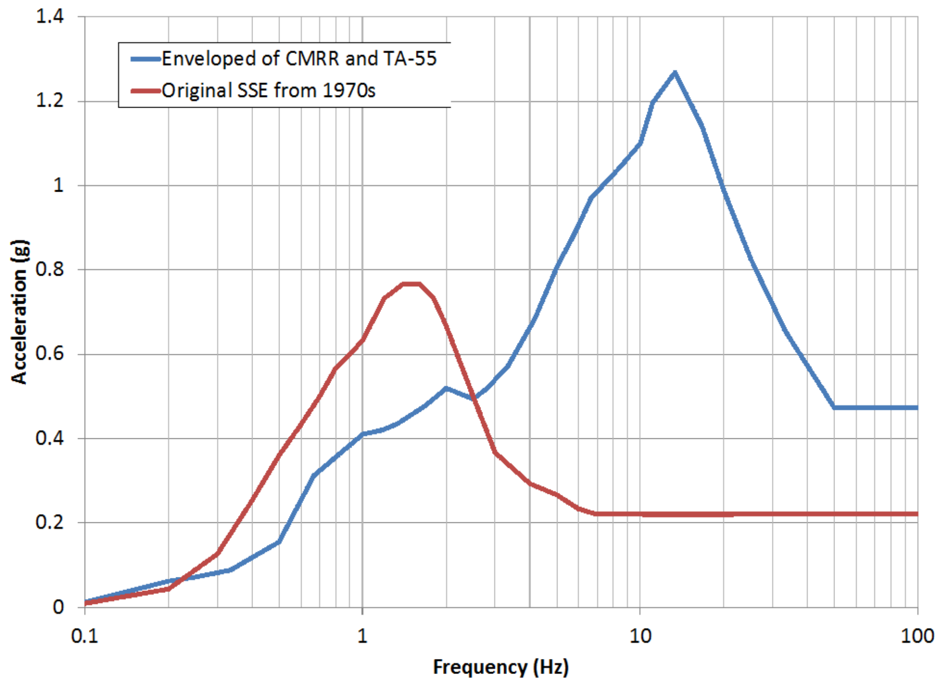


b. Plan view of shear walls between the first floor and roof slabs

Figure 1.5. Locations of shear walls in PF-4 (courtesy of LANL)



a. Horizontal acceleration response spectra



b. Vertical acceleration response spectra

Figure 1.6. Seismic hazard at LANL (SGH 2014a)

2 Seismic Evaluation of PF-4 by Carl J. Costantino and Associates

2.1 Charter

The re-evaluation of the seismic hazard at Los Alamos National Laboratory (LANL) in 2007 (Wong et al. 2007), with an update in 2009 (Wong et al. 2009), indicated more severe shaking than previously understood. Since the completion of these seismic hazard studies, the Plutonium Facility (PF-4) has been the subject of a series of analyses to characterize the performance of its structure, systems and components (SSCs) for the increased hazard. Facility PF-4 was reanalyzed using the 2002 edition of DOE Standard 1020 (DOE 2002), ASCE 4-98 (ASCE 1998), ASCE 43-05 (ASCE 2005), and other national consensus codes and standards as appropriate. Carl J. Costantino and Associates (CJC) performed these analyses. Dr. Robert Kennedy and Mr. Loring Wyllie reviewed the CJC work products on behalf of LANL. Both Dr. Kennedy and Mr. Wyllie are members of the Seismic Expert Panel (SEP) and are authors of this report.

2.2 Key Products

CJC performed two evaluations of PF-4: 1) a linear elastic analysis to establish demand-capacity ratios (Mertz et al. 2011), and 2) a nonlinear static analysis to establish annual frequencies of unacceptable performance (Mertz et al. 2012). Additional studies to address open issues in Mertz et al. (2012) were completed in 2013 (Mertz et al. 2013).

2.3 Findings and Proposed Upgrades

The 2011 evaluation of PF-4 used acceleration response spectra with an annual frequency of exceedance of 4×10^{-4} (return period of 2,500 years), which was appropriate for a PC3 facility per DOE 1020-2002. The evaluation identified some structural components with elastic seismic demand (D) in excess of *design* capacity (C), that is, $D/C > 1.0$. Design capacity was calculated per ACI Standard 349 (ACI 2006) for reinforced concrete components. For those elements with very high D/C, more refined analysis would not have shown that the performance goal would have been reached and recommendations for strengthening were made. Key results of the analysis are presented in Table 2.1; maximum values of demand-to-capacity are reported. Complete results are available in Mertz et al. (2011). The table identifies the proposed strengthening actions that reduced the demand-capacity ratios to less than 1.0.

Table 2.1 identifies, in italics, members for which D/C were greater than 1.0 and seismic strengthening was required. A probabilistic assessment using the results of nonlinear static analysis was subsequently performed by CJC; results are reported in Mertz et al. (2012, 2013). Table 2.2 provides key results of these analyses, with detailed information available in Mertz et al. (2012, 2013); the numbers in the first column of the table map to those in Mertz et al. (2013). In this table, C_{50} is the median capacity, β is the log standard deviation, and P_f is the probability of failure. (Code *design* capacity is assumed to be at approximately the 98%-ile, namely, that *design* capacity will be exceeded in 98 experiments of 100. The 98%-ile capacity and the 50%-ile (median) capacity are related through the log standard deviation: the greater the dispersion, the greater the percentage difference in the two capacities.) All structural components, when considered alone, met the PC3 performance goal. An assessment of the facility fragility, when component fragilities are assumed partially correlated, is provided in the last three rows of the table; numbers 12 and 13 identify combined failure modes with probability of failure greater than the performance goal of 1×10^{-4}.

ASCE 43-05 permits an alternative procedure for demonstrating compliance with a target performance goal, namely, show that there 1) is a 1% probability of unacceptable performance for Design Basis Event (DBE) shaking, and 2) is a 10% or less probability of failure for 150% DBE shaking. Results for selected components and combinations of components are presented in Table 2.3; cells highlighted in yellow identify non-compliance.

The performance of PF-4 per Table 2.3 was deemed adequate for an *existing* facility per DOE 1020-2002. DOE Standard 1020-2012 (DOE 2012) provides more prescriptive guidance for existing facilities than 1020-2002. Standard 1020-2012 permits seismic evaluation of structures, systems and components in existing facilities at twice the hazard exceedance frequency (i.e., 8×10^{-4} or a return period of 1,250 years) used for new facilities (i.e., 4×10^{-4} or a return period of 2,500 years), but limits the reduction in seismic demand to 20%. For the site of PF-4 and a natural vibration frequency of 5 Hz (an appropriate value for the building), the reduction in seismic hazard by doubling the hazard exceedance frequency is approximately 30%, which exceeds the limit of 20%. Applying the 20% reduction to DBE and 150% DBE shaking (i.e., shaking at 0.8DBE and 1.2DBE), the conditional probabilities of failure fall below the ASCE 43-05 thresholds of 1% and 10%, respectively.

Facility PF-4 did not meet the performance goals for a new facility per DOE 1020-2012 and its referenced codes and standards. In response, LANL performed the following upgrades based on the CJC recommendations:

- Added a drag strut to the roof on Line 8
- Braced the steel mezzanines
- Cut the basement CMU walls back from concrete columns
- Strengthened the connections of the firewalls to the roof
- Wrapped the basement Type V captured columns with FRP.

Table 2.1. Actions resulting from seismic performance assessment of PF-4 (from Mertz et al. 2011)

Item	D/C[1,2]	Resulting action
Lateral load path		
Roof girder, E8-H8	~~1.70~~[3]	Drag strut added to provide a load path from roof diaphragm to shear wall
Shear wall on Line 8, in-plane	1.08	Permitted inelastic energy absorption factor of 1.5 (>1.08); no action required
Shear wall on Line H, out-of-plane	0.93	Mezzanines strengthened such that wall support not needed
Laboratory floor diaphragm, shear	0.71	None
Vertical load path		
Roof slab, flexure	0.80	None
Roof girders, flexure and tension	1.00	Permitted inelastic energy absorption factor >1.0; no action required
Laboratory floor slab, flexure	1.17	Permitted inelastic energy absorption factor of 1.25 (>1.17); no action required
Laboratory floor slab, shear	0.86	None
Columns at laboratory floor level supporting roof girders; Type II columns	1.44	*Columns investigated further in the follow-on analysis*
Miscellaneous		
CMU rooms 4, 8, 13	1.64	Roof beam-to-column connections reconfigured and strengthened
Mezzanine 2, 5-8	8.00	Mezzanines strengthened
Ceiling support steel	2.41	Bracing added
Firewalls	1.38	Firewall connections to the roof strengthened
Basement columns embedded in walls; Type V columns	6.00	*Columns investigated further in the follow-on analysis*
Basement columns engaged with CMU walls		Columns cut free from the CMU walls

1. Elastic demand divided by code design capacity

2. Maximum value reported for the class of components.

3. Number ~~abc~~ is the demand-capacity ratio before strengthening.

Table 2.2. Selected results of PF-4 nonlinear analyses and fragility evaluation (from Mertz et al. 2013)

No.	Failure Modes	C_{50}/D	β	$P_f \times 10^4$
1	Shear failure of interior roof girder on Line 8 leading to collapse	2.94	0.53	0.9
2	Tension-shear failure of interior roof girders leading to collapse	3.07	0.50	0.8
3	Shear failure of exterior roof girder	3.80	0.53	0.5
4	Captured Type V column exceeds rotation limit and possible collapse	3.07	0.29	0.5
6	Punching shear failure of laboratory floor slab leading to collapse	4.1	0.30	0.3
12	Combined fragility of lateral and vertical failure modes	2.33	0.37	1.2
13	Combined fragility of vertical mechanisms	2.67	0.46	1.0
14	Combined fragility of lateral mechanisms	3.07	0.29	0.5

Table 2.3. Conditional probabilities of failure (from Mertz et al. 2013)

No.	Failure Mode	100% DBE	150% DBE
1	Shear failure of interior roof girder on Line 8 leading to collapse	2.1%	10%
2	Tension-shear failure interior roof girders leading to collapse	1.2%	8%
3	Shear failure of exterior roof girder	0.6%	4%
4	Rotation limit of captured Type V column exceeded and collapse	<0.1%	0.7%
7	Axial load resistance of captured Type V column lost and collapse	<0.1%	0.4%
12	Combined fragility of lateral and vertical failure modes.	2.2%	12%
13	Combined fragility of vertical mechanisms	2.2%	11%
14	Combined fragility of lateral mechanisms	<0.1%	0.7%

3 Seismic Evaluation of PF-4 by Simpson Gumpertz and Heger

3.1 Charter

The National Nuclear Security Administration (NNSA) funded an independent evaluation of the Plutonium Facility (PF-4) by Simpson, Gumpertz and Heger (SGH) to understand the influence of structural softening and damage on its seismic performance. The evaluation was to use "…a second modeling approach, termed a modal loading analysis" (Poneman 2012). Phase 1 of the evaluation involved a code-type assessment of PF-4 using the surface free field spectra for design basis earthquake shaking adopted by Carl J. Costantino and Associates (CJC) that was described in Section 2. Phase 2 of the evaluation was to perform a fragility assessment of the facility using the analysis methodology developed as part of Phase 1. Only the Phase 1 study has been completed. Independent Review Panel members Brian McDonald, Troy Morgan, and Andrew Whittaker reviewed the SGH work products on behalf of NNSA. Drs. McDonald, Morgan and Whittaker are members of the Seismic Expert Panel (SEP) and are authors of this report.

3.2 Key Products

Options for the nonlinear dynamic analysis of PF-4, which would permit inclusion of structural degradation and higher mode effects, were 1) nonlinear dynamic analysis, and 2) an adaptive multi-mode pushover analysis (MMPA). Given that the DNFSB would not support option 1, option 2 was adopted by SGH. The MMPA was an unproven and untested methodology but with potential advantages over the more traditional, single-mode nonlinear static (pushover) procedure, which was first documented in FEMA 273/274 (FEMA 1997). One potential advantage is that the applied loading reflects many potentially uncoupled, simultaneously activated vibration modes. Second, the methodology addresses changes in the structural response due to damage and associated strength and stiffness degradations. A key disadvantage is that it is difficult to quantify any inherent bias (either conservative or not) in the analysis. SGH compared sample results of analysis using the MMPA to those of response-history analysis, for simple planar structures, and concluded that MMPA demands were at approximately the 80%-ile and thus suitable for the code-type evaluation per ASCE Standard 4.

SGH prepared a series of three comprehensive reports summarizing the results of their seismic evaluation of the PF-4 facility. The first report (SGH 2014a) described the MMPA method used to determine demands on the structure and its components. Because this approach to nonlinear analysis has been reported in the academic literature and applied to simple frame-type structures, but had not seen practical application (to the knowledge of either SGH or the peer reviewers), SGH conducted verification exercises for the project-specific application to PF-4, which are summarized in their first report.

The second report (SGH 2014b) summarized design inputs for their code-type evaluation, described gravity and seismic load paths, characterized the behavior of critical members constituting those load paths, and developed and validated component models for use in pushover analysis.

The third report (SGH 2014c) presented the global model attributes, component capacities, and the results of the code-like evaluation expressed as demand-capacity ratios for critical elements. The third report also presented SGH's detailed evaluation of column capitals, and their recommendation for further studies.

3.3 Findings and Proposed Upgrades and Actions

SGH invested considerable effort in the development of the MMPA methodology and much of their work was devoted to this fundamental task. SGH claimed that seismic demands computed using the MMPA for PF-4 were approximately at the 80%-ile and thus appropriate for an evaluation per ASCE 43-05 (ASCE 2005). Component capacities were established at the *design* level (the level adopted by CJC) per ACI Standard 349 (ACI 2006). Component capacities in shear were also calculated using a more modern theory that is presented in the *AASHTO LRFD Bridge Guide Specification* (AASHTO 2013).

Table 3.1 presents results of the SGH code-type analysis of PF-4 in terms of demand-capacity ratios (*D*/*C*). Maximum values of *D*/*C* are reported for selected components. The upgrades to PF-

4 recommended by CJC (see Section 2.3) that had been implemented at the time of their work were reflected in the SGH analysis model.

The column capitals at the tops of the columns supporting the laboratory floor slab are unreinforced and there is very limited test data on their integrity during severe horizontal and vertical earthquake shaking. Figure 1.4a and Figure 1.4b present sections through two of the unreinforced capitals. Failure of the unreinforced capitals would likely lead to punching shear failure of the floor slabs under gravity loads and so their integrity must be maintained. SGH recognized the importance of the column capitals to the seismic integrity of PF-4 and invested significant effort in finite element analysis of the slab-capital-column assembly (Figure 3.1). Alternate constitutive models for concrete and bond of reinforcement to concrete, and a range of concrete tensile strengths were investigated with the finite element analysis code DIANA (TNO DIANA 2014) to envelop the likely response of the slab-capital-column systems. SGH sought to validate the DIANA model with data from one relevant test performed by To and Moehle (2012): a complex and challenging undertaking rarely pursued in either the academic or consulting domains.

On the basis of the code-type analysis, SGH recommended the upgrades proposed in Table 3.1 and that the column capitals be studied further to better characterize their behavior. Subsequent to the issue of SGH (2014c), additional analysis by SGH showed that a) the demand-capacity ratios for the laboratory floor slab, per results listed in Appendix B and assuming the column capitals remained intact, were less than 1.0 (and thus acceptable), and b) the demand-capacity ratios for the soil beneath the walls were less than 1.0, per results listed in Appendix B and thus acceptable.

3.4 Unresolved Items

Although SGH made substantial progress in the development of the MMPA method, for which they are to be congratulated, the reviewers did not accept the technical basis of the proposed method.

This is not a criticism of SGH's implementation of the method for their Phase 1 evaluation. Rather, it is simply a fact that the Multi-Mode Pushover Analysis (MMPA) has not yet been vetted by the profession, and does not have a pedigree of successful application required for general acceptance. The SEP was not tasked with verifying the MMPA method. The SEP (Kennedy, McDonald, Morgan, Whittaker, Wyllie) relied on a prior determination by an Independent Review Panel (IRP: McDonald, Morgan and Whittaker) tasked by DOE to review the Phase 1 SGH study: a code-type evaluation of PF-4.

The IRP evaluated SGH's implementation of the MMPA for Phase 1 investigation and found that it provided reasonable results for the PF-4 structure subject to code-level shaking at that site. The IRP was unable to independently verify the MMPA method in general, and indeed, the IRP was never tasked with doing so. In their Phase I study, SGH compared the results of analysis of simple models (by comparison with a numerical model of PF-4) performed using nonlinear dynamic procedures and MMPA. In the elastic range of response, displacements and forces calculated using the two methods were similar, with MMPA generally predicting slightly greater demands. Given that the MMPA appeared to overpredict elastic response, as one might expect from a procedure based on response-spectrum analysis, the IRP could raise no objection to its use for a code-type analysis of PF-4 for which response to design basis shaking was by-and-large elastic. The IRP did not endorse the use of MMPA for analysis of a) structures other than PF-4, and b) the response of PF-4 further into the nonlinear range. Several letters were issued to NNSA cautioning against the general use of MMPA, and the IRP comment log contains several open items establishing their concern that MMPA could not produce results that would be useful in a fragility analysis.

The technical basis of the MMPA is not yet proven. We are not suggesting it cannot be proven but have yet to see the comprehensive vetting that would elevate the method to a status sufficient for its widespread adoption by the structural engineering community in the United States. We note that the MMPA has not been rigorously peer reviewed by others and not used by expert design professionals other than SGH.

The SEP spent considerable effort reconciling the demands and capacities of the independent investigations by SGH and Carl J. Costantino and Associates (CJC). After accounting for differing assumptions and modeling techniques, it appeared that the demands calculated by SGH using MMPA were higher than the 80%-ile loads calculated by CJC (see Chapter 2) using more traditional methods. The greater demands (bias) are attributed in part to the MMPA method itself. Systematic quantification of that bias for the PF-4 structure would take considerable effort outside the scope of the SEP, and the IRP and SEP understood that to be a task contemplated for the Phase 2 investigation, if undertaken.

At least five challenges remain with the use of the MMPA for the nonlinear analysis of PF-4 and other building structures: 1) verification of the underlying mathematics, 2) peer review of the MMPA by the design professional community, 3) the need to produce site-specific displacement amplification factors 4) quantification of the bias in the MMPA with respect to results of nonlinear dynamic analysis, which is likely specific to the building and its siting with respect to active faults, and 5) lack of prior use by other expert design professionals.

