Seismic Evaluation of City of Salem Civic Hall

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1	1/18/2017	Revision per Reviewer Comments

Executive Summary

MSC Engineers, Inc. has completed a seismic evaluation of the City of Salem Civic Center in Salem, Oregon. This evaluation is based on the agreement that all critical emergency response functions will be relocated to a new Police Facility. This allows the Risk Category to be reduced from IV to III, and the performance objective reduced from "Immediate Occupancy" to "Life Safety" with and Importance Factor, I_e = 1.25. Analysis of the structure considered seismic loads required by the current building code. The use of the term deficiency intends to highlight possible structural instability during an earthquake of significant magnitude, not as a result of gravity loading in the absence of such an event.

As described in the report, four cases have been modeled. Case 1 represents the as-built condition as shown in the original construction drawings, Case 2 shows the as-built configuration with the existing canopy roof structure removed. Case 3 illustrates the recommended shearwall upgrades retaining the existing expansion joints. Finally, Case 4 includes the recommended shearwall upgrades including tying the building wings together across the existing expansion joints. Therefore, we have found that after implementation of the proposed seismic upgrades the structure achieves an adequate performance level as related to current code seismic and serviceability requirements. Some recommended non-structural upgrades are proposed to provide an adequate level of safety with regards to falling hazards during a seismic event. These recommendations refer to proper attachment and bracing of partition walls, suspended ceilings, HVAC mechanical equipment and fire suppression piping.

In addition to the proposed structural and non-structural upgrades, we have explored the necessity for waterproofing the parking structure ramps, driveways and roof slab as well as the plaza at the first-level between the building winds after removal of the roof canopy. Proper waterproofing and re-conditioning of the walking surfaces is essential for the preservation of the structural waffle slab.

A contractor-based construction cost estimate for the structural and non-structural work described in our seismic evaluation was provided by Dalke Construction with the intent to get a representative contractor's view on the approximate cost associate with our proposed conceptual upgrade plan. Therefore, we are estimating an approximate cost of $12,894,500. We expect that these figures will be merged with those prepared by staff for the broader scope of mechanical and electrical upgrades as well as large capital improvement and deferred maintenance aspects.

If any questions or comments arise as to the findings or recommendations presented herein, please do not hesitate to contact us at (503) 399-1399. We thank you for your time and consideration and look forward to working with you to bring these upgrades to fruition.

Respectfully Submitted,
MSC Engineers, Inc.

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Section 1. Seismic Evaluation

1.1 Introduction

This report presents methodology, assumptions, findings, and recommendations for the structural seismic evaluation of the City of Salem Civic Center, council chambers, walkways and parking structure, Figure 1. Our evaluation followed the outline of tasks presented in the proposal letter dated July 1, 2016. Additionally, our office performed a cursory peer review of the evaluation report by BergerABAM date 2/14/2014 for the City of Salem Central Library (Ref.3.3)