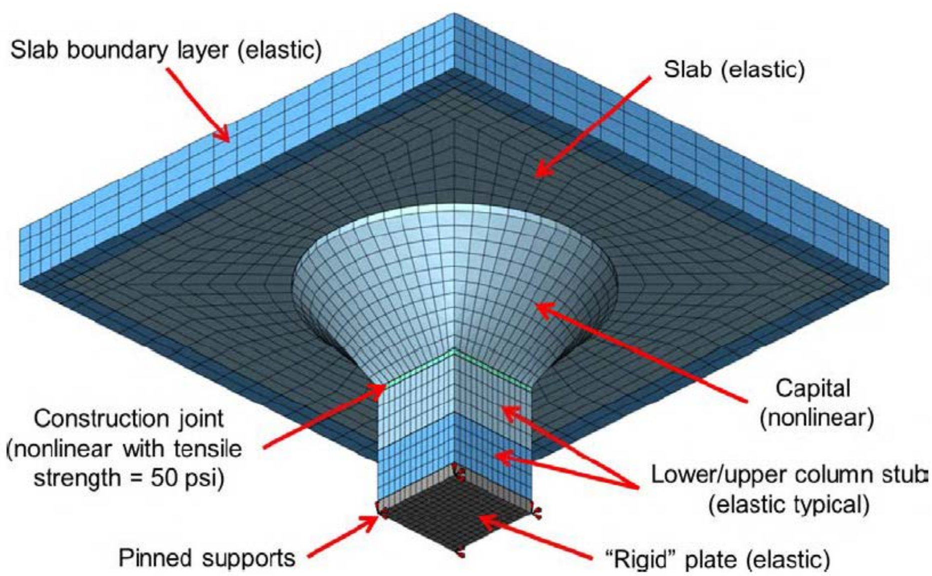
Table 3.1 Actions resulting from seismic performance assessment of PF-4 (from SGH 2014c)

Item	*D/C*	Required action
Exterior roof girders, shear	1.01	Strengthen in shear using FRP wrap
Interior roof girders, shear	1.26	Strengthen in shear using FRP wrap
Exterior shear walls	0.49	None
Interior shear walls on Line H, basement	0.21	None
Interior shear walls on Line H, lab floor	0.19	None
Interior shear walls on Line 8, basement	0.52	None
Interior shear walls on Line 8, lab floor	0.60	None
Laboratory floor slab, punching shear	1.02[1]	Further analysis or upgrade
Laboratory floor slab, column capitals	*	Nonlinear finite element analysis could not rule out possible failure – further evaluation required
Foundations beneath exterior walls	~2.4[2]	Overpressure is localized, and further calculation or local upgrade recommended

1. Subsequent analysis reported a peak value of demand-to-capacity of 0.94, with no action required; see Appendix B.

2. Subsequent analysis reported a peak value of demand-to-capacity of 0.5, with no action required; see Appendix B.

a. as-built column capital (courtesy of LANL)



b. SGH finite element model of a column capital (SGH 2015)

Figure 3.1. PF-4 Type V column capital and finite element model

4 Seismic Expert Panel Review of PF-4 Documents

4.1 Introduction

Although many aspects of the Carl J. Costantino and Associates (CJC) and Simpson, Gumpertz and Heger (SGH) evaluations described in the previous sections were in reasonably good agreement, there were some important differences. Key tasks assigned to the Seismic Expert Panel (SEP) by Los Alamos National Laboratory (LANL) were 1) identify and understand the differences, 2) determine the specific technical reasons for the differences, and 3) resolve the associated recommendations. In this section we describe our evaluation of each key difference. Section 4.2 addresses the high soil bearing pressures reported by SGH. Section 4.3 resolves the differences in girder end shear demand-capacity ratios. Section 4.4 evaluates column punching shear results. Section 4.5 addresses the vulnerability of column capitals at the underside of the laboratory floor slab. We requested supplemental linear elastic analyses to assist us in identifying and quantifying the effects of different modeling assumptions used by SGH and CJC and those results are discussed in Section 4.6.

Appendices A and B provide lists of the documents developed by CJC and SGH that were transmitted by LANL to the SEP. Those documents provided in response to SEP requests (Appendix B) are included in a companion electronic attachment: *SEP-Documents.zip*

4.2 Bearing pressure

4.2.1 Similarities and differences

As discussed in Section 3.3, SGH determined that the earthquake-induced bearing pressures below the exterior shear walls were higher than the allowable value recommended by Dames and Moore (1972). The bearing pressure distributions reported by SGH indicated that a substantial portion of the peak pressure under the wall footing was due to out-of-plane rotation that peaked at the pilasters. The bearing pressures below the interior walls and columns were less than the allowable value and not of concern. SGH concluded that soil failure beneath the perimeter walls was a concern for PF-4 if it was subjected to PC3 earthquake shaking.

CJC did not identify soil failure to be a concern for PC3 shaking. They modeled the connection of the wall/pilaster to the strip footing as a pin, transferring no moment to the strip footing. At the November 2014 meeting of the SEP, CJC reported that, by their calculation and review of the original Dames and Moore soils report, the allowable bearing pressure should be much greater than originally recommended.

At our request, SGH re-evaluated the wall/pilaster/foundation/slab-on-grade/soil system. Results are presented in file *Tasks-2b PF-4 Foundation 2-27-2015.pdf* (included in the companion electronic document SEP-Documents.zip). The evaluation included a 2D (plane strain) finite element analysis. Based on these analyses, SGH concluded that bearing pressures at the exterior walls, after accounting for the flexural stiffness of the slab-on-grade and the soil overburden acting on the toe of the wall, were less than the allowable value recommended by Dames and Moore (1972), with a factor of safety against bearing failure of at least 2.0.

4.2.2 Findings of the SEP

We concur with the findings of the supplemental investigation by SGH. Differences remain between CJC and SJH regarding the most appropriate value for soil bearing capacity at PF-4. We did not attempt to resolve that difference because it would not have affected our recommendations.

4.2.3 Recommendations of the SEP

The difference in the CJC and SGH recommendations regarding soil pressure has been resolved. Soil failure is not a concern for PC3 shaking at PF-4. No further action is required.

LANL may wish to formally review values of allowable soil bearing pressure for gravity and seismic design. In-situ tests (e.g., borings, load tests) may be needed to support updates to the 1972 recommendations.

4.3 Roof Girder End Shear

4.3.1 Introduction

Section 4.3.2 identifies the similarities and differences between roof girder end-shear demand and capacity results computed by CJC and SGH. Section 4.3.3 presents our findings. Section 4.3.4 summarizes the most recent calculations of the annual frequency of unacceptable performance (AFUP) prepared by CJC and presented in file *beam-fragility-study.pdf* (included in the companion electronic document *SEP-Documents.zip*). Section 4.3.5 presents our recommendations for action regarding roof girder shear failure.

The most recent demand-capacity ratios (D/C) reported by CJC and SGH differ somewhat from prior results. Table 4.1 and Table 4.2 present the current SGH and CJC shear demands, shear capacities and D/C ratios for the interior and exterior girders, respectively.[1] (File names referenced in footnotes refer to files found on the companion electronic file *SEP-Documents.zip*) The 80% non-exceedance probability (NEP) seismic demands were calculated by CJC using accelerations from a soil-structure interaction analyses, and are denoted hereafter as $D_{80\%}$ shear demands (gravity and seismic, V_G+V_S). The ACI-349 *design* capacities (C_{ACI}) were calculated as $\phi V_{n,ACI}$. The seismic demands computed by SGH were computed using their multi-mode pushover analysis (MMPA) procedure and these are denoted hereafter as D_{MMPA}. SGH reported both ACI-349 code capacities (C_{ACI}) and the design capacities calculated using the AASHTO (AASHTO 2013) implementation of the Modified Compression Field Theory (C_{MCFT}). For the purpose of this report, comparisons are based on the ACI design capacities (C_{ACI}).

Results are presented in Table 4.1 and Table 4.2 for interior girders between Lines E and H (the EH girders) and the exterior girders between Lines A and D (the AD girders), respectively. Similar results were obtained for interior girders HK and exterior girders LD. In these tables, shearing demands are reported at a distance d (the effective depth of the cross section) from the inboard face of the supports. The results presented in these tables are used as the basis for the following discussion.

[1] *SGH and CJC reporting of roof girder shears and lab floor shear stresses 2015-02-07 (2).xlsx* (Appendix B)

4.3.2 Similarities and differences

The CJC and SGH calculations of code shear capacities are very similar, with the ratio of CJC and SGH capacities ranging between 1.00 and 1.03 for 49 of the 56 reported values. In five of the remaining seven cases, the ratio ranged between 1.06 and 1.13, with only minor differences in how these capacities were computed. Either the SGH or the CJC shear capacities could be used without influencing our recommendations that are presented in Section 4.3.5.

One significant difference in shear capacity is at the north end of the interior girder EH on Line 8 for which the CJC capacity of 443 kips is 1.55 times the SGH capacity of 288 kips. Since issuing their third report, SGH has revised their girder end capacities by refining the way that axial tension is incorporated. The MMPA method relies on modal combination of spectral responses, and there is no way to rigorously track tension and compression in the girders. Based on this new (more conservative) method, the axial tension in the Line 8 girder is much greater than in any of the others.[2]

The typical SGH-computed shear capacity of the roof girders on Line H is approximately 320 kips. The calculated tensile load in the girder on Line 8 reduces this shear capacity to 288 kips. The SGH capacity on Line 8 conservatively ignores any strength contribution from the new drag strut. The CJC capacity for the girder on Line 8 at H (443 kips) is greater than the adjacent girders because this calculation utilizes the reinforcing steel in the drag strut to calculate ρ_w for inclusion in Equation (11-5) of ACI 349. The reported CJC capacity was not reduced for axial tension on Line 8. (We note that CJC provided the Line 8 girder capacity of 443 kips to help the SEP reconcile differences between CJC and SGH results.) Thus the SGH capacity at Line 8 is decreased (from the typical girder) to account for tension in this girder, while the CJC capacity is increased (relative to typical) due to the steel reinforcement in the drag strut above this girder: the CJC capacity is 1.55 times the SGH capacity. For a code-like evaluation, we believe that the SGH calculation of capacity is likely conservative since it ignores the contribution of the drag strut and the CJC calculation is likely high because it includes the drag strut longitudinal rebar in a code equation that is intended for situations without axial tension. For a code-like evaluation and for the purpose of reconciling the independent evaluations, the SEP recommends that the capacity be taken as 322 kips, which ignores any contribution of the drag strut (cross section or

[2] *Task 1d - GirderAxialStudy_01_09_2015.pptm* (Appendix B)

longitudinal reinforcement) to the shear capacity and assigns the axial tension on Line 8 to the drag strut only.

A similar agreement does not exist between the CJC and SGH reported demands. There are three major causes for the difference. Each is discussed below.

4.3.2.1 Difference in total shear demand

The total shear demand $\sum V$ between the supports of the girders is given by:

$$\sum V = V_{North} + V_{South} \tag{4.1}$$

where V_{North} and V_{South} are the shear forces calculated a distance *d* from the north and south supports, respectively.

Table 4.3 presents the total shear demand, $\sum V$, from the CJC and SGH analysis presented in Table 4.1 and Table 4.2 for both V_G (gravity) and V_S (seismic). The total gravity shear $\sum V_G$ computed by CJC ranges between 1.00 and 1.18 times the total gravity shear computed by SGH, with an average ratio of 1.08. Both estimates of $\sum V_G$ are considered to be reasonable and the differences are not sufficiently large to influence our recommendations.

Significant differences do exist in the calculated values of total seismic shear $\sum V_S$ for the interior girders EH. The CJC $\sum V_S$ range from 0.50 to 1.18 times those computed by SGH, with a mean ratio of 0.74. On average, the SGH $\sum V_S$ exceeds the CJC $\sum V_S$ by a factor of 1.35 for these interior girders. A large difference was not seen in the total seismic shear computed for the exterior girders AD. The CJC $\sum V_S$ range from 0.84 to 1.16 times those computed by SGH, with a mean ratio of 0.95; on average, the SGH $\sum V_S$ exceeds the CJC $\sum V_S$ by a factor of 1.06 for these exterior girders. The large differences between the SGH and CJC total seismic shears for the interior girders EH prompted a detailed review of both sets of seismic demand calculations.

The probabilistic $D_{80\%}$ seismic demand calculations performed by CJC were relatively straightforward to review because their model was easily understood. The median values and dispersions used for all parameters (inputs) in the probabilistic demand analyses appeared to be reasonable. The resulting median seismic demands ($D_{50\%}$) and 80% NEP seismic demands

($D_{80\%}$), both outputs, all seemed reasonable. On this basis, we judged that the CJC $\sum V_S$ on the interior girders EH were reasonable estimates of $D_{80\%}$.

Our review of the SGH calculations of seismic demand D_{MMPA} was more challenging. The structural model was more detailed than the CJC model. In addition, the MMPA method used by SGH to compute D_{MMPA} is rather opaque and thus very difficult to independently review and confirm. However, and with considerable assistance from SGH, we identified sources of conservatism in the SGH-computed seismic demands, which are explained below.

First, SGH used 4% modal damping for their response analyses. This value was chosen because hysteretic energy dissipation was included explicitly for members that exhibited significant nonlinearity due to cracking. However, SGH reduced the vertical stiffness of the roof girders to account for flexural cracking. No additional hysteretic energy dissipation (damping) appears to have been included to account for flexural cracking of the roof girders, and their analysis results indicated that the these girders remained essentially linear. In their probabilistic demand analyses, CJC treated damping as an uncertain variable with a median value of 7% and a logarithmic standard deviation consistent with a value at the 84%-ile of 4%. Each of the 32 probabilistic based seismic analyses used a different but equally probable value for damping. As a result, five of these 30 trials used damping values less than or equal to 4%. The use of a median value for damping of 7% is consistent with the recommendations of ASCE Standard 4-98 (ASCE 1998), ASCE Standard 43-05 (ASCE 2005), and USNRC Regulatory Guide 1.61 (USNRC 2007) for linear analysis of members that exhibit flexural cracking. In our judgment, the use of 4% damping introduces a 10% to 15% conservative bias in the SGH-computed demands above the $D_{80\%}$ seismic demands that would result from following the approach taken by CJC and treating damping as an uncertain parameter.

A second source of conservatism in the SGH-computed seismic demand was introduced by the use of vertical soil springs beneath the columns and walls in the SGH model. Whereas SGH modeled foundation compliance using discrete springs, CJC modeled the soil-structure system. The use of discrete soil springs increased the seismic demands on the roof girders with respect to a fixed-base model. The explicit consideration of soil-structure-interaction (SSI) reduced demands with respect to a fixed base model. A sensitivity study presented to us by SGH (Study 3) indicated that the soil springs increased the seismic shears on the interior girders EH by an average amount of about 15% with respect to a fixed-base model.[3] Soil-structure-interaction analysis accounts for radiation of energy away from a structure back into the soil, which generally reduces seismic demands with respect to a fixed-base model. CJC presented sensitivity results to us that indicated that the SSI-computed seismic demands on the interior were approximately 10% less than those from a fixed-base analysis.

SGH presented two sensitivity studies (Study 1 and Study 6) from which they concluded that the MMPA methodology estimates responses consistent with about the 80%-ile values computed from nonlinear response-history analysis.[4] We questioned this conclusion because it was reached using 11 sets of ground motions scaled to the same value of peak ground acceleration (PGA). Figure 4.1 presents spectra from ground motions scaled differently: a) spectral matching and b) scaled to peak ground acceleration. Figure 4.1b presents the resulting 80%-ile response spectrum and the target response spectrum for 11 scaled records. Over the frequency range from 5 Hz to 7 Hz that dominates the vertical response of the roof girders, the average ratios of the 80% NEP to the target spectral acceleration are greater than 1.3. This peak and valley variability in the spectrum is too great because it double counts some of the aleatory variability already included in the seismic hazard study that generated the target spectrum.

Figure 4.2 presents the shears at the south ends (E) of interior girder EH calculated using the MMPA, response-spectrum analysis (RSA), and response-history analysis (denoted TH in the legend to the figure). Results are presented for Lines 2 through 15, noting that Lines 1 and 16 are walls (no girders). In these analyses, a) the MMPA considers nonlinear response but the roof girders are modeled as equivalent linear elements, b) the response-spectrum analysis is (by definition) linear, and c) the response-history analyses are linear. The MMPA shears are generally greater than the 80% NEP shears from the 11 response-history analyses using the scaled ground motions, noting that the dispersion in the scaled motions is too high, as noted above. This figure indicates that the MMPA method has a significant conservative bias, namely, it overestimates demand. However, we were not able to determine why the MMPA method calculates greater girder shears than those calculated using the spectrally-matched ground motions (Median-Matched TH in the figure) when the same building model and damping are

[3] Task *1b - Confirm interior girder shear results 12_18_2014.pptm* (Appendix B)

[4] *Task 1b - Confirm interior girder shear results 12_18_2014.pptm*

used. Accordingly, we cannot characterize the conservative bias in the MMPA methodology with respect to the 80% NEP that would be calculated from a formal probabilistic assessment, and so have set it aside for the discussion that follows.

We have estimated that the SGH total seismic shears $\sum V_S$ are conservatively biased above the 80% NEP seismic demands by a factor (F_{SGH}) of about:

$$F_{SGH} = \Delta_{damping} \times \Delta_{soil\_spring} \times \Delta_{SSI} = 1.125 \times 1.15 \times 1.1 = 1.42 \qquad (4.2)$$

where the factors are derived above. To obtain an improved estimate of the 80% NEP total seismic shear $\sum V_S$ on the interior girders EH, we would reduce the SGH-computed $\sum V_S$ by a factor $f_{\sum V_s}$:

$$f_{\sum V_s} = F_{SGH}^{-1} = 0.70 \qquad (4.3)$$

4.3.2.2 Effect of rotational resistance on girder shears at interior girders EH

Any rotational resistance at Line E will significantly affect the distribution of the total girder shear $\sum V_S$ between Lines E and H. CJC and SGH have made different assumptions regarding the rotational resistance at Line E for these girders, as described below.

CJC assumed that the girders are continuous at Line H and are pinned (free to rotate) at Lines E and K. This assumption maximizes the fraction of $\sum V_S$ resisted by girders at Line H and minimizes the fraction at Lines E and K. The CJC assumption is likely consistent with that made by the designers in the early 1970s.

SGH developed a much more detailed model of the framing in the vicinity of the service chases. They estimated that a free-to-slip feature associated with the chase floor slab (see Figure 1.3d, left hand panel) is likely to lock-up under earthquake shaking. Accordingly, the SGH model provides significant rotational resistance to the interior girders at Lines E and K.

Our review of the CJC and SGH models and analysis output leads us to conclude that the difference in distribution in roof girder shears on the interior girders EH is largely due to the different treatments of the their interaction with the service chase. The models adopted by SGH and CJC collectively bound the restraint provided to the EH girders, with the *actual* restraint along the service chase being highly dependent on the timing and sequence of the removal of the shoring beneath the roof girders and slabs, both of which are unknown, and the integrity of the slip detail associated with the service chase floor slab, which is also unknown. The state of the sliding joint and its compressible material may also have been affected by concrete shrinkage, but that was not monitored and its effect has not yet been investigated.

Table 4.4 reports the girder shear ratio, defined as the fraction of total girder shears, $\sum V_G$ and $\sum V_S$, at end E of the interior girders EH, as obtained from Table 4.3. The CJC model produces about:

$$\frac{V_E}{\sum V} = 0.33, \quad \frac{V_H}{\sum V} = 0.67 \tag{4.4}$$

where $\sum V_E$ and $\sum V_H$ are the shears at ends E and H, respectively. The SGH model produces about:

$$\frac{V_E}{\sum V} = 0.45, \quad \frac{V_H}{\sum V} = 0.55 \tag{4.5}$$

To reasonably bound the girder shears at Line E, we recommend increasing the CJC-computed shears at end E by the factor

$$F_{CJC,E} = 0.45/0.33 = 1.36 \tag{4.6}$$

Conversely, to reasonably bound the girder shears at Line H, we recommended increasing the SGH-computed shears at end H by the factor

$$F_{SGH,H} = 0.67/0.55 = 1.20 \tag{4.7}$$

No similar adjustment is needed for the exterior girders since both the CJC and SGH models distribute close to 50% of the total shear to each end of these exterior girders. (However, we have recommended that LANL carefully study both the restraint provided to the exterior roof girders by the service chase and its possible impact on their performance in PC3 shaking.)

4.3.2.3 Adjustments to the computed D/C_{ACI} to enable bounding estimates of $D_{80\%}$ for the interior girders EH

No adjustment is suggested for the CJC $D_{80\%}/C_{ACI}$ computed ratios near Line H. The CJC values of $D_{80\%}/C_{ACI}$ near Line E should be increased by 1.36 per Eqn. (4.6). The SGH calculations of D_{MMPA}/C_{ACI} at Line H should be adjusted by

$$V_{A,H} = F_{SGH,H}\left(V_G + f_{\Sigma V_s} V_S\right) \tag{4.8}$$

where $F_{SGH,H} = 1.20$ from Eqn. (4.7) and $f_{\Sigma V_s} = 0.70$ from Eqn. (4.3) to bound $D_{80\%}/C_{ACI}$ at Line H. For the D_{MMPA}/C_{ACI} at Line E, $V_{A,E}$ can be estimated by:

$$V_{A,E} = V_G + f_{\Sigma V_s} V_S \tag{4.9}$$

Table 4.5 presents the bounded estimates of $D_{80\%}/C_{ACI}$ based on the above presentation and the CJC and SGH results reported in Table 4.1, under the sub-heading of "As-Built." These As-Built estimates of demand-to-capacity are approximate and were derived solely to facilitate the development of recommendations.

4.3.3 Observations of the SEP on adjusted roof girder shears

4.3.3.1 Interior girders EH and HK

We find that the CJC median and 80% NEP values of demand, $D_{50\%}$ and $D_{80\%}$, respectively, are reasonable near Line H. As a result, their demand-capacity ratios, $D_{50\%}/C_{ACI}$ and $D_{80\%}/C_{ACI}$ are also reasonable near Line H.

At Lines E and K, we consider the CJC-computed values of $D_{50\%}$ and $D_{80\%}$ to be potentially low because they have considered these girders to be pinned (free to rotate) at these supports. Considerable uncertainty exists about the amount of rotational restraint that these supports. In Section 4.3.2.2, we suggest the CJC-computed demands be increased by a factor of 1.36 at Lines E and K to reasonable bound the demand. Table 4.5 presents the adjusted CJC $D_{80\%}/C_{ACI}$ at Line E under the sub-heading of "As-Built."

On the basis of the documents made available to us, we judge the SGH-computed seismic demands, D_{MMPA}, to be more conservative than $D_{80\%}$ seismic demands. Four sources for this conservatism were discussed in Section 4.3.2.1. Therein, we suggested reducing the SGH-computed D_{MMPA} seismic demand on the interior girders by a factor of 0.7 to estimate $D_{80\%}$. In Section 4.3.2.2, we recommended increasing the SGH-computed gravity and seismic demands at Line by 1.2 to reasonably bound the effect of unknown rotational restraint to the girders by the service chase construction. These two suggested adjustments have been made in Table 4.5 for the interior girders: see sub-heading "As-Built".

The following findings are based on the adjusted $D_{80\%}/C_{ACI}$ reported in Table 4.5 for interior girders EH (and HK).

1. The adjusted $D_{80\%}/C_{ACI}$ for CJC and SGH are, on average, similar, with a mean ratio of CJC/SGH of 1.02. However, along individual gridlines, considerable variability exists with the ratio ranging between 0.80 and 1.30. This variability reflects the differences in the models and the methods of analysis. We consider both models to be reasonable.

2. At Line H, the *unadjusted* CJC $D_{80\%}/C_{ACI}$ range between 0.81 at Line 8 and 0.99 at Line 4. The *adjusted* SGH $D_{80\%}/C_{ACI}$ range between 0.68 at Line 15 and 1.10 at Line 12. One finding is that the highest CJC and SGH values $D_{80\%}/C_{ACI}$ differ by only a factor of 1.11, which is within the range expected by the SEP for computing $D_{80\%}/C_{ACI}$.

3. At Line H, the values of $D_{80\%}/C_{ACI}$ are generally high, with either the CJC or SGH ratio exceeding 0.9 at 10 of the 14 locations.

4. At Line E, the *adjusted* CJC $D_{80\%}/C_{ACI}$ range between 0.58 and 0.77 and the *adjusted* SGH $D_{80\%}/C_{ACI}$ range between 0.45 and 0.79. An important finding is that the values are all less than 0.80 on Line E.

4.3.3.2 Exterior girders AD and LO

We find that the CJC median demands, $D_{50\%}$, and 80% NEP demands, $D_{80\%}$, are reasonable for the ends of the exterior girders AD (and LO), subject to the rotational restraint provided by the service chases (see above) being small to none.

For the reasons given above, we judge the SGH-computed seismic demands D_{MMPA} to be more conservative than $D_{80\%}$ but did not attempt to adjust them. Instead, we based our findings on the CJC values of $D_{80\%}/C_{ACI}$ that are reported in Table 4.2.

Except at Line 8, the CJC-computed $D_{80\%}/C_{ACI}$ ranges between 0.69 and 0.83. No meaningful difference exists in the $D_{80\%}/C_{ACI}$ ratios at the two ends. The ratio is marginally higher on Lines 2 through 7 than on Lines 9 through 15. On Line 8, the ratio $D_{80\%}/C_{ACI}$ is high at both ends of girder AD: 0.94 at Line A and 0.95 at Line D.

4.3.4 Findings of the SEP

We have reviewed the CJC fragility analyses and annual frequencies of unacceptable performance (AFUP) summarized in Mertz et al. (2015; see Appendix B). Note that the fragilities and AFUP have been separately calculated for each end of each individual girder. As a result, the AFUP reported by Mertz et al. are more accurate and lower than the bounding fragilities and bounding AFUP previously reported by CJC. Figure 4.3 presents these AFUP for the roof girders.

We have carefully reviewed the fragility analysis methodology followed by CJC, and find it to be reasonable. We have reviewed the fragility calculations for the interior girders EH at Line H. These fragility results are reasonable. We agree with how the fragilities were convolved with the mean seismic hazard to produce estimates of the mean AFUP.

We concentrated our review on the fragilities at Line H of these interior girders because we felt that we needed estimates of the AFUP values at Line H to assist us in making recommendations about increasing girder shear capacities in the vicinity of Line H. Because of time constraints on issuing our report, we have not reviewed the fragility calculations at other roof girder locations.

We find the AFUP results reported in Figure 4.3 along Line H to be reasonable and consistent with the $D_{80\%}/C_{ACI}$ reported by CJC in Table 4.1 and our adjusted values that are reported in Table 4.5. The AFUP on Line H range from a high of 0.68×10^{-4} at Line H of girder EH on Line 4 to a low of 0.35×10^{-4} at Line H of girder HK on Line 13. However, we also recognize that considerable variability exists between the $D_{80\%}/C_{ACI}$ computed by CJC and our adjusted SGH $D_{80\%}/C_{ACI}$ on Line H, as is shown in Table 4.5. Therefore, we recommend that any increase in shear capacity to the girders be applied on both sides of Line H from Lines 2 through 15 and that the decision whether to upgrade be based on the highest AFUP of 0.68×10^{-4}.

The very low values of the AFUP reported at Line E of girder EH and Line K of girder HK seem to be consistent with the small values of $D_{80\%}/C_{ACI}$ reported by CJC in Table 4.1. However, in Table 4.5 we increased these ratios by a factor of 1.35 to address the unknown rotational restraint provided by the service chases. This increase will result in a substantial increase in the AFUP at Lines E and K above those values shown in Figure 4.3. However, even after this increase, we expect the AFUP at Lines E and K to be low.

We do not understand the low values of AFUP values reported in Figure 4.3 for the exterior girders AD and LO. These low values are not consistent with the values of $D_{80\%}/C_{ACI}$ reported by CJC in Table 4.2. Based on the reported values of $D_{80\%}/C_{ACI}$, we would have expected the AFUP for the exterior girders AD and LO Girders to have been between 0.3×10^{-4} and 0.5×10^{-4} on Lines 2 through 7 and 9 through 15. Regardless, these values are low and do not warrant further review unless the studies identified in Section 5 determine that the service chases provide significant rotational restraint to the exterior girders, drawing a significant fraction of the total shearing force away from the perimeter support of these girders.

The low values of AFUP on Line 8 for girder AD (=0.000029 and 0.000030) could impact our recommendations for its shear retrofit . The high values of $D_{80\%}/C_{ACI}$ reported by both CJC and SGH (see Table 4.2: 0.89 to 0.97) indicate that retrofit must be considered but this is at odds with the calculated AFUP.

To help resolve this apparent inconsistency, we requested CJC to check both $D_{80\%}/C_{ACI}$ and AFUP they reported for girder AD on Line 8. CJC responded that the exterior girders have a low AFUP with respect to their D/C because the latter results include additional conservatism. The CJC capacity in Table 4.2 is based on an effective shear area that neglects both the flange (slab) and drag strut concrete areas, whereas the failure frequencies calculated in Mertz et al. (2015b) includes portions of both the flange and drag strut in the effective concrete shear area (see Figure 4.4). This change results in an 11% increase in effective shear area for girders without drag struts and a 24% increase in shear area for the AD girder that supports the drag strut. CJC revised their calculations of $D_{80\%}/C_{ACI}$ based on the capacities used for the AFUP calculations: the updated demand-capacity ratio of girder AD on Line 8 is 0.81 (versus 0.95 in Table 4.2) and $P_f = 0.3\times10^{-4}$; for girder AD on Line 2, the updated demand-capacity ratio is 0.77 (versus 0.82 in Table 4.2) and $P_f = 0.33\times10^{-4}$. These failure probabilities and revised D/C ratios are consistent.

4.3.5 Recommendations of the SEP

Based on the CJC estimated values of $D_{80\%}/C_{ACI}$, several interior girders barely satisfy the DOE-1020 and ASCE 43-05 requirements that $D_{80\%}/C_{ACI}$ be less than 1.0. However, the overriding DOE-1020 and ASCE 43-05 target performance goal of $\text{AFUP} \le 1.0\times10^{-4}$ for a PC3 component is easily satisfied by each of the roof girders, individually. The highest AFUP computed by CJC is 0.68×10^{-4} per year, which is well below the PC3 target performance goal.

Based on the *adjusted* values of SGH $D_{80\%}/C_{ACI}$, shown in Table 4.5, the requirement of $D_{80\%}/C_{ACI} \leq 1.0$ is not met for two of the EH girders. It is also probably not met by at least one of the HK girders near Line H. However, the maximum *adjusted* $D_{80\%}/C_{ACI}$ is 1.10, or only 11% higher than the greatest CJC $D_{80\%}/C_{ACI}$ estimate. This 11% increase is not expected to result in an increase in the AFUP of more than about 25%. Therefore, even using the *adjusted* values of SGH $D_{80\%}/C_{ACI}$, the estimated AFUP will be less than 0.9×10^{-4} per year, which satisfies the target performance goal of 1.0×10^{-4} per year.

We conclude that it is not necessary to increase the shear capacity of any of the roof girders to specifically satisfy DOE-1020 or ASCE 43-05. However, several other issues must be considered. First, even if the AFUP of every component in PF-4 is less than 1.0×10^{-4}, the combined AFUP for PF-4 might exceed this value because of the combination of failures of partially independent components. This *systems* issue is not addressed by the criteria in either DOE-1020 or ASCE 43-05. Second, the DOE-1020 performance goal for PC3 was to aim at a target AFUP of 1.0×10^{-4} against the limit state of *Loss of Confinement*. No performance goal was established for the collapse limit state because it was generally believed the AFUP against collapse would be meaningfully less than that for loss of confinement. In this regard, it must be be emphasized that roof girder shear failure represents both loss of confinement and collapse. For these reasons, the SEP firmly believes it would be prudent to increase the shear capacity of all of the interior girders EH and HK in the vicinity of Wall H.

We recommend that the shear capacity of the interior girders on both sides of Line H be increased by about 120 kips. Externally applied carbon fiber polymer reinforcement (CFRP) is likely the most efficient solution. The increase in strength will reduce the highest CJC value of $D_{80\%}/C_{ACI}$ to approximately 0.80, reduce the highest *adjusted* SGH $D_{80\%}/C_{ACI}$ to approximately 0.80, and reduce the highest estimated AFUP to below 0.5×10^{-4}. Estimates of the CJC and SGH $D_{80\%}/C_{ACI}$ after the augmentation of shear capacity are presented in Table 4.5 under the sub-heading “Recommended Retrofit.”

Reducing the highest AFUP to less than 0.5×10^{-4} will remove roof-girder shear failures from being a dominant contributor to the risk of collapse of PF-4 and more evenly distribute seismic risk across the structural components of PF-4, which is desirable from the perspective of a risk portfolio. The AFUP for roof girder shear will be similar in magnitude to other failure modes that could trigger collapse. Little additional benefit in terms of reduced seismic risk to PF-4 would be gained by increasing C_{ACI} by more than 120 kips. In addition, reducing the Line H girder *D*/*C* to 0.8 will make them consistent with the *D*/*C* of the exterior girders, thereby more equally distributing the risk across the PF-4 structure.

The other roof girder of immediate concern is the exterior girder AD on Line 8. Notwithstanding the low AFUP computed by CJC (see Figure 4.3), but based on the high *D*/*C* computed by CJC (see Table 4.5), we recommend increasing its shear capacity at both ends. The low AFUP computed by CJC was based on potentially unconservative assumptions, namely, the contribution to the shear resistance from flange (slab) and drag strut concrete. We recommend the shear capacity of both ends of girder AD on Line 8 be increased by a minimum of 60 kips, which will reduce the CJC-computed *D*/*C* ratio from 0.95 to 0.80: see Table 4.5 under the sub-heading “Recommended Retrofit.”

4.4 Laboratory Floor Punching Shear

4.4.1 Introduction

In Section 4.4.2 the similarities and differences between the laboratory floor punching shear demands and capacities reported by CJC and SGH are identified. Sections 4.4.3 and 4.4.4 present our findings and recommendations.

The SGH punching shear demands, computed using the MMPA, are presented in Table 4.6. The CJC punching shear demands are presented in Table 4.7. (The current demand-capacity ratios (*D*/*C*) from CJC and SGH differ somewhat from prior tabulated results.) The last column in these tables presents *D*/*C* after accounting for floor openings that cross the shear perimeter. The results presented in these tables form the basis of our discussion below. We note that these results assume:

1. Live loads are reduced from the original design values to calculate gravity (V_G) and seismic (V_S) shears. On-site postings and LANL administrative measures must be

implemented to ensure that this assumption is maintained for the life of the facility. See the required actions of Section 6.2.

2. The column capitals remain intact under DBE shaking and the shear perimeter is calculated accordingly.

4.4.2 Similarities and differences

SGH report, per Table 4.6, *D*/*C* ranging from 0.43 to 0.94, with the greatest values at columns G5 and E7; *D*/*C* exceeds 0.80 at 13 locations. CJC report, per Table 4.7, *D*/*C* ranging between 0.29 and 0.75, with the greatest values at columns C5, F5 and G5.

Rather than reviewing all of the punching shear results, we have concentrated our review on the 13 columns identified by SGH as having *D/C* greater than 0.80. Table 4.8 presents the CJC and SGH punching shear results at these 13 columns. The gravity demands, V_G, do not differ significantly between SGH and CJC with CJC/SGH results ranging between 0.91 and 1.24, with an average of 0.99. These differences are not sufficient to affect any of our findings or recommendations.

There is a significant difference between the 80% NEP seismic shears, $V_{S,80}$, reported by CJC and the seismic shears, $V_{S,MMPA}$, reported by SGH. The ratio of CJC-to-SGH seismic shears range between 0.36 and 0.90, with an average of 0.64: on average, the SGH seismic shears are 1.56 times the CJC seismic shears. The ratio of seismic shears is highly variable for these 13 columns.

Both structural models are considered to be reasonable and both seismic analyses are of high quality. In our judgment, the column-to-column variability is likely due to differences in the models and analysis methods used.

Slab-column connection capacities were reported by SGH in units of stress (psi) whereas CJC reported capacities in units of force (kips). As such, a direct comparison was not possible. In Table 4.6 we have replaced SGH's capacity in psi with an *approximate* capacity in kips by multiplying the psi by the critical area for each column. This enables a comparison of the reported capacities, which are sufficiently similar so as not to affect our findings and recommendations.

A large difference exists between the reported seismic demands $V_{S,80}$ and $V_{S,MMPA}$. Section 4.3.2.1 discusses the differences in the roof girder demands. Applying the same logic here, the SGH seismic demands, $D_{MMPA,}$ should be reduced by a factor, $f_{\Sigma V_s} = 0.70$, to make them approximately equivalent to the CJC-computed 80% NEP seismic demands, $D_{80\%.}$ The net effect is to reduce the SGH demand-capacity ratios by factor F_A:

$$F_A = \frac{V_G + 0.7V_S}{V_G + V_S} \qquad (4.10)$$

to convert (D_{MMPA}/C) to approximate estimates of ($D_{80\%}/C$). This adjustment has been made in Table 4.8 to the SGH demand-capacity ratios for the 13 columns. The maximum *adjusted* D/C is 0.80.

The greatest *adjusted* SGH $D_{80\%}/C$ of 0.80 occurs at column G6 and is only 7% higher than the highest CJC $D_{80\%}/C$ of 0.75 at columns G5 and F6.

4.4.3 Findings of the SEP

We consider the $D_{80\%}/C_{ACI}$ estimates from CJC to be reasonable with the maximum value of 0.75. The maximum *adjusted* SGH $D_{80\%}/C_{ACI}$ is 0.80, which is close to the maximum CJC value.

4.4.4 Recommendations of the SEP

We see no need to increase the punching shear strength of the laboratory floor slabs provided a) on-site postings and administrative measures are enacted to limit the live load to that assumed for the CJC and SGH analyses, and b) the column capitals remain intact for DBE shaking. We believe it is prudent to test the column capitals to confirm this important assumption, as noted in Section 6.

4.5 Column Capitals

4.5.1 Introduction

The evaluations of Section 4.4 assumed that the column capitals remained intact and undamaged during DBE shaking. The demand-capacity ratios for punching shear with intact capitals are sufficiently high that we believe that the loss of a column capital could be catastrophic and result in the partial collapse of the floor slab and loss of confinement.

4.5.1.1 CJC Evaluation

CJC evaluated the column capitals using an elastic model with explicit reinforcing elements. In the November 2014 meeting, CJC reported the results of such analyses to be sensitive to the presumed bond model. At the February 2015 meeting, Dr. Mertz (CJC) explained that the CJC model predicted minor cracking at the rebar-to-steel interface at the base of the capital, but that those cracks did not propagate vertically along the bars. CJC concluded that the possibility of capital damage is very low, and that there was no need to include this failure mode in the fragility analyses.

4.5.1.2 SGH Evaluation

SGH evaluated the column capitals using a nonlinear damage model (smeared crack model) as implemented in the DIANA finite element code (TNO DIANA 2014). The model utilized an elastic slab segment to impose the column forces extracted from their global ABAQUS model, and those imposed tractions were resisted by inelastic column capital elements and an elastic column stub extending two feet below the base of the capital. Like the CJC evaluation, the rebar was modeled explicitly with a presumed bond relation. SGH also found that the predicted cracking is sensitive to the assumed bond relationship, and lowering the bond quality decreases the cracking at the column-to-capital construction joint. SGH based their bond model and parameters in part on replicating test results reported by Viwathanatepa (1979). They also found increased cracking when using a very low concrete strength, as would be expected. In addition to modeling three variations of the PF-4 column capitals, SGH validated the software and modeling technique by replicating the laboratory testing of a column capital by To and Moehle (2012) and by comparing the model predictions to the cyclic behavior and damage patterns described in the test summary.

We noted that SGH focused their study on the column capital at G2, which is one of the *short* Type V columns integral with the basement walls below. The loads applied to column capital G-2 came from Run 68 of the global model, a run with Sensitivity 03 model to maximize demand on the wall on Line 8. Run 68 was superseded by Run 85 reported in Appendix G of SGH Report

121049-RPT-03 and contains bounding values of member properties. Other global analyses generated forces on Type V column capitals that were nearly as great as Run 68, indicating that this choice of loadings would not affect our conclusions or recommendations.

The SGH evaluation predicted more extensive vertical cracking along the reinforcement than had been predicted by CJC. At the February 2015 meeting, SGH clarified that many of the “cracks” that are reported are likely micro-cracks that would not be visible. The SGH model also predicted considerable horizontal cracking between the top of the capital and the soffit of the slab, but this cracking is likely an artifact of the interface of inelastic capital elements and elastic slab elements. It is the vertical cracks along the column rebar that are of concern and those are addressed herein. Based on their results, SGH were unable to conclude that capitals would be undamaged in DBE shaking. SGH did not attempt to assess the impact of this cracking and if it would affect the ability of the column capital to maintain support of the laboratory slab flat slabs. SGH prudently recommended further evaluation of the column capitals.

4.5.2 Findings of the SEP

The SEP consider the SGH analysis to be the very best and most rigorous performed to date, in either academia or professional practice. SGH attempted to validate their software and modeling technique by replicating a laboratory test by To and Moehle (2012). Although this test is likely the best available, the geometry of the capital, column axial loads, and test boundary conditions do not closely represent the PF-4 geometry, demands, and boundary conditions, thus making it very difficult to provide conclusive validation. Importantly, the metadata required for a formal validation per ASME (2006) were not available. (We are grateful to Dr. To and Dr. Moehle for making the test data available to SGH.) The column capital specimens tested by To and Moehle were of lightweight concrete, were cyclically tested and had their first significant drop in strength at about 3.5% drift. The capital continued to support the slab up to 7.0% drift. Failure was a reverse shear crack in the slab above the capital. Some spalling of the capital was reported. It should be noted that the predicted seismic drift in the lower story of PF-4 is about 0.15% drift, or about 0.35% drift if the wall below is assumed rigid at the captured Type V columns.

The fundamental difference between the CJC and SGH results is the severity of the cracking that forms at and propagates along the longitudinal column reinforcement extending into the capital.

Based on Dr. Mertz's description of the CJC results, it appears that the SGH model predicts more extensive cracking, which opens the possibility of capital spalling on subsequent reversed cycles of loading. One reason for the different results is the assumed reinforcement bond model. For the lower bond capacity used in the CJC model, we understand that the cracking is constrained to the area immediately adjacent to the construction joint at the base of the capital. In the SGH model, which considers a range of bond strengths and models, the cracking propagates farther along the rebar. We are aware of no test data available to quantify the in-situ bond quality at PF-4, and therefore definitive resolution of this difference is not possible.

We are aware of several structures of similar construction with unreinforced column capitals that have experienced high drifts in past earthquakes without failure. Although such observations add to our judgment that the PF-4 column capitals will survive DBE shaking, they are not definitive proof that column capital failures will not occur.

Based on our collective experience and the predicted deformations at DBE shaking, we believe that spalling of capitals and subsequent punching shear is unlikely for PC3 shaking. However, the analyses to date do not allow us to definitively dismiss the potential for failure. Because the demand-capacity ratios for punching shear are relatively high with intact capitals, and thus imply potentially catastrophic consequences if there should be capital failure(s), the evaluation of the unreinforced column capitals warrants a very conservative approach.

4.5.3 Recommendations of the SEP

We believe that it would be prudent to perform limited laboratory testing of column capitals representative of those found in PF-4. We all agree that the Type V captured columns are the most critical and should, at a minimum, form a prototype for such tests. Results from a properly-designed test program, in conjunction with the CJC and SGH evaluations performed to date, will allow LANL to determine whether column capital retrofit is required to reliably and conservatively meet their earthquake performance objectives. If the tests show unacceptable performance, specimens could be retrofitted and retested to validate any proposed strengthening measures. This matter is discussed further in Sections 5 and 6.

4.6 Supplemental Linear Elastic Analyses

4.6.1 Introduction

One of the SEP tasks was to identify and quantify the factors that led to different SGH and CJC conclusions and recommendations. To help do so, we requested supplemental linear elastic analyses be performed by both CJC and SGH. These analyses allowed us to remove differences due to loading, boundary conditions, and nonlinear response. The loading we prescribed was simple enough to be easily dissected, yet realistic enough to be informative. We identified three static load cases: a 1 *g* uniform acceleration in each principal direction. We also requested a linear response history analysis using a single ground motion (provided by SGH).

4.6.2 Interior girder end shear

We investigated the differences in modeling approaches by CJC and SGH related to analysis of the PF-4 interior roof girders. This review was enabled by a set of results submitted by both SGH and CJC that summarized roof girder demands under the static and dynamic inputs described above. Here we focus on the 1*g* vertical static load applied to the linear elastic, fixed base finite element model. These results are sufficient to establish the modeling assumptions that underpin some of the differences in the outputs reported in prior sections.

We focus on the 11 girders between Lines E and H, along Lines 3 through 7 and 9 through 14. Line 8 includes a drag strut that was added after the original construction, and is therefore a unique case. The CJC static shears at a distance *d* from the column/pilaster face for these girders are all approximately 72 kips for the south end and 132 kips for the north end. This leads to a north/south shear ratio of 1.83. If we examine the shear diagram for a standard Bernoulli-Euler beam with pinned boundary at the south end and fixed boundary at the north, we compute an end shear ratio of 1.67, independent of the assumed length. However, considering a beam of length 56.5 feet and a girder depth d = 7 feet, the shear ratio becomes 1.83 at a distance *d* from the supports, precisely corresponding to that reported by CJC. From this result and other descriptions contained in the original calculations, we verify that the CJC typical girder model incorporates an idealized moment release at the south end and idealized fixity at the north end over the wall on Line H.

The results provided by SGH indicate different boundary fixity at the south end of these girders. The SGH static shear results at a distance *d* from the column/pilaster face for these girders are all approximately 85 kips for the south end and 103 kips for the north end. This leads to a north/south shear ratio of 1.21, much lower than the 1.83 reported by CJC. In short, while the south end of the interior girder is pinned in the CJC model, the south end in the SGH model is partially restrained, and the induced south end moment redistributes more of the total shear to the south.

In examining the connectivity of the EH girders to the service chase between Lines D and E, we find that the top and bottom slabs of the chase appear to be positioned like continuity plates in a steel moment connection, potentially restraining the top and bottom of the roof girder (Figure 1.3b). The structural drawings show that the chase roof slab has construction joints over both chase walls (Lines D and E) but with continuous reinforcement crossing the joint, and girder top bars hooked into the pilaster. The detail for the floor slab is different: The south end is detailed with a slip joint in which the six-inch slab rests on a steel angle in a pocket filled with compliant material, presumably to allow unrestricted movement (see Figure 4.5, which is a zoomed view of the left panel of Figure 1.3d). The intent of the designer appears clear, namely, to avoid axial load in the floor slab, allowing the girder to freely rotate. How well this detail was constructed and its ultimate performance in this role is unknown.

The SGH model explicitly addresses these condition details, as described in Figure 4.6. The chase roof slab is connected with compression and tension springs intended to model the stiff concrete bearing resistance in compression, and the compliance of the rebar in tension. The service chase floor slab appears to be free to slip in tension but is provided with a “stiff” compression spring, reflecting a perceived unreliability of the joint to slide freely or perhaps locking of the compression material after a finite movement. From this figure, it appears that the chase roof and/or slab shell elements indeed provide some measure of resistance to girder flexure by virtue of their elevations relative to the top and bottom of the rigid zone defined by the girder depth. That is, either the roof or floor slab of the chase corridor can introduce a kinematic rotational restraint under vertical loading; the degree of restraint depends on the stiffness of the springs used to model the joints and the restraint provided by the exterior girder spans.

Our review of the CJC and SGH models and analysis output leads us to conclude that the differences in roof girder shear demands is largely due to different treatments of the connectivity of the south end of the girders to the service chase at Line E. Whereas CJC assumed an idealized moment release, SGH included the flexural interaction of the end of the girder with the roof and floor slabs of the chase corridor.

The models adopted by SGH and CJC effectively bound the restraint provided to the EH girders, with the *actual* restraint along the chase/service tunnel being dependent on the as-built joint detail and on the timing / sequence of the shoring removal from beneath the roof girders and slabs, both of which are unknown. We have quantified this difference and adjusted the SGH and CJC demands accordingly; see Section 4.3.

4.6.3 Soil boundary conditions and damping

The linear analyses were also very helpful in quantifying the differences due to boundary conditions and damping. SGH incorporated discrete soil springs below the footings. The stiffness of the soil springs was based on the footing geometry and the measured soil properties. The soil compliance increased some demands on the structure. In contrast, the CJC demands were derived from a soil-structure interaction (SSI) analysis based on a different methodology. Whereas the soil compliance caused the SGH seismic demands to *increase*, CJC reported that incorporating SSI resulted in a *decrease* in in seismic demand. We believe that the full SSI solution is likely more realistic, albeit computationally more onerous, and adjusted the SGH demands down accordingly; see Section 4.3.

SGH and CJC also differed in their modeling of structural damping. Whereas SGH used 4% modal damping in their response analyses, CJC assumed a median value of 7%. SGH evaluated the effect of damping on the response of the roof girders, and they found that the structural response increased approximately 25% when the damping was decreased from 7% to 4%. The SGH approach is typical for nonlinear analysis, that is, to use a relatively low value of structural damping that would be supplemented by hysteretic energy dissipation. However, some SGH member properties were presumed cracked for the roof girders and the laboratory slab floor, but the energy dissipation associated with that damage was not included. Therefore, for the evaluation of the roof girder and lab floor slab response, the SEP believes that 7% damping is appropriate and we adjusted SGH demands down accordingly; see Sections 4.3 and 4.4.

Table 4.1. Summary of SGH and CJC interior girder shears (adapted from SGH 2014c and Mertz et al. 2013)

		SGH Girder Shear Evaluation						CJC Girder Shear Evaluation					Ratios of Results $^{CJC}/_{SGH}$				
		V_G	V_S	V_T	ϕV_n	Amp	D/C	V_G	V_S	V_T	ϕV_n	D/C	V_G	V_S	V_T	ϕV_n	D/C
	Grid	kip	kip	kip	kip	()	()	kip	kip	kip	kip	()	()	()	()	()	()
Interior Girders EH, North End	2	119	151	270	319	1.00	**0.84**	149	146	295	322	**0.92**	1.25	0.97	1.09	1.01	**1.09**
	3	115	187	302	319	1.01	**0.96**	152	163	314	323	**0.97**	1.32	0.87	1.04	1.01	**1.02**
	4	115	228	343	321	1.01	**1.08**	152	168	320	323	**0.99**	1.31	0.74	0.93	1.01	**0.91**
	5	115	216	331	318	1.02	**1.06**	143	149	292	321	**0.91**	1.25	0.69	0.88	1.01	**0.86**
	6	114	183	297	317	1.00	**0.94**	143	144	287	320	**0.90**	1.26	0.79	0.97	1.01	**0.95**
	7	112	162	274	321	1.00	**0.85**	151	156	306	323	**0.95**	1.35	0.96	1.12	1.01	**1.11**
	8	115	163	278	286	1.00	**0.97**	183	175	357	443	**0.81**	1.59	1.07	1.29	1.55	**0.83**
	9	105	127	232	321	1.00	**0.72**	142	140	282	321	**0.88**	1.35	1.10	1.22	1.00	**1.22**
	10	107	143	250	317	1.00	**0.79**	142	146	287	321	**0.89**	1.32	1.02	1.15	1.01	**1.13**
	11	111	197	308	318	1.00	**0.97**	147	160	307	322	**0.95**	1.32	0.81	1.00	1.01	**0.98**
	12	115	256	370	321	1.00	**1.15**	141	155	297	323	**0.92**	1.23	0.61	0.80	1.01	**0.79**
	13	111	218	330	320	1.00	**1.03**	137	142	278	323	**0.86**	1.23	0.65	0.84	1.01	**0.84**
	14	108	160	268	319	1.05	**0.88**	142	147	288	322	**0.89**	1.32	0.91	1.08	1.01	**1.01**
	15	105	109	213	320	1.03	**0.69**	142	135	278	321	**0.86**	1.36	1.25	1.30	1.00	**1.25**
												Average	**1.32**	**0.89**	**1.05**	**1.05**	**1.00**
Interior Girders EH, South End	2	92	114	206	303	1.01	**0.68**	67	73	140	305	**0.46**	0.73	0.64	0.68	1.01	**0.67**
	3	88	158	246	303	1.02	**0.83**	68	86	154	305	**0.50**	0.78	0.54	0.63	1.01	**0.61**
	4	88	196	285	303	1.01	**0.95**	68	90	158	305	**0.52**	0.77	0.46	0.56	1.01	**0.55**
	5	89	175	264	303	1.04	**0.90**	62	74	136	305	**0.45**	0.69	0.42	0.52	1.01	**0.49**
	6	89	138	228	303	1.03	**0.78**	62	68	130	305	**0.43**	0.69	0.49	0.57	1.01	**0.55**
	7	90	128	219	303	1.00	**0.72**	70	80	150	305	**0.49**	0.77	0.62	0.68	1.01	**0.68**
	8	115	148	264	296	1.03	**0.92**	87	87	174	305	**0.57**	0.75	0.59	0.66	1.03	**0.62**
	9	86	104	190	303	1.00	**0.63**	64	69	132	305	**0.43**	0.75	0.66	0.70	1.01	**0.69**
	10	85	100	185	289	1.02	**0.65**	64	71	135	305	**0.44**	0.75	0.71	0.73	1.06	**0.68**
	11	87	156	243	303	1.02	**0.82**	66	83	149	305	**0.49**	0.76	0.53	0.61	1.01	**0.60**
	12	89	216	305	303	1.00	**1.01**	63	80	144	305	**0.47**	0.71	0.37	0.47	1.01	**0.47**
	13	87	188	274	303	1.00	**0.91**	61	75	136	305	**0.45**	0.71	0.40	0.50	1.01	**0.49**
	14	84	130	215	303	1.05	**0.74**	64	74	138	305	**0.45**	0.76	0.57	0.64	1.01	**0.61**
	15	80	79	160	303	1.08	**0.57**	64	68	132	305	**0.43**	0.80	0.85	0.82	1.01	**0.76**
												Average	**0.74**	**0.56**	**0.63**	**1.01**	**0.60**

Table 4.2. Summary of SGH and CJC exterior girder shears (adapted from SGH 2014c and Mertz et al. 2013)

		SGH Girder Shear Evaluation						CJC Girder Shear Evaluation					Ratios of Results $^{CJC}/_{SGH}$				
		V_G	V_S	V_T	ϕV_n	Amp	D/C	V_G	V_S	V_T	ϕV_n	D/C	V_G	V_S	V_T	ϕV_n	D/C
	Grid	kip	kip	kip	kip	()	()	kip	kip	kip	kip	()	()	()	()	()	()
Exterior Girders AD – North End	2	116	128	244	293	1.00	**0.83**	131	121	251	303	**0.83**	1.12	0.95	1.03	1.03	**1.00**
	3	114	117	231	293	1.00	**0.79**	122	117	240	303	**0.79**	1.08	1.00	1.04	1.03	**1.01**
	4	114	119	233	293	1.09	**0.87**	122	119	241	303	**0.79**	1.07	0.99	1.03	1.03	**0.92**
	5	114	137	251	293	1.02	**0.87**	113	105	218	303	**0.72**	0.99	0.77	0.87	1.03	**0.82**
	6	113	106	219	271	1.00	**0.81**	113	107	220	303	**0.73**	1.00	1.01	1.00	1.12	**0.90**
	7	113	110	222	271	1.00	**0.82**	122	121	244	303	**0.80**	1.09	1.11	1.10	1.12	**0.98**
	8	135	115	251	267	1.03	**0.97**	148	137	285	302	**0.94**	1.09	1.18	1.13	1.13	**0.97**
	9	107	115	222	293	1.00	**0.76**	114	108	223	303	**0.74**	1.07	0.94	1.00	1.03	**0.97**
	10	108	114	222	293	1.00	**0.76**	114	110	224	303	**0.74**	1.06	0.96	1.01	1.03	**0.98**
	11	111	101	212	272	1.01	**0.79**	109	104	213	303	**0.70**	0.98	1.02	1.00	1.11	**0.89**
	12	114	122	236	293	1.01	**0.82**	113	111	224	303	**0.74**	1.00	0.91	0.95	1.03	**0.91**
	13	111	131	242	293	1.00	**0.82**	118	118	236	303	**0.78**	1.07	0.90	0.98	1.03	**0.95**
	14	107	127	235	293	1.00	**0.80**	114	108	222	303	**0.73**	1.06	0.85	0.95	1.03	**0.92**
	15	100	102	202	293	1.05	**0.72**	115	98	213	303	**0.70**	1.15	0.96	1.05	1.03	**0.97**
												Average	**1.06**	**0.97**	**1.01**	**1.06**	**0.94**
Exterior Girders AD – South End	2	107	125	231	294	1.12	**0.88**	129	119	248	301	**0.82**	1.21	0.96	1.07	1.03	**0.93**
	3	105	110	216	293	1.00	**0.74**	120	116	237	301	**0.79**	1.14	1.05	1.10	1.03	**1.07**
	4	106	111	217	293	1.00	**0.74**	120	117	238	301	**0.79**	1.13	1.06	1.10	1.03	**1.06**
	5	106	128	234	293	1.00	**0.80**	111	103	214	301	**0.71**	1.05	0.81	0.92	1.03	**0.89**
	6	106	119	225	293	1.00	**0.77**	111	106	217	301	**0.72**	1.05	0.89	0.97	1.03	**0.94**
	7	104	125	229	293	1.06	**0.83**	121	119	240	301	**0.80**	1.16	0.96	1.05	1.03	**0.96**
	8	118	121	239	277	1.03	**0.89**	150	138	288	302	**0.95**	1.27	1.14	1.20	1.09	**1.07**
	9	97	114	212	293	1.00	**0.72**	113	106	219	301	**0.73**	1.16	0.93	1.03	1.03	**1.00**
	10	99	116	215	293	1.00	**0.74**	112	108	220	301	**0.73**	1.14	0.93	1.02	1.03	**0.99**
	11	103	111	214	293	1.00	**0.73**	108	102	209	301	**0.69**	1.05	0.92	0.98	1.03	**0.95**
	12	106	120	226	293	1.00	**0.77**	111	110	221	301	**0.73**	1.05	0.92	0.98	1.03	**0.95**
	13	103	136	238	293	1.00	**0.81**	116	117	233	301	**0.77**	1.13	0.86	0.98	1.03	**0.95**
	14	99	127	226	293	1.00	**0.77**	113	106	218	301	**0.72**	1.14	0.83	0.97	1.03	**0.94**
	15	93	104	197	293	1.06	**0.72**	113	97	210	301	**0.70**	1.22	0.93	1.07	1.03	**0.97**
												Average	**1.13**	**0.94**	**1.03**	**1.03**	**0.98**

Table 4.3. Total shear for interior and exterior spans (adapted from SGH 2014c and Mertz et al. 2013)

		SGH		CJC		CJC / SGH	
		ΣV_G	ΣV_S	ΣV_G	ΣV_S	ΣV_G	ΣV_S
	Grid	kip	kip	kip	kip	()	()
Interior Girders EH	**2**	211	265	216	219	1.02	0.83
	3	203	345	220	248	1.08	0.72
	4	204	424	220	258	1.08	0.61
	5	204	392	205	224	1.01	0.57
	6	203	321	205	212	1.01	0.66
	7	202	290	220	235	1.09	0.81
	8	230	311	269	262	1.17	0.84
	9	191	231	206	209	1.08	0.90
	10	192	243	206	216	1.07	0.89
	11	198	353	213	243	1.07	0.69
	12	204	472	205	236	1.00	0.50
	13	198	406	198	217	1.00	0.53
	14	192	291	206	221	1.07	0.76
	15	185	188	206	203	1.12	1.08
Exterior Girders AD	**2**	223	252	259	240	1.16	0.95
	3	219	227	242	234	1.11	1.03
	4	220	230	242	236	1.10	1.02
	5	220	265	224	208	1.02	0.78
	6	219	225	225	212	1.02	0.94
	7	217	234	243	241	1.12	1.03
	8	254	237	298	275	1.17	1.16
	9	204	230	227	215	1.11	0.93
	10	207	231	226	218	1.09	0.95
	11	214	212	217	205	1.01	0.97
	12	220	241	225	220	1.02	0.91
	13	214	266	235	235	1.10	0.88
	14	206	254	227	213	1.10	0.84
	15	193	206	227	195	1.18	0.95

Table 4.4. Fraction of total shear demand at end E of girder EH

	Grid	SGH V_G	SGH V_S	CJC V_G	CJC V_S
Interior Girders EH	**2**	0.43	0.43	0.31	0.33
	3	0.43	0.46	0.31	0.34
	4	0.43	0.46	0.31	0.35
	5	0.44	0.45	0.30	0.33
	6	0.44	0.43	0.30	0.32
	7	0.45	0.44	0.32	0.34
	8	0.50	0.48	0.32	0.33
	9	0.45	0.45	0.31	0.33
	10	0.44	0.41	0.31	0.33
	11	0.44	0.44	0.31	0.34
	12	0.44	0.46	0.31	0.34
	13	0.44	0.46	0.31	0.34
	14	0.44	0.45	0.31	0.34
	15	0.43	0.42	0.31	0.33

Table 4.5. Adjusted $D_{80\%}/C_{ACI}$ estimates for interior EH girders, and interior girder AD on Line 8

			As-Built			Recommended Retrofit		
		Grid	SGH	CJC	CJC / SGH	SGH	CJC	CJC / SGH
Interior Girders EH	North End (Line H)	**2**	0.84	0.92	1.09	0.61	0.67	1.09
		3	0.92	0.97	1.05	0.67	0.71	1.05
		4	1.03	0.99	0.96	0.75	0.72	0.96
		5	1.00	0.91	0.91	0.73	0.66	0.91
		6	0.92	0.90	0.98	0.66	0.65	0.98
		7	0.84	0.95	1.12	0.61	0.69	1.12
		8	0.96	0.81	0.84	0.68	0.81	0.84
		9	0.72	0.88	1.21	0.53	0.64	1.21
		10	0.78	0.89	1.14	0.57	0.65	1.14
		11	0.94	0.95	1.01	0.68	0.69	1.01
		12	1.10	0.92	0.84	0.80	0.67	0.84
		13	0.99	0.86	0.87	0.72	0.63	0.87
		14	0.83	0.89	1.08	0.60	0.65	1.08
		15	0.68	0.86	1.27	0.49	0.63	1.27
	South End (Line E)	**2**	0.57	0.62	1.09			
		3	0.65	0.68	1.04			
		4	0.75	0.70	0.94			
		5	0.70	0.60	0.86			
		6	0.61	0.58	0.94			
		7	0.59	0.66	1.11			
		8	0.74	0.77	1.04			
		9	0.52	0.59	1.12			
		10	0.54	0.60	1.11			
		11	0.65	0.66	1.02			
		12	0.79	0.64	0.80			
		13	0.72	0.60	0.84			
		14	0.58	0.61	1.06			
		15	0.45	0.58	1.30			
	A	**8**	0.68	0.94	1.39	0.55	0.79	1.42
	D	**8**	0.62	0.95	1.53	0.51	0.80	1.55

Table 4.6. SGH punching shear demands and capacities

	Grid	V_G kip	V_S kip	M_G kip-ft	M_S kip-ft	ϕV_n kip	D/C ()
G	2	97	136	19	164	339	0.92
	3	98	86	11	118	339	0.77
	4	94	103	10	57	339	0.68
	5	118	126	12	73	339	0.79
	6	125	123	13	72	339	0.94
	7	96	118	9	59	339	0.75
	Wall 8						
	9	84	105	9	56	339	0.69
	10	87	108	13	74	339	0.72
	11	88	107	11	62	339	0.69
	12	121	121	13	68	339	0.78
	13	122	150	14	75	339	0.87
	14	86	104	10	62	339	0.63
	15	87	100	8	55	339	0.67
F	2	67	92	14	116	339	0.58
	3	97	108	11	143	339	0.80
	4	85	118	4	57	339	0.73
	5	102	134	4	63	339	0.75
	6	107	132	6	66	339	0.91
	7	96	121	7	56	339	0.93
	Wall 8						
	9	92	130	7	58	339	0.83
	10	86	118	9	70	339	0.73
	11	78	112	6	60	339	0.64
	12	100	107	5	60	339	0.65
	13	106	121	6	68	339	0.72
	14	76	99	4	56	339	0.56
	15	82	111	4	53	339	0.69
E	2	47	57	54	271	362	0.46
	3	59	51	11	420	362	0.60
	4	66	85	5	402	362	0.77
	5	68	78	12	467	362	0.73
	6	67	91	13	385	362	0.75
	7	72	101	18	385	362	0.92
	Wall 8						
	9	67	85	16	439	362	0.75
	10	63	76	15	453	362	0.68
	11	64	75	17	459	362	0.73
	12	61	76	7	469	362	0.72
	13	58	66	3	420	362	0.68
	14	59	59	13	374	362	0.58
	15	70	85	26	272	362	0.67

Table 4.6. SGH punching shear demands and capacities (cont'd)

Grid		V_G kip	V_S kip	M_G kip-ft	M_S kip-ft	ϕV_n kip	D/C ()
D	2	73	63	93	323	362	0.62
	3	56	91	76	323	362	0.66
	4	58	94	79	358	362	0.66
	5	59	94	82	363	362	0.70
	6	60	106	79	348	362	0.75
	7	59	103	68	354	362	0.66
	Wall 8						
	9	56	95	64	315	362	0.61
	10	52	84	73	306	362	0.56
	11	57	91	79	350	362	0.70
	12	62	98	82	371	362	0.80
	13	56	89	74	333	362	0.72
	14	52	83	73	290	362	0.66
	15	53	78	64	245	362	0.52
C	2	117	93	21	72	339	0.72
	3	85	133	5	63	339	0.75
	4	89	131	4	60	339	0.70
	5	109	131	5	78	339	0.77
	6	115	135	2	59	339	0.87
	7	91	126	3	57	339	0.68
	Wall 8						
	9	73	84	7	56	339	0.54
	10	69	59	10	62	339	0.43
	11	112	115	11	88	339	0.86
	12	116	135	2	80	339	0.91
	13	84	106	5	65	339	0.70
	14	80	103	3	53	339	0.73
	15	78	95	2	51	339	0.64
B	2	105	121	16	73	339	0.78
	3	82	133	3	61	339	0.75
	4	85	121	2	62	339	0.69
	5	106	123	4	69	339	0.77
	6	114	122	4	72	339	0.82
	7	87	129	1	61	339	0.79
	Wall 8						
	9	51	87	0	30	339	0.47
	Dissolver Pit						
	11	107	105	9	76	339	0.74
	12	113	123	2	65	339	0.78
	13	81	113	3	69	339	0.67
	14	77	95	1	55	339	0.72
	15	77	91	1	51	339	0.74

Table 4.7. CJC punching shear demands and capacities

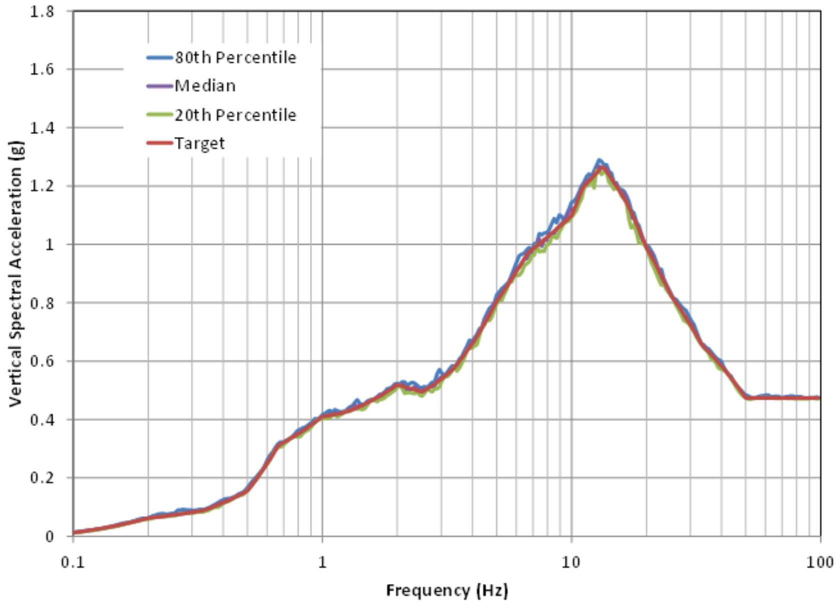
	Grid	V_G kip	V_S kip	M_G kip-ft	M_S kip-ft	ϕV_n kip	D/C ()
G	2	105	72	0	86	302	0.59
	3	95	78	0	81	275	0.63
	4	95	70	0	51	306	0.54
	5	117	87	0	53	273	0.75
	6	117	91	0	51	291	0.72
	7	95	74	0	42	305	0.56
	Wall 8						
	9	86	73	0	44	295	0.54
	10	86	71	0	88	308	0.51
	11	91	66	0	53	308	0.51
	12	117	83	0	51	296	0.68
	13	113	78	0	52	299	0.64
	14	86	61	0	47	305	0.48
	15	91	70	0	40	303	0.53
F	2	98	98	0	55	297	0.66
	3	89	98	0	60	283	0.66
	4	89	87	0	37	303	0.58
	5	109	96	0	36	278	0.74
	6	109	93	0	36	270	0.75
	7	94	82	0	32	289	0.61
	Wall 8						
	9	89	100	0	32	299	0.63
	10	85	89	0	63	278	0.62
	11	85	68	0	41	293	0.52
	12	109	73	0	42	285	0.64
	13	105	77	0	37	286	0.64
	14	85	78	0	33	306	0.53
	15	96	89	0	35	300	0.62
E	2	75	34	80	170	299	0.36
	3	68	34	80	184	301	0.34
	4	68	34	80	214	301	0.34
	5	78	39	80	221	298	0.39
	6	78	39	80	219	298	0.39
	7	73	36	80	202	299	0.36
	Wall 8						
	9	70	37	80	217	300	0.36
	10	65	33	80	214	302	0.33
	11	64	32	80	209	302	0.32
	12	78	39	80	199	298	0.39
	13	74	37	80	192	299	0.37
	14	65	34	80	185	301	0.33
	15	72	44	80	174	298	0.39

Table 4.7. CJC punching shear demands and capacities (cont'd)

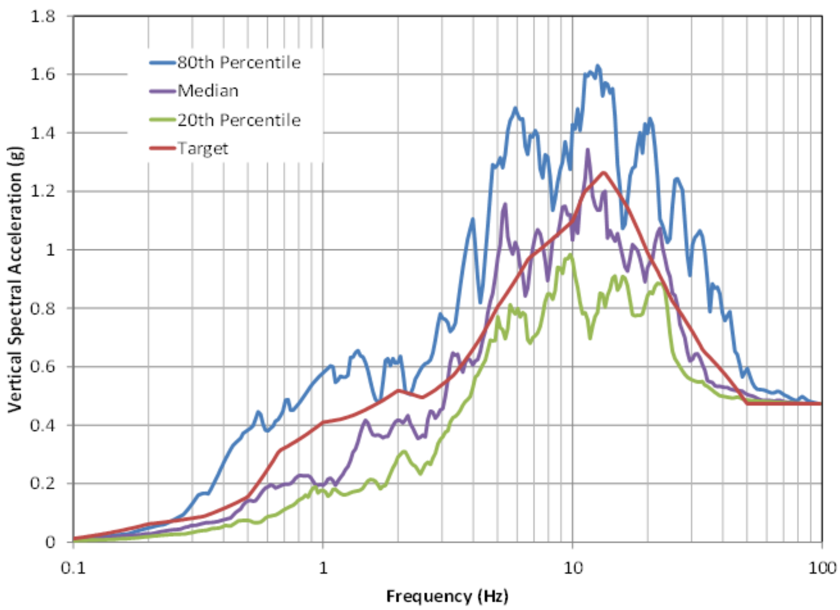
	Grid	V_G kip	V_S kip	M_G kip-ft	M_S kip-ft	ϕV_n kip	D/C ()
D	2	84	54	80	244	308	0.45
	3	69	34	80	233	301	0.34
	4	69	34	80	257	301	0.34
	5	78	39	80	251	298	0.39
	6	78	39	80	243	298	0.39
	7	69	34	80	232	301	0.34
	Wall 8						
	9	62	31	80	228	322	0.29
	10	62	31	80	239	303	0.31
	11	74	37	80	236	299	0.37
	12	78	39	80	234	298	0.39
	13	65	33	80	238	302	0.33
	14	62	31	80	220	303	0.31
	15	62	36	80	196	302	0.32
C	2	116	70	0	117	292	0.64
	3	90	73	0	55	275	0.60
	4	90	88	0	37	302	0.59
	5	109	94	0	36	275	0.74
	6	109	90	0	32	297	0.67
	7	90	74	0	29	306	0.54
	Wall 8						
	9	81	46	0	45	316	0.40
	10	81	41	0	86	300	0.41
	11	104	89	0	62	298	0.65
	12	109	79	0	34	300	0.63
	13	86	67	0	37	309	0.49
	14	81	68	0	32	310	0.48
	15	81	74	0	31	308	0.50
B	2	109	82	0	50	291	0.66
	3	91	71	0	47	306	0.53
	4	91	77	0	46	305	0.55
	5	110	87	0	47	287	0.69
	6	110	89	0	42	297	0.67
	7	91	72	0	36	306	0.53
	Wall 8						
	9	82	41	0	33	317	0.39
	Dissolver Pit						
	11	105	89	0	68	298	0.65
	12	110	81	0	40	299	0.64
	13	87	61	0	43	310	0.48
	14	82	58	0	36	312	0.45
	15	82	67	0	31	310	0.48

Table 4.8. CJC and SGH punching shear results and adjusted SGH D/C

	Gravity V_G (kips)			Seismic V_S (kips)			D/C			Adjusted SGH	
Grid	CJC	SGH	CJC / SGH	CJC	SGH	CJC / SGH	CJC	SGH	CJC / SGH	F_A	D_{80} / C
G 2	105	97	1.09	72	136	0.53	0.59	0.92	0.64	0.82	0.76
G 6	117	125	0.94	91	123	0.74	0.72	0.94	0.76	0.85	0.80
G 13	113	122	0.92	78	150	0.52	0.64	0.87	0.73	0.83	0.73
F 3	89	97	0.91	98	108	0.90	0.66	0.80	0.82	0.84	0.68
F 6	109	107	1.02	93	132	0.71	0.75	0.91	0.82	0.83	0.76
F 7	94	96	0.98	82	121	0.68	0.61	0.93	0.66	0.83	0.77
F 9	89	92	0.97	100	130	0.77	0.63	0.83	0.76	0.82	0.68
E 7	73	72	1.02	36	101	0.36	0.36	0.92	0.40	0.82	0.76
D 12	78	62	1.24	39	98	0.40	0.39	0.80	0.49	0.82	0.66
C 6	109	115	0.95	90	135	0.67	0.67	0.87	0.77	0.84	0.73
C 11	104	112	0.93	89	115	0.78	0.65	0.86	0.76	0.85	0.73
C 12	109	116	0.93	79	135	0.59	0.63	0.91	0.69	0.84	0.76
B 6	110	114	0.96	89	122	0.73	0.67	0.82	0.82	0.85	0.69



a. Spectrum-matched motions



b. Motions scaled by peak ground accelerztion

Figure 4.1. Alternate methods for scaling earthquake ground motions (SGH 2014c)

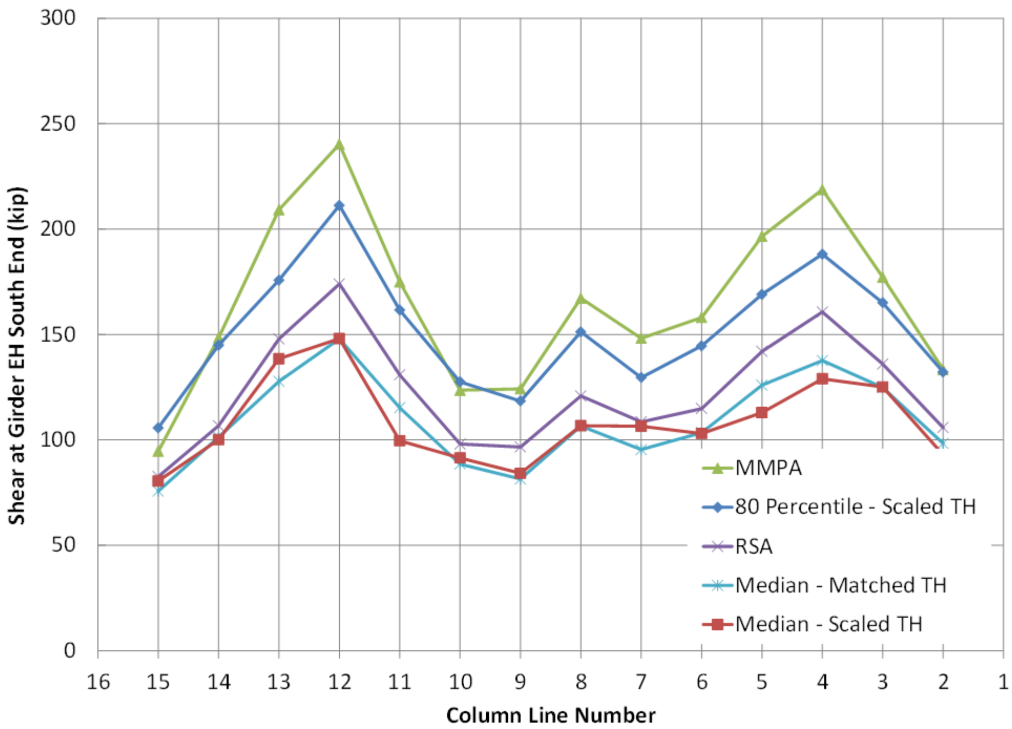


Figure 4.2. MMPA, RSA, and modal response history shear at south end of girder EH (SGH 2014c)

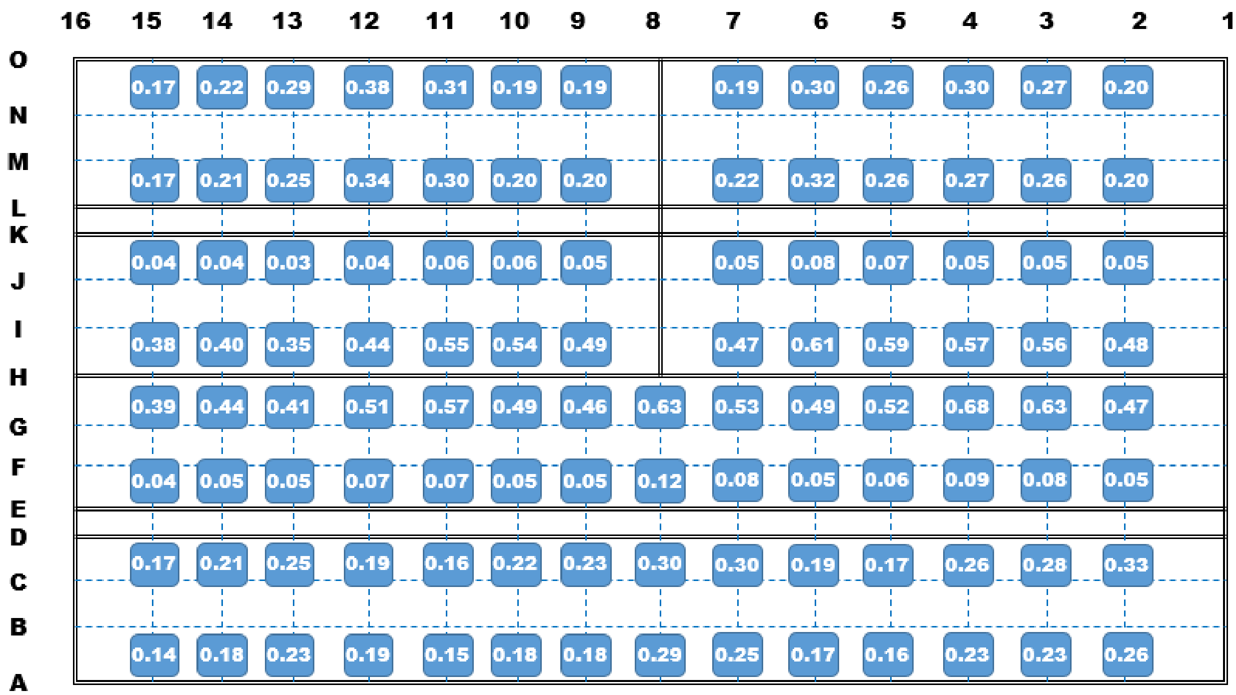


Figure 4.3. AFUP ($\times 10^{-4}$) of roof girder failure due to shear (adapted from Mertz et al. 2015b)

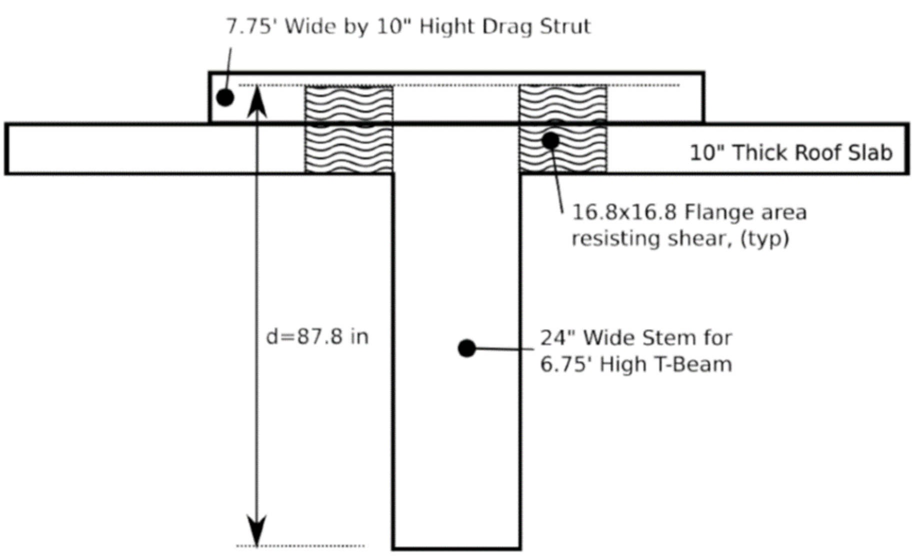


Figure 4.4. Cross section through roof girder at Line 8 drag strut used to calculate shear capacity (courtesy of CJC)

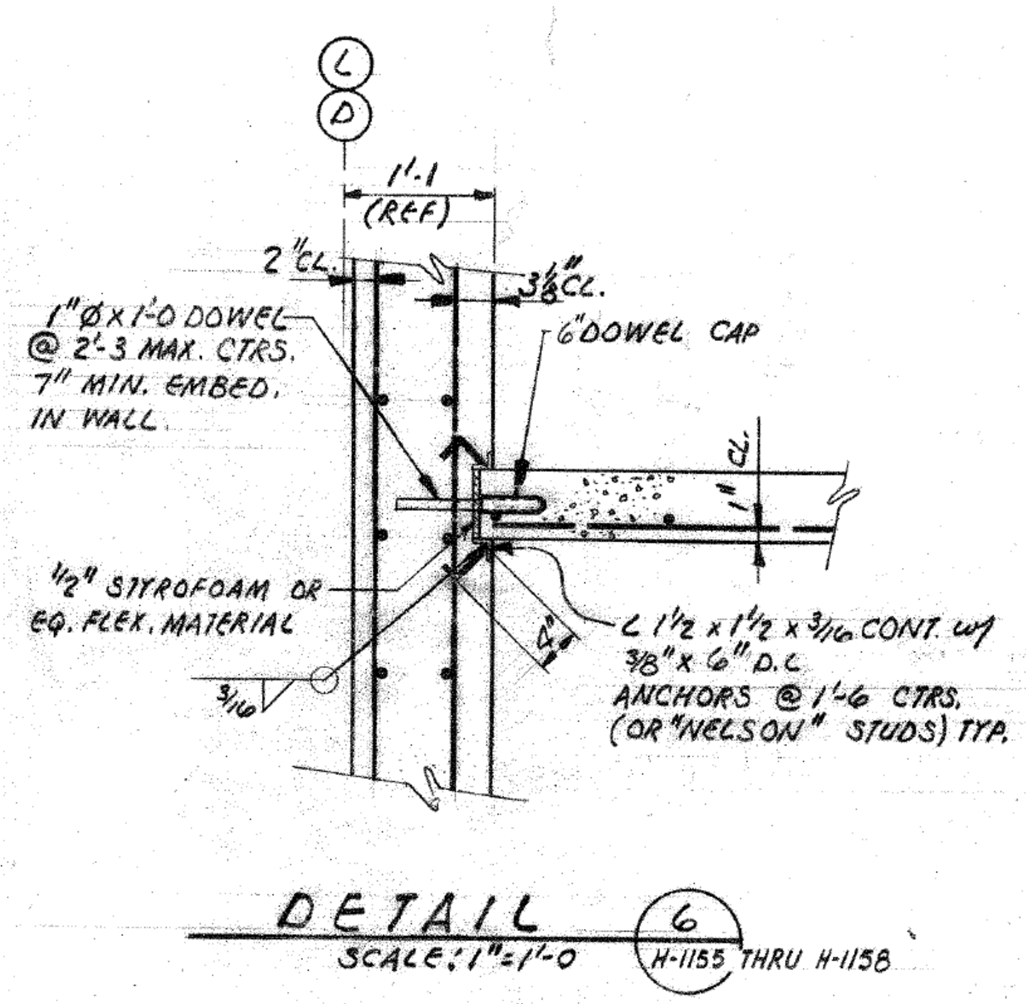


Figure 4.5. Chase floor slab connection to chase wall from sheet 76 of the structural drawings (from Fluor 1977)

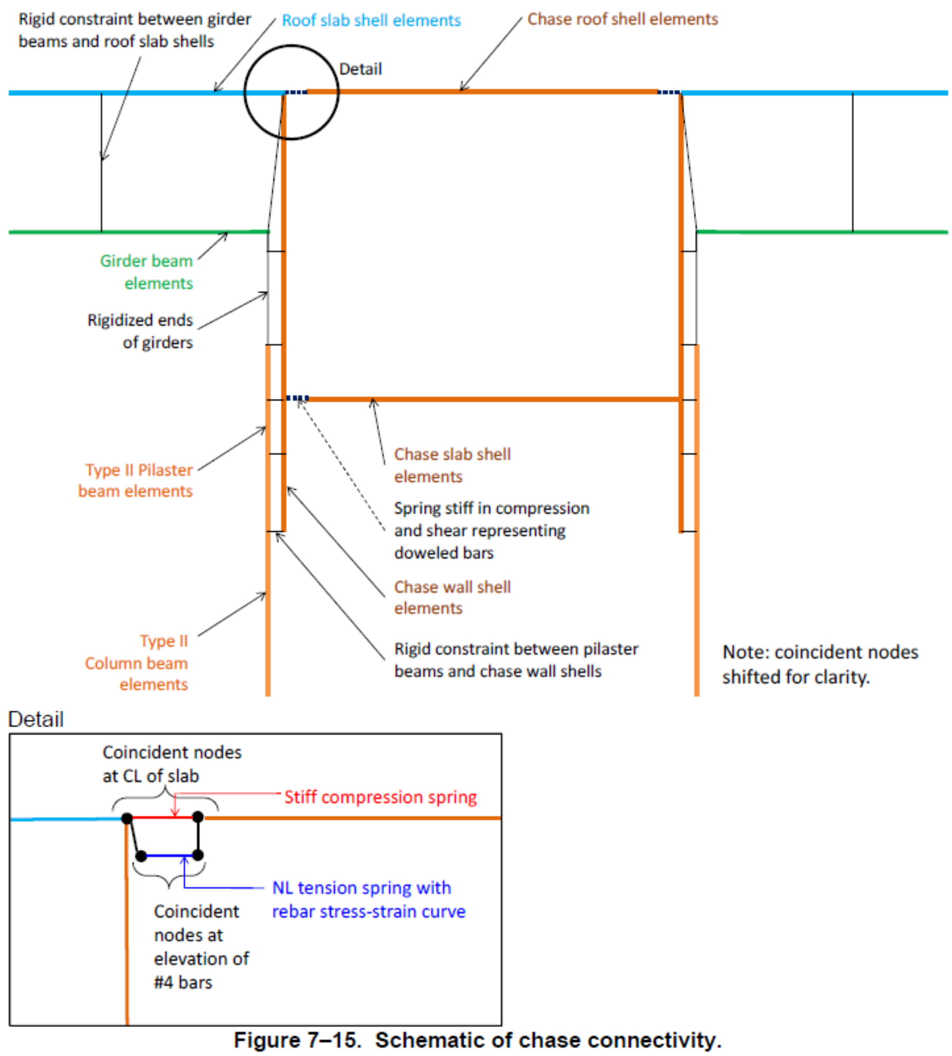


Figure 4.6. SGH model of girder to chase corridor connection (SGH 2014c)

5 Additional Studies

This section identifies studies that must (identified as *required*) and should (identified as *prudent*) be performed by Los Alamos National Laboratory (LANL), in the opinion of the Seismic Expert Panel (SEP).

5.1 Chase Floor Slab Connection

Figure 1.3 (Drawing sheets 36, 61 and 76 from the as-built document set prepared by Fluor 1977) presents a partial east elevation of the interior wall on Line 8 (panel a), an elevation of the roof beams at Line 8 between Lines A and H (panel b), the reinforcement of the roof slab to the service chase (panel c), a typical section through the service chase [with the roof slab excluded] (panel d), and the connection of the service chase floor slab to the service chase walls (panel e). The service chase construction details between Lines K and L are similar (and not addressed specifically here).

At Lines D and E, the negative moment (top) reinforcement in the roof beams are hooked down into the column below, and positive moment (bottom) reinforcement extends straight into the column: see Figure 1.3b. The roof slab over the service chase (Figure 1.3c) was cast after the adjacent roof beams and spans as a simply supported element between the walls of the service chase on Lines D and E. Starter bars from the adjacent roof slab are lapped with the positive moment (bottom) reinforcement in the chase roof slab. The typical section through the service chase (Figure 1.3d) identifies floor slab to service-chase wall connection details: a sliding connection on Lines D and L (detail 6) and a conventional connection on Lines E and K (detail 7). We believe that the designer’s intent with the sliding connection was to prevent axial force transfer through the floor slab. We have no means by which to confirm that either the dowels or the flexible (compressive) material shown in the detail can serve the intended purpose, forty years after construction. If this detail does not function as intended, and the service chase floor slab restrains lateral movement of the column, the column will be captured and very high shear forces may be developed in the column during earthquake shaking. The dashed orange boxes in Figure 5.1 identify the regions of concern to the SEP.

The SEP recommends four studies be performed in regard to this detail: 1) an inspection of the tops of the columns, above the service chase floor slab and below the underside of the roof beams, on Lines D, E, K and L, to gage whether any of the columns have cracked in shear under gravity-induced loadings, 2) a physical inspection of the sliding joint detail (left panel of Figure 3d) at a number of locations along each service chase to determine the current condition of the flexible material, 3) an analysis of the potential for shear failure of the columns and the resultant vulnerability of PF-4, and 4) an analysis of the seismic vulnerability of the exterior roof girders if the examination of 3) shows that the service chase construction provides fixity to the girders. These studies are considered *required*.

The depth of the column, immediately below the underside of the roof beam is about 15 inches, and the distance from the underside of the beam to the top of the service chase floor slab is also about 15 inches. An analysis of the potential for shear failure (task 3) must recognize that a) a shear crack would have to form at a very shallow angle to the horizontal, and b) a shear crack, if formed, would have to propagate through both the column and the adjacent service chase walls for failure (i.e., loss of gravity load resistance) to occur.

5.2 Physical Testing of Column Capitals

Section 4.4 of this report describes the SGH assessment of the column capitals that support the laboratory floor slab. A sample capital is shown in Figure 3.1a. The integrity of the column capitals is critical to the support of the laboratory floor under gravity loadings; the loss of the capitals, for whatever reason, will sufficiently reduce the punching shear perimeter of the slab to trigger a local collapse. The column capitals are not reinforced, as would be routine practice today. Their integrity as a continuum of concrete cannot be guaranteed in the event of design basis shaking because there are no test data available from which to either judge likely performance in the event of design basis shaking or validate numerical tools with sufficient confidence to accurately predict performance.

We recommend that representative slab-capital-column systems be physically tested to simulate gravity and earthquake effects. As a minimum, a sufficient number of tests should be performed, at or near full scale, to characterize the performance of the Type V slab-capital-column assemblies in PF-4. The testing program should provide the raw data and metadata needed to validate numerical models to the standard expected in the nuclear industry, represented here by ASME guidelines (ASME 2006). Earthquake shaking effects of at least 200% DBE shaking should be imposed on the test specimens to enable development of fragility functions for possible later use in a probabilistic risk assessment. If the specimens are badly damaged for the effects of 200% DBE shaking or less, retrofit strategies should be developed and implemented on virgin specimens to help guide Los Alamos National Laboratory (LANL) decision-making. It is expected that fiber-reinforced polymer (FRP) solutions would be most appropriate for strengthening the column capitals—likely similar to the details used to date to retrofit columns in PF-4. Each test should be run through failure, regardless of whether the specimens represent the as-built condition or a retrofitted condition. These studies are considered *prudent*.

5.3 Fragility Analysis

Carl J. Costantino and Associates (CJC) estimated the seismic vulnerability of PF-4 as described in Section 2. We consider the methodology they used to represent best practice in the nuclear industry at this time.

We do not see the need to perform an independent (new) vulnerability assessment at this time. CJC did not include column-capital fragilities in their vulnerability assessment but neither CJC nor SGH can do so until the full-scale tests described in Section 5.2 are performed and the data reduced to develop a family of fragility functions suitable for risk assessment.

If the Department of Energy chooses to perform an independent assessment, we recommend the following: 1) the assessment be performed after the slab-capital-column tests have been completed, the data reduced and the fragility curves have been peer reviewed, 2) the assessment be performed after the service-chase studies of Section 0 have been completed and results assessed, 3) all of the component fragilities be reviewed, including those for the low aspect ratio reinforced concrete shear walls, and 4) nonlinear dynamic analysis of a numerical model of PF-4 be performed to estimate seismic demands. Although we acknowledge that SGH have substantially advanced adaptive multimode pushover analysis, the method has not been formally verified and the bias in the predicted results has not been established. Nonlinear response-history analysis of PF-4, using a model with sufficient detail to compute deformations and forces in key components of the lateral force resisting system, is viable: computer codes such as LS-DYNA

(LSTC 2013), ABAQUS (Dassault 2014) and PERFORM-3D (CSI 2011) could be used for this purpose. Soil-structure-interaction effects could be considered by either time-domain modeling of the soil-structure system or through the use of calibrated soil springs.

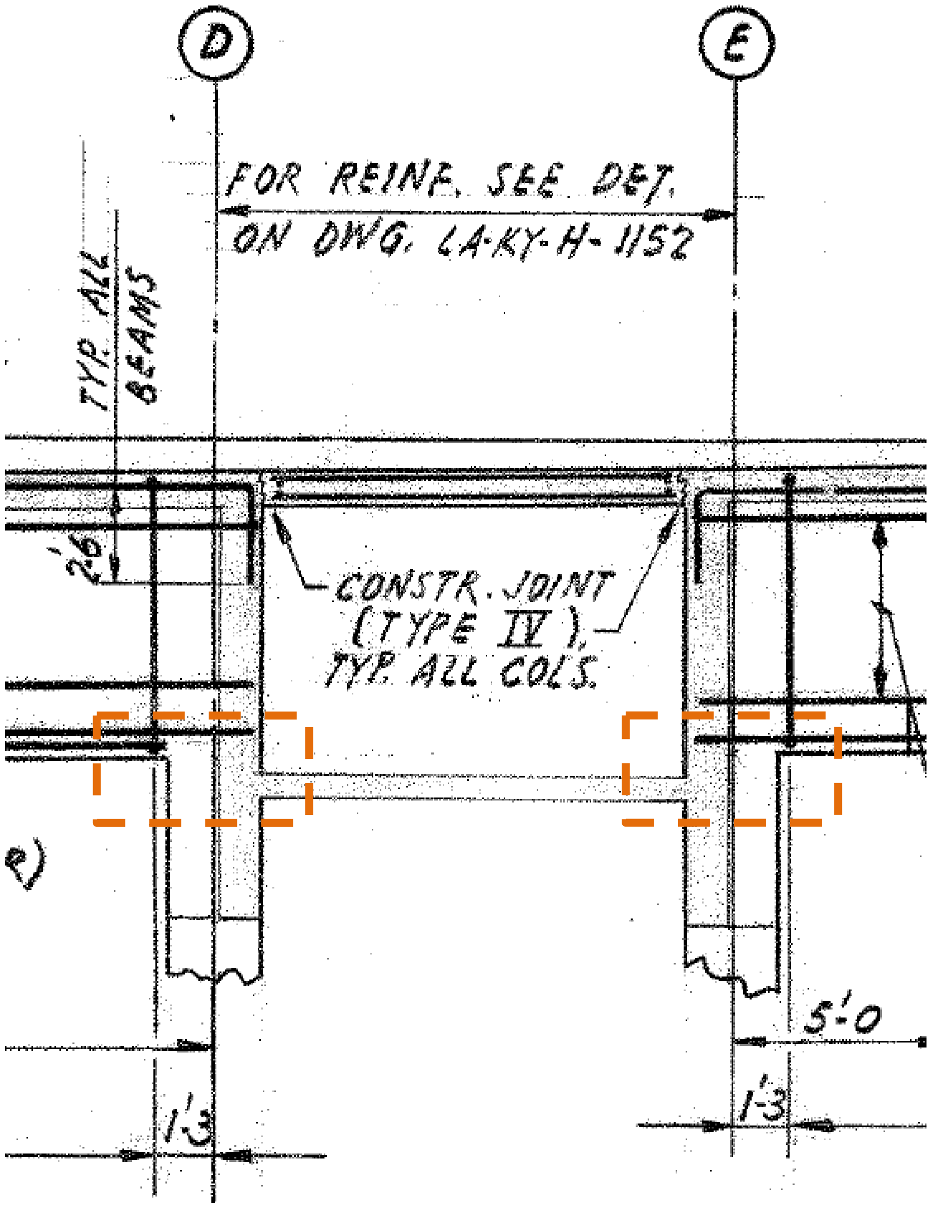


Figure 5.1. Potential column capture by the service chase floor slab (from Fluor 1977)

6 Summary of SEP Recommendations

6.1 PF-4 and the Seismic Requirements of DOE 1020-2002

One important question posed to the Seismic Expert Panel (SEP) was "Does PF-4 meet the seismic requirements of DOE 1020-2002?" If the question is framed in terms of PF-4 being an existing structure, and the permitted reduction in seismic hazard is taken advantage of, the answer is *yes*. If the question is framed in terms of PF-4 being an enduring facility, key to the national security of our country for decades to come, PF-4 should be considered as a new building and its performance judged accordingly. If so, the answer is *likely yes*, with the possible exception of the column capitals that support the laboratory floor slab, as discussed below.

Despite our answer of *likely yes,* the SEP firmly believes that it is prudent and appropriate to - increase the shear capacity of the interior roof girders on either side of Line (wall) H and the exterior girder on Line 8 between Lines A and D. This recommendation is discussed in detail in Section 4.4.4 and summarized in Section 6.2.

The column capitals represent a dilemma to all involved in the project. Although it is the consensus opinion of the SEP that they will not fail for design basis (PC3) earthquake shaking, we cannot quantify the margin against failure because a) there are no test data to support the calculation of a margin, and b) validated numerical models of the slab-capital-column system do not exist, primarily due to a lack of test data. We cannot estimate demand-capacity ratios or an Annual Frequency of Unacceptable Performance (AFUP) without test data. This matter is described in detail in Section 4.5 and summarized in Section 6.2 below.

6.1.1 Demand-capacity calculations

Carl J. Costantino and Associates (CJC) performed a probabilistic seismic demand analysis to estimate demand-capacity ratios for design basis (PC3) earthquake shaking, where gravity demands were added to seismic demands computed at the 80%-ile non-exceedance probability (NEP). Component capacities were calculated at the design level using consensus materials standards. The demand-capacity ratios were less than 1.0 for all of the components CJC evaluated, with the greatest value being 0.99 (and thus very close to the limit) for the interior roof girders on Line 8. Demand-capacity ratios for other components were significantly smaller. On the basis of the CJC calculations, all of the structural components in PF-4 met the demand-capacity acceptance criterion (value less than 1.0).

SGH computed seismic demands using a multi-mode pushover procedure, which has yet to be formally validated. On the basis of SGH's most recent calculations, the greatest demand-capacity ratio for PF-4 is 1.15 for an interior roof girder on Line H. Demand-capacity ratios for other components, including punching shear for the laboratory floor slab, are less than 1.0 (assuming the column capitals remain intact). We believe that the multi-mode pushover procedure is conservatively biased in terms of computing demands, but we are unable to formally quantify the bias. Even after adjusting the seismic demands to the approximate 80%-ile demand (see above), we calculate demand-capacity ratios greater than 1.0 for at least two interior girders.

The differences between the highest CJC- and SGH-calculated demand-capacity ratios for the interior roof girders, after adjustments by the SEP, is 11%, which is not a cause for concern given that very different approaches and different computer codes were used to calculate the seismic demands on PF-4.

6.1.2 Expected performance of PF-4 in terms of target goals

Section 1.6 describes how DOE-1020-2002 and ASCE 43-05 measure performance, namely, at the component level. A calculation of the performance of the PF-4 building, as a whole, would require an understanding of the correlations of individual component performance (or fragilities) and we (and the profession) have no basis for such calculations at the time of this writing.

Vulnerability estimates for each roof girder have been provided by CJC (2015a) and we consider those estimates to be reasonable. Importantly, of all the components evaluated by CJC, the roof girders were the most vulnerable. (The column capitals were not considered vulnerable by CJC.) The Annual Frequency of Unacceptable Performance (AFUP) of the most vulnerable roof girder is approximately 0.7×10^{-4}, which is smaller than the target performance goal of 1.0×10^{-4}. On this basis, we conclude that all structural components of PF-4 evaluated by CJC meet the PC3 performance goal.

In a prior analysis, CJC (2012) had estimated a maximum AFUP of 0.92×10^{-4} for the interior roof girder on Line 8. After considering other partially independent failure modes, the AFUP for PF-4 increased to approximately 1.2×10^{-4}, which is greater than the PC3 performance goal imposed at the component level. Updated calculations of the PF-4 AFUP based on the revised roof girder AFUP of 0.7×10^{-4} have neither been prepared nor reviewed, but it is our expectation that the facility value will be less than or equal to 1.05×10^{-4}. Accordingly, we conclude that PF-4 meets the target performance goal of DOE 1020-2002 for a PC3 facility.

6.2 Proposed Actions for Consideration by LANL

One of the tasks assigned to the SEP was to develop a list of actions for consideration by DOE/NNSA/LANL. We were asked to assign actions to two bins: 1) *required* actions, and 2) *prudent* actions. We do so below, recognizing that PF-4 is an enduring facility, with a lifespan of at least another 30 years, containing national assets that cannot be readily moved or stored elsewhere in our country. These proposed actions represent the consensus opinion of the SEP, and may not be supported by either CJC or SGH.

On the basis of our review of the analyses performed by CJC and SGH, there are three *required* actions.

1. Inspect the tops of the columns, above the service chase floor slab and below the underside of the roof beams, on Lines D, E, K and L, to gage whether the columns have cracked in shear under gravity-induced loadings, and analyze the potential for shear failure of these columns and the resultant vulnerability of PF-4. See Section 5.1 for details.

2. Understand and evaluate if rotational restraint is provided to the roof girders by the slip joint in the service chase slab. This will require physical inspections of this condition in the building. If the joint material is crushed and providing restraint, re-evaluate the roof girders for increased end moments at the chase and any resulting redistribution of girder end shear to determine if roof girder shear reinforcing is prudent along Lines D and L. See Section 5.1 for details.

3. The seismic demands calculated by SGH and CJC are based on live loads significantly reduced from the original design values. Those areas for which reduced live load has been assumed must be posted and administered by LANL. The posted live loads must be no greater than the live load assumed by CJC and SGH for their analysis.

There are three *prudent* actions:

1. Increase the shear capacity of all interior roof girders on both sides of Line H by a minimum of 120 kips using carbon fiber-reinforced polymer (CFRP) sheets, anchored to develop the capacity of the sheets as close as possible to the underside of the roof slab.

2. Increase the shear capacity of the exterior roof girder at Line 8 at lines A and D by a minimum of 60 kips in a similar manner to the interior girders.

3. Perform laboratory testing of slab-capital-column assemblies, representative of the Type V columns as a minimum, in sufficient numbers to a) develop fragility functions, and b) validate numerical tools.

Although soil failure due to excessive bearing pressure is not expected under design basis or beyond design basis shaking, it is clear that there is no consensus on the capacity of the soil beneath PF-4 for either transient (earthquake) or sustained (gravity) loadings. LANL should consider developing recommendations for these capacities to aid in future performance assessments, either at the site of PF-4 or at sites in the immediate vicinity of PF-4. Additional physical testing (e.g., borings, load tests) may be required to support new recommendations.

6.3 Closing Recommendations to DOE

In closing, we provide three additional recommendations to DOE.

1. The relief offered to the evaluation of existing DOE facilities in terms of a reduction in the seismic hazard should be set aside for PF-4 given both its enduring mission to our nation and its expected service life of at least another two or three decades.

2. We see no need to perform another vulnerability assessment of PF-4 at this time.

3. The cost and effort associated with another vulnerability assessment should be directed towards the retrofit measures (interior roof girders on Line H and exterior girder on Line 8 between A and D) and the physical testing of the column capitals.

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Appendix A: List of Documents Initially Provided to the SEP

Los Alamos National Laboratory (LANL) sent the following documents to the SEP on September 30, 2014:

1. File: **PF-4 Seismic Performance Evaluation Revision 2 11-25-13.pdf**: “Evaluation of Existing PF-4 Seismic Analyses Against DOE Standard 1020-2012”. This report was generated at the request of NNSA to determine how the seismic performance of PF-4 measured up to the current DOE requirements in DOE Standard 1020-2012 and to a certain extent summarizes the results of the analyses performed by LANL – CJC & Associates.
2. Folder: **Linear-Elastic Analyses**:
    - File: **SBDO-CALC-08-033-SAC -Rev1_Aug2010.pdf**: Basis for the seismic evaluation of PF-4.
    - File: **SB-DO CALC 10-014 - SSI_w_Appendices.pdf**: Develops demand loads based on probabilistic SSI analysis of PF-4.
    - Folder: **Capacities**: Contains report and appendices documenting member capacities.
    - Folder: **Rev 1 - Seismic Performance Calc 9-13-11**: Contains report and appendices documenting the structural performance of PF-4 to the Design Basis Earthquake.
3. Folder: **Service Chase Roof Slab Testing**:
    - File: **LANL Final Report (February 7, 2013).pdf**: Final report of testing performed at UCSD
    - File: **CJC-PF-4-013-Rev0-SCRS.pdf:** Sensitivity of UCSD testing results on the fragility of the PF-4 structure (extension of nonlinear pushover analysis)
4. Folder: **Nonlinear Pushover Analyses**:
    - Folder: **CJC-PF-4-010_Rev1**: Contains report and appendices documenting the pushover analysis and member/system fragilities
    - File: **CJC-PF-4-011-Rev0.pdf**: “Sensitivity Study of the PF-4 Lab Floor Using Yield Line Analyses”
    - File: **CJC-PF-4-015-r0.pdf**: “Inelastic Behavior of PF-4 Roof Girders”

Los Alamos National Laboratory sent the following documents to the SEP on October 16, 2014:

1. File: **ta55_14-0915(1.sgh_092_seismic_121049-RPT-01-Rev1-bookmarked.pdf**: "Incremental Adaptive Nonlinear Multi-Mode Pushover Methodology for NNSA PF-4 Facility Evaluation", Document ID: 121049-RPT-01, Revision 1, 15 September 2014
2. File: **ta55_14-0919 (1.sgh_092_seismic_121049-RPT-02-Rev0-bookmarked.pdf**: "Characterization of Structural Component Behavior in NNSA Plutonium Facility-4", Document ID: 121049-RPT-02, Revision 0, 19 September 2014

3. File: **ta55_14-1002(1.sgh_092_seismic_121049-RPT-03-Revi0-bookmarked.pdf**: "Seismic Analysis and Evaluation of NNSA Plutonium Facility 4 (PF-4)- Phase I: Code-Like Procedure", Document ID: 121049-RPT-03, Revision 0, 2 October 2014

Appendix B: List of Additional Products Provided by SGH and CJC at the Request of the SEP

Over the course of our evaluation, the SEP received from SGH and CJC various results in response to our requests for additional information following our November 4-5, 2014 kick-off workshop. The responses are too voluminous to feasibly include in their entirety herein as an appendix, and are instead provided in an accompanying electronic document (*SEP-Documents.zip*).

At the workshop, the SEP requested additional information regarding the gravity and earthquake contributions to roof girder and punching shear demands, roof girder capacities, soil bearing pressure and validation analysis for the column capital study. The following documents were provided in response to this request:

Date	From	File Name
12/18/14	SGH	Task 1b - Confirm interior girder shear results 12_18_2014.pptm
12/18/14	SGH	Task 1c - Reduction to Stirrup Shear Strength 12_18_2014.pptm
1/9/15	SGH	Task-1d_GirderAxialStudy_01-09-2015.pdf
1/9/15	SGH	Tasks-1e-1f-1g_GirderTables_01-09-2015.xlsx
1/9/15	SGH	Tasks-2a-2b_Soil_Bearing_Rressure_01-09-2015.pdf
1/9/15	SGH	Tasks-3a-3b-3c_PunchingShear_01-09-2015.pdf Tasks-3be-3c_PunchingTables_01-09-2015.xlsx
1/13/15	SGH	SGH and CJC reporting of roof girder shears and lab floor shear stresses 011315.xlsx
1/14/15	SGH	Task-5a-5b-Column_Capital_Study-1-14-2015.pdf
1/20/2015	CJC	SGH and CJC reporting of roof girder shears and lab floor shear stresses 2015-01-20.xls

Subsequent to receiving this information, the SEP had additional comments and questions for SGH regarding the apparent conservative bias of the MMPA demands (relative to the CJC results) and the methods used for the column capital finite element analysis. SGH provided the following:

Date	From	File Name
2/18/15	SGH	Responses to SEP comments regarding Task 1b Confirmation of Girder Shear 07-02-2015
2/18/15	SGH	Responses to SEP comments regarding Column Capital Evaluation 07-02-2015

The SEP requested supplemental linear elastic analyses be performed by both CJC and SGH to identify and quantify the factors that led to different SGH and CJC conclusions. We requested three static load cases (1 g uniform acceleration in each principal direction) and linear response history analyses. In response, SGH and CJC provided the following results:

Date	From	File Name
1/29/15	SGH	PF4TH01_H1_acc.csv
1/29/15	SGH	PF4TH01_H2_acc.csv
1/29/15	SGH	PF4TH01_Matched.xlsx
1/29/15	SGH	PF4TH01_V_acc.csv
2/16/15	CJC	sup-el-anal-study.pdf
2/16/15	CJC	sup-el-anal-study-data.csv
2/16/15	CJC	abqs-input-files.tar.gz
2/16/15	CJC	beam-fragility-study.pdf

Date	From	File Name
2/18/15	SGH	Task-8-One G Static and Response-History Analysis 02-21-2015
2/18/15	SGH	SGH and CJC reporting of roof girder shears and lab floor shear stresses 2015-02-07

In response to a request from the SEP, SGH provided a refined study of the soil bearing pressures below exterior wall footings and CJC provided an analysis comparing some roof girder demand-to-capacity ratios with their associated failure frequencies:

Date	From	File Name
2/27/15	SGH	Tasks-2b PF-4 Foundation 2-27-2015.pdf
3/9/15	CJC	2015-03-09-discussion.pdf

Additional material provided by LANL included soils information and a calculation regarding the composite failure frequency for roof girders:

Date	From	File Name
11/2/14	LANL	Dames and Moore Appendix F.pdf
11/2/14	LANL	Dames and Moore Foundation Section.pdf
11/2/14	LANL	Kleinfelder - RLUOB Report.pdf
2/22/15	LANL	CAL-14-TA55-STR-024-S Rev 0.pdf

Appendix C: SEP Information Requests

Date	Request for Information
October 31, 2014	Attached. The SEP requested basic information for preparation for the November 4-5, 2014 workshop in Los Alamos
February 3, 2015	Attached. The SEP requested additional information from SGH regarding soil pressures, column capital analysis and calculation of the 80%-ile seismic demands in the context of MMPA.
February 5, 2015	Attached. The SEP requested that both SGH and CJC perform linear elastic analyses of fixed base models.

Appendix D: SEP Meeting Minutes

Date and venue	Minutes
October 3, 2014 (phone)	Attached
November 4-5, 2014 (Los Alamos)	Attached
January 20, 2015 (phone)	Attached
January 28, 2015 (phone)	Products of this meeting include RFI to SGH and CJC for linear elastic, fixed base analysis, as well as requests to SGH for additional information regarding soil stresses, column capital finite element analysis and 80%-ile seismic demands from MMPA. These requests comprise the meeting minutes. See Appendix C.
February 3, 2015 (phone)	Attached
February 12, 2015 (phone)	Attached. The purpose of this meeting was to finalize the agenda for the February 22-23 SEP meeting in San Francisco, and that agenda serves as the meeting minutes.
February 22-23, 2015 (San Francisco)	This meeting culminated in a draft outline of the SEP final report, including draft conclusions and recommendations and assignments for drafting sections of the final report. No other meeting minutes were recorded.
March 17, 2015 (San Francisco)	The purpose of this meeting was to finalize a draft of the SEP report, which was edited and completed during the meeting. No other meeting minutes were recorded.

Appendix E: SGH, CJC, and SEP Correspondence on Rev. 0 Report

Appendix E includes the following:

1. Letter from Dr. Said Bolourchi and Dr. Andrew Sarawit (SGH) to Ms. Joanna Serra (NNSA) dated May 15, 2015.

2. Letter from Dr. Greg Mertz (CJC) to Mr. Lawrence Goen (LANL) dated April 30, 2015.

3. Letter from Dr. Andrew Whittaker (SEP Chair) to Mr. Lawrence Goen (LANL) dated September 1, 2015, in response to SGH Letter (item 1 above).

4. Letter from Dr. Andrew Whittaker (SEP Chair) to Mr. Lawrence Goen (LANL) dated August 29, 2015, in response to CJC Letter (item 2 above).

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

15 May 2015

Ms. Joanna Serra
Department of Energy
19901 Germantown Road
Germantown, MD 20874

Project 121049 – Alternative Analysis and Evaluation of PF-4 Structure

Document ID No. 121049-L-009

Dear Ms. Serra:

Seismic Expert Panel (SEP) issued a report titled "Independent Review of Seismic Performance Assessments for the Plutonium Facility PF-4" on 31 March 2015. The panel compared the results from the code-like evaluation performed by Simpson Gumpertz & Heger Inc. (SGH) and the prior evaluation performed by CJC. Some of their recommendations for upgrading shear capacity of roof girders and testing of the column capitals are consistent with SGH recommendations, but we disagree with a number of their recommendations and representations of SGH evaluations. The scope of this letter is limited to the discussion of their representation of the methodology SGH developed for calculating the seismic demands of the PF-4 structure and accuracy of the demands SGH calculated.

NNSA commissioned SGH to perform an alternate evaluation of PF-4 structure with the main objective as directed in the September 2012 statement of work to *"Develop and execute an acceptable MPA including seismic effects. The MPA results will serve in part as a confirmatory analysis to the static pushover analysis. This is needed to provide additional confidence that a complete list of structural components whose failure could lead to loss of confinement or potentially partial or complete structural collapse and provide an additional method of determining the probability of these potential failures."*

On March 2013, the scope of work was modified to a two-phase approach. Phase 1 scope included developing the Modal Pushover Analysis (MPA) and performing deterministic code-like evaluations of PF-4 to identify weak links in order to rehabilitate the structure. Phase 2 scope included performing fragility analysis using the MPA for the building with "as-proposed fixes from code-like evaluation" to confirm that the building meets the PC-3 performance goal.

SGH developed Multi-Mode Pushover Analysis (MMPA) methodology according to our contract scope of work. We summarized the methodology and supporting validation documentations in Report 120149-RPT-01. This report has been reviewed by DOE Independent Reviewer Panel (IRP), DNFSB, and DOE and our responses to all questions related to the MMPA have been accepted as documented in Attachments 1, 2, and 3 of the report.

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We performed the code-like evaluation of the PF-4 in accordance to our scope of work and summarized our evaluation in Report 121049-RPT-03. The intent of the code-like evaluation was to calculate the demands to at least 80th percentile and compare the demands to 98th percentile capacity. This report was also reviewed by IRP, DNFSB, and DOE, and we received many comments from their reviews. Based on our responses to the comments, most were accepted and closed and some required DOE response and therefore transferred to DOE, and a few were transferred to be closed as part of the Phase 2 scope of work, which is the fragility analysis to evaluate the probabilistic performance goals for the PF-4 facility. The main comment that was transferred to Phase 2 was question #4 from IRP that they indicated the calculated demands from MMPA may exceed 80th percentile.

Section 3.4 of SEP's report states that they "*did not accept the technical basis of the proposed (MMPA) method,*" without providing any reason. SGH was responsive to SEP; we participated in any meeting they requested and presented the results and conclusions per their request and responded to all requests such as linear parametric studies, summarizing results in tabular format. We never received any specific comment or request on MMPA methodology except on the double CQC summation method used in MMPA, which we provided comparison of responses calculated using single and double CQC combinations. The same Section 3.4 acknowledges that *"the calculated demands for PF-4 using MMPA were reasonably close to those calculated by nonlinear dynamic analysis but conservatively biased.*" We find the above two statements of the SEP report to be contradictory that they could not accept the technical basis and at the same time acknowledging the results being reasonable but conservatively biased.

MMPA has sound technical basis described in the report 121049-RPT-01 along with assumptions and limitations of this method. The MMPA validations provided for PF-4 structure in this report are more detailed than what is available for the "commonly used" similar methods. We were never commissioned to validate the MMPA for general use for other structures, and we never claimed that this method can be used for other structures without further validations. Furthermore, MMPA does not provide conservative biased responses for PF-4 type structures compared to the 80th percentile results. The Appendix D of Report 121049-RPT-01 was prepared in response to the IRP comment on demand level calculated by MMPA. Figure D84 of this appendix shows that the demand calculated by MMPA at or below 1 DBE (basically in linear elastic range) is higher than mean but lower than 80th percentile of scaled time history responses, and the MMPA responses above 1.5 DBE (in nonlinear range) become more conservative than the 80 percentile level. The reason for the conservatism in nonlinear range is mainly from smoothing and broadening used for displacement amplification factor curves that were intended by design for use in the code-like evaluation.

As we mentioned above, we intended to calculate the code-like demands to be at least 80th percentile. In order to achieve this goal, we considered 4% modal damping value along with hysterics damping, developed displacement amplification factors using scaled time histories and smoothened and broadened the amplification curves for nonlinear response range, and performed sensitivity analyses considering both with and without soil springs and varying relative stiffnesses of different structural components. Without the above-mentioned considerations, the MMPA results for in the linear elastic range would essentially be same as the traditional response spectrum analysis results which would be somewhat between median and 80th percentile of selected time history responses. Although we could not provide sufficient confirmatory proof for achieving the 80th percentile goal by use of all the above-mentioned considerations, we do not agree with SEP assessment that the implemented MMPA has a conservative bias. Code-like evaluation was intended to be an intermediate step for better understanding of the structural behavior, and these results were not for final structural evaluation step. With DOE and IRP consents, we have deferred the confirmation of the demands to the probabilistic evaluation of this facility of Phase 2. We also clearly stated our concern of the demands level in the executive summary and conclusion sections of the report 121049-RPT-03:

> "*Since there are inherent uncertainties in the code-like evaluation both for demands and capacity calculations and lack of detailing for structural integrity and lack of redundancy in structural systems that could result in sudden collapse of the building due to brittle failure, the structural performance of this facility should be evaluated in probabilistic basis with stringent acceptance probabilities to properly evaluate structural performance for meeting the PC-3 requirements*"

> "*The members are assumed to meet the code-like evaluation when the effective demand is less than member capacity ($D_e/C<1.0$). However, due to inherent uncertainties in achieving 80th percentile demand and 98th percentile capacity, brittle failure modes with D_e/C exceeding 0.85 are considered susceptible to failure during a design basis earthquake event*"

In summary, SGH's position is that MMPA is an advanced methodology validated with significant examples corresponding to PF-4 structural behavior. This methodology does not necessarily result in conservative bias responses. Our goal was to calculate at least the 80th percentile responses for the code-like step evaluation by using sensitivity studies and analysis parameters selected. The code-like results were not calculated to be used for probabilistic evaluations.

Sincerely yours,

Said Bolourchi
Senior Principal

Andrew T. Sarawit
Senior Project Manager

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1921 Camino Durasnilla
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April 30, 2015

Mr. Lawrence Goen
Engineering Services Division Leader
MS K575
Los Alamos National Laboratory
P.O. Box 1663
Los Alamos, New Mexico 87545

Dear Larry

Thank you for forwarding a copy of the PF-4 Seismic Experts Panel (SEP) report[1] "Independent Review of Seismic Performance Assessments for the Plutonium Facility PF-4" on April 28, 2015. In general I concur with the SEP assessment of our results and understand that their proposed path forward presents an opportunity to obtain closure for the outstanding technical concerns.

However, I do want to make you aware that I take issue with two portions of the SEP report.

1. In Section 4.3.2, third paragraph. In my opinion, the following sentence is based on a misunderstanding.

   > *. . . The CJC capacity for the interior (Line 8) EH girder is greater than the other girders, namely, 443 kips, because this calculation a) includes area beyond the web, including the drag strut and part of the web, and b) utilizes ACI 318 Equation (11-5) without reduction for axial tension but including drag strut rebar to calculate ρw. . . .*

   Thus, I disagree the following statement in the same paragraph.

   > *. . . We believe that the SGH calculation of capacity is likely conservative and the CJC calculation is unconservative. . . .*

   I would like to make the following points regarding these statements.

   - The 443 kip capacity was prepared in response to a SEP inquiry and reported in an Excel spreadsheet template that was prepared by SGH. The 443 kip capacity does not appear in any of the CJC calculations.
   - Combined axial tension and shear were considered in the performance evaluation (SB-DO CALC-10-015) and is the reason that LANL upgraded the seismic capacity of the PF-4 roof by installing a drag strut.

[1]Rev 0, Dated 3/31/2015

- Axial tension was included in the fragility evaluation (SB-DO CALC-10-015) by assigning all axial tension to the drag strut and omitting the shear capacity of the drag strut.
- Combined axial tension and shear are considered in the individual beam fragility study "Seismic Fragility of Individual PF-4 Roof Girders" dated 2/16/15. Note that this study was prepared at the request of the SEP and is included in Appendix B of the SEP report as "beam-fragility-study.pdf" dated 2/16/15.
- The 443 kip capacity does not include the area beyond the web as stated in the SEP report. This was clarified in an attachment (2015-03-09-discussion.pdf) to an 3/9/15 e-mail addressed to one of the SEP members. This attachment is referenced in Appendix B of the SEP report. Note that the 433 kip capacity does include the drag strut longitudinal rebar, which is allowed per ACI 318 Equation 11-5.
- The SGH Excel spreadsheet did not identify a tension demand and I thought that tension had been omitted from the SGH capacity. Thus, I provided a capacity that was was believed to be compatible with the SGH capacity and clearly identified that axial tension was not included in the Excel spreadsheet.
- The 3/9/15 clarification (2015-03-09-discussion.pdf) includes a reduction for axial tension, $\phi Vn = 0.842\phi Vc + \phi Vs$. Applying this reduction to the concrete portion of the 443 kip capacity yields a shear capacity of 390 kips.
- The primary reason that the CJC capacity is higher than the SGH capacity is because SGH neglected the strength of the longitudinal T-beam rebar and drag strut rebar crossing the crack plane.

2. The last paragraph of Section 4.3.5 begins:

   > *The other roof girder of immediate concern is the exterior girder AD on Line 8. Notwithstanding the low AFUP computed by CJC (see Figure 4.3), but based on the high D/C computed by CJC (see Table 4.5), we recommend increasing its shear capacity at both ends. ...*

   In my opinion, the following statement is not accurate.

   > *The low AFUP computed by CJC was based on potentially unconservative assumptions, namely, the contribution to the shear resistance from flange (slab) and drag strut concrete.*

   From a behavior perspective, the shear failure of a T-beam occurs on an inclined crack plane. The inclined crack plane must cross the T-beam flange and on Gridline 8, the crack plane must also cross the drag strut. Thus, some portion of the roof flange and the drag strut will resist shear in addition to the T-beam web.

   This behavior is confirmed by shear test of T-beams, reported by ACI 426R, which indicates that a portion of the flange is also effective in resisting shear. A figure from

ACI 426R, reproduced in ‘‘Seismic Fragility of Individual PF-4 Roof Girders’’ (2/16/15), shows several tests with a 20% to 30% increase in shear strength due to the shear strength of the T-beam flanges. The SEP report does not appear to recognize either this behavior or the ACI 426R test data. The flange shear resistance used to develop the AFUP is based on the shear strength area from ACI 426R.

As discussed above, the D/C ratio reported in response to the SEP data request (Excel spreadsheet) neglected the contribution of the flanges and drag strut to shear capacity.

In the November SEP review meeting, both CJC and SGH were told that we would be given a draft of the report to review for factual accuracy before the report was issued. We, CJC, were not given that opportunity, as I did not see any of the material beyond an early (3/16) draft of Chapters 1 through 3 before April 28. In my opinion, the SEP characterization of certain aspects of my work as unconservative, without allowing me an opportunity to provide a clarification before publication, is at a minimum unfair and possibly unprofessional.

In summary,

- The SEP characterization of the CJC shear capacity as ‘‘unconservative’’ in Section 4.3.2 of the their report is based on a misunderstanding of the CJC data.
- The SEP characterization of ‘‘potentially unconservative’’ for considering the shear resistance of the drag strut and T-beam flange in Section 4.3.5 of the their report neglects the available test data and does not account for the failure plane crossing these elements.

I strongly suggest that the SEP report be revised to address the issues identified in this letter. I am available to discuss this if further clarification is needed.

Sincerely yours,

Greg Mertz, PhD, PE
CJC and Associates

cc: Carl J. Costantino

September 1, 2015

Mr. Larry Goen
Division Leader, Engineering Services
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P.O. Box 1663, MS M990
Los Alamos, NM 87545
United States of America Via email: lgoen@lanl.gov

Re: PF-4 Seismic Expert's Panel Response to SGH 5/15/2015 Letter to Joanna Serra

Dear Mr. Goen,

The PF-4 Seismic Expert Panel (SEP) distributed Revision 0 of "Independent Review of Seismic Performance Assessments for the Plutonium Facility PF-4" on March 31, 2015. Dr. Said Bolourchi and Dr. Andrew Sarawit of Simpson, Gumpertz and Heger (SGH) reviewed that version of the report and provided feedback to Ms. Joanna Serra of DOE in a letter dated May 15, 2015.

In that letter SGH wrote (numbers in [*] added for clarity):

> [1]: Section 3.4 of SEP's report states that they "*did not accept the technical basis of the proposed (MMPA) method*," without providing any reason.
>
> [2]: The same Section 3.4 acknowledges that "*the calculated demands for PF-4 using MMPA were reasonably close to those calculated by nonlinear dynamic analysis but conservatively biased*."
>
> [3]: We find the above two statements of the SEP report to be contradictory that they could not accept the technical basis and at the same time acknowledging the results being reasonable but conservatively biased.
>
> [4]: Although we could not provide sufficient confirmatory proof for achieving the 80th percentile goal by use of all the above-mentioned considerations, we do not agree with SEP assessment that the implemented MMPA has a conservative bias.

Our responses to these statements, which in our opinion are the key items in the SGH letter, are provided below in order.

<u>Item 1</u>

This statement is not a criticism of SGH's implementation of the method for the Phase 1 evaluation, it is simply a fact that the Multi-Mode Pushover Analysis (MMPA) has not yet been vetted by the profession, and does not have a pedigree of successful application required for general acceptance. The SEP was not tasked with verifying the MMPA method. The SEP (Kennedy, McDonald, Morgan, Whittaker, Wyllie) relied on a prior determination by an Independent Review Panel (IRP: McDonald, Morgan and Whittaker) tasked by DOE to review the Phase 1 SGH study: a code-type evaluation of PF-4.

The IRP evaluated SGH's implementation of the MMPA for Phase 1 investigation and found that it provided reasonable results for the PF-4 structure subject to code-level shaking at that site. The IRP was unable to independently verify the MMPA method in general, and indeed, the IRP was never tasked with doing so. In their Phase I study, SGH compared the results of analysis of simple models (by comparison with a numerical model of PF-4) performed using nonlinear dynamic procedures and MMPA. In the elastic range of response, displacements and forces calculated using the two methods were similar, with MMPA generally predicting slightly greater demands. Given that the MMPA appeared to overpredict elastic response, as one might expect from a procedure based on response-spectrum analysis, the IRP could raise no objection to its use for a code-type analysis of PF-4 for which response to design basis shaking was by-and-large elastic. The IRP did not endorse the use of MMPA for analysis of a) structures other than PF-4, and b) the response of PF-4 further into the nonlinear range. In fact, several letters were issued to NNSA cautioning against the general use of MMPA, and the IRP comment log contains several open items establishing their concern that MMPA could not produce results that would be useful in a fragility analysis.

The technical basis of the MMPA is not yet proven. We are not suggesting it cannot be proven but have yet to see the comprehensive vetting that would elevate the method to a status sufficient for its widespread adoption by the structural engineering community in the United States. We note that the MMPA has not been rigorously peer reviewed by others and not used by expert design professionals other than SGH.

<u>Item 2</u>

This statement refers to the IRP review of the Phase 1 SGH study, as described in item 1 above. Analysis of models using the MMPA predicted greater demands than those calculated by nonlinear dynamic analysis, noting that the degree of nonlinear action was small. For this reason the IRP could raise no objection to the use of MMPA for the code-type evaluation of PF-4.

<u>Item 3</u>

We do not find the SEP statements ([1] and [2]) to be contradictory. Statement 1 is clear and applies to the general use of MMPA by the structural engineering profession: an adequate technical basis for the broad use of MMPA has not been provided. Statement 2 is also clear and applies to the specific use of MMPA for the PF-4 code-like evaluation: it simply compares results of two methods of analysis and nothing more.

<u>Item 4</u>

The SEP spent considerable effort reconciling the demands and capacities of the independent investigations by SGH and Carl J. Costantino and Associates (CJC). After accounting for differing assumptions and modeling techniques, it appeared to us that the demands calculated by SGH using MMPA were higher than the 80%-ile loads calculated by CJC using more traditional methods. We attribute those higher demands (bias), at least in part, to the MMPA method itself. Systematic quantification of that bias for the PF-4 structure would take considerable effort outside the scope of the SEP, and the IRP and SEP understood that to be a task contemplated for the Phase 2 investigation.

At least five challenges remain with the use of the MMPA for the nonlinear analysis of PF-4 and other building structures: 1) verification of the underlying mathematics, 2) peer review of the MMPA by the design professional community, 3) the need to produce site-specific displacement amplification factors 4) quantification of the bias in the MMPA with respect to results of nonlinear dynamic analysis, which is likely specific to the building and its siting with respect to active faults, and 5) lack of prior use by other expert design professionals.

-o-

As we (the SEP) have stated on several occasions, the quality of the work of both SGH and CJC has been outstanding. Previously, the IRP noted on a number of occasions that the quality of the Phase 1 SGH work was extremely high and that the use of a multi-mode pushover analysis method had been imposed on SGH. SGH substantially advanced multi-mode pushover analysis (normally the purview of academics given years to complete the task) and for that they deserve a great deal of credit.

Throughout the process, Drs. Bolourchi and Sarawit were very helpful and forthright regarding any questions about their analysis or conclusions. We are very grateful for these important contributions because it facilitated our work for LANL and DOE.

Do not hesitate to contact us if additional information is needed. To expedite delivery of this letter, I am signing on behalf of the SEP, each of whom, bar me, is copied below.

Sincerely yours,

ANDREW WHITTAKER, PH.D., S.E.
SEP Chair

Cc: Robert Kennedy, Ph.D., NAE, SEP
Brian McDonald, Ph.D., S.E., SEP
Troy Morgan, Ph.D., P.E., SEP
Loring Wyllie, S.E., NAE, SEP
Chuck Keilers, Ph.D., NNSA
Joanna Serra, NNSA

August 29, 2015

Mr. Larry Goen
Division Leader, Engineering Services
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United States of America

Re: PF-4 Seismic Expert's Panel Response to CJC's 4/30/2015 Letter to Lawrence Goen

Dear Mr. Goen,

The PF-4 Seismic Expert Panel (SEP) distributed Revision 0 of "Independent Review of Seismic Performance Assessments for the Plutonium Facility PF-4" on March 31, 2015. Dr. Greg Mertz of CJC and Associates reviewed that version of the report and provided feedback to Mr. Lawrence Goen of LANL in a letter dated April 30, 2015. In that letter Dr. Mertz strongly suggested that the SEP revisit two statements in the report that Dr. Mertz believes were made in error, and that the SEP consider revising the report. In addition, Dr. Mertz expressed his disappointment at not having the opportunity to discuss these issues with the SEP prior to release of the Revision 0. Herein we respond to each of Dr. Mertz's concerns.

Issue 1: Shear Capacity of Roof Girder 8 at Gridline H

Section 4.3.2 of Rev.0, Page 30, states the following:

> *The CJC capacity for the interior EH girder is greater than the other girders, namely, 443 kips, because this calculation a) includes area beyond the web, including the drag strut and part of the web, and b) utilizes ACI 318 Equation (11-5) without reduction for axial tension but including drag strut rebar to calculate* ρ_w *... We believe that the SGH calculation of capacity is likely conservative and the CJC calculation is unconservative.*

The roof girder along Gridline 8 differs from other gridlines because of the recent addition of a reinforced concrete drag strut directly above. The drag strut has a cross section of 10 inches high by 7 feet, 8 inches wide, and is reinforced in the longitudinal direction with 36 #11 bars. In their evaluation of PF-4, Simpson Gumpertz and Heger (SGH) assumed, conservatively, that the drag strut does not contribute to the shear capacity at Grid 8H; the SGH capacity used to calculate demand-to-capacity (D/C) ratios was 285.7 kips. In contrast, CJC recognized and quantified extra shear capacity provided by the drag strut; the CJC capacity used to calculate D/C ratios was 443 kips, or 55% higher than the associated SGH strength.[1] In our report, we described the SGH capacity as likely conservative (because it ignored the capacity provided by the strut), and the associated CJC capacity as (likely) unconservative.

The SEP is grateful to Dr. Mertz for correcting / clarifying our description of the CJC shear capacity calculation in Section 4.3.2:

1. As stated above, the capacity reported by CJC did not include the reduction for combined tension plus shear. CJC did consider this reduction in other calculations, but, in an effort to simplify the

[1] Another reason for the difference is that SGH reduced their capacity for potential tension in girder, whereas CJC reported their capacity without such a reduction (for comparison purposes). Using the basis equations and techniques in the CJC description of the 443 kip capacity, the SEP calculates that the corresponding tension-reduced CJC capacity to be 390 kips.

SEP efforts to reconcile the SGH and CJC values, they did not include it in a response to a SEP request. The SEP report should have been clearer in this regard, and should have compared the SGH capacity of 285.7 kips with the associated CJC capacity of 390 kips (not 443 kips), both of which include the reduction for combined tension plus shear. The characterization of the CJC calculation and the comparison to the SGH value will be clarified in the next revision of the SEP report.

2. Upon review, the SEP report is not correct in its description of the CJC calculation of this shear capacity. Revision 0 of the SEP report incorrectly states that the CJC capacity was based on a concrete area that included not only the effective web ($b_w d$) but also a portion of the "T" flange (composed of the slab and strut); this is not the case. The CJC-reported shear capacity of 443 kips is based on the girder cross section alone (no strut), but with the amount of longitudinal steel (for ρ_w in ACI 349-06 Equation 11-5) increased by the area of the 36 #11 bars of the strut. The description of the CJC shear capacity calculation will be corrected in Revision 1 of the SEP report.

However, the SEP maintains that the CJC calculation of capacity is potentially unconservative in the context of a code-like evaluation of the girder capacity for the following reasons:

1. There is no explicit provision in ACI 349-06 for the shear capacity contribution of a post-installed, heavily reinforced tension member atop an existing girder. As such, CJC estimated the combined girder/strut shear capacity using the ACI 349 code provisions and ACI 426R.[2] Although we agree with CJC that, notwithstanding the potentially high demands on the strut in its role as a collector, the combined shear capacity of the girder and strut is likely higher than the girder alone, the CJC calculation is not in strict conformance with ACI 349 and thus not appropriate for a code-like evaluation.
    a. To account the longitudinal reinforcement of the girder and drag strut, CJC increases the shear capacity in accord with ACI 349-06 Equation 11-5. However, ACI 349 states the equation is for members subject to shear and flexure *only* (emphasis added). A strict interpretation of the code would not allow increasing the shear capacity due to the presence of longitudinal steel (when significant axial tension could be present).
    b. To account for concrete in the flange of the effective "T" section formed by the slab and strut over the girder web, CJC increased the girder shear capacity in accord with a relationship proposed by Zsutty as described in ACI 426R. The relationship is based on tests of "T"-shaped members, which demonstrated increased shear capacity associated with a portion of the flange concrete immediately adjacent to the web. However, ACI 426R states that this proposal was not incorporated into code provisions.
    c. For members subjected to significant axial tension, the ACI 349 directs the user to Equation 11-8, which reduces the capacity for tension, with no increase associated with longitudinal steel or "T" sections. Equation 11-8 predicts a substantially lower shear capacity, even if the concrete area is increased per the T-beam proposal in ACI 426R.
2. While the SEP recognizes that the drag strut longitudinal steel and additional concrete area would likely contribute shear resistance to the girder, we feel that ignoring these contributions is a reasonable assumption for a code-like evaluation. The drag strut was sized to resist collector loads, and a code-like evaluation should not utilize its reinforcement for other purposes. (Modern building codes recognize the essential nature of a drag strut/collector, and require them to be designed to remain essentially elastic/undamaged.) The SEP feels that all of the reinforcement in

the drag strut should be available to resist drag loads; shear and other load effects should be fully resisted by independent and redundant load paths.

Issue 2: Shear Capacity of Roof Girder 8 at Gridlines A and D

Section 4.3.5 of Rev.0, Page 41, states the following:

> *Notwithstanding the low AFUP computed by CJC (see Figure 4.3), but based on the high D/C computed by CJC (see Table 4.5), we recommend increasing its shear capacity at both ends. The low AFUP computed by CJC was based on potentially unconservative assumptions, namely, the contribution to the shear resistance from flange (slab) and drag strut concrete.*

This issue is very similar to the first, and involves shear capacity of the girder along Gridline 8, this time the exterior span. The SEP recommended retrofit of the outside girder based on high CJC D/C ratio at each end, but noted that those D/C ratios were not consistent with CJC's associated annual frequency of unacceptable performance (AFUP). The SEP requested CJC's help in reconciling these two measures, and CJC provided a detailed explanation.[3] In short, the capacity used to calculate the D/C ratio was not increased to include flange or strut concrete shear area, while the AFUP capacity was increased. Recalculating the D/C ratios using strengths increased for flange/strut shear areas results in more consistent D/C and AFUP values. However, for the reasons listed for the issue above, the SEP feels that it is not appropriate to use higher capacities than those allowed by strict conformance with the code when performing a code-like evaluation of D/C ratios. As such, we feel that the D/C ratios based on more conservative code-based strengths provide a more appropriate basis for our retrofit recommendation.

Issue 3: Opportunity for Parties to Discuss and Challenge SEP Report Statements

The SEP is disappointed to learn that Dr. Mertz feels he was treated unfairly by our review, and particularly by our characterization of two of his capacities as *(likely) or potentially unconservative*. Our use of the term unconservative in no means implies that we think the CJC evaluation does not meet (exceed) the professional standard of care or that it could somehow lead to compromised building safety. The term was intended as a relative measure of the degree of conservatism we were using in a strict code-like evaluation, as opposed to attempts to achieve more accurate estimates of strength by going beyond simple code calculations. As we have stated on several occasions, the quality of the work of both CJC and SGH has been outstanding.

Throughout the process, Dr. Mertz was very helpful and forthright regarding any questions about his analysis or conclusions. Delivering a report to meet a deadline and allowing ample time for discussion is always a challenge. However, it was always our intent to discuss the report with both CJC and SGH, and incorporate their feedback, where we feel it's appropriate, in Revision 1. We still hope to have those discussions.

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[3] 2015-3-09-discussion.pdf

Do not hesitate to contact us if additional information is needed. If you agree, we will update the Revision 0 report per the above proposals. To expedite delivery of this letter, I am signing on behalf of the SEP, each of whom, bar me, is copied below.

Sincerely yours,

ANDREW WHITTAKER, PH.D., S.E.
SEP Chair

Cc: Robert Kennedy, Ph.D., NAE, SEP
Brian McDonald, Ph.D., S.E., SEP
Troy Morgan, Ph.D., P.E., SEP
Loring Wyllie, S.E., NAE, SEP
Chuck Keilers, Ph.D., NNSA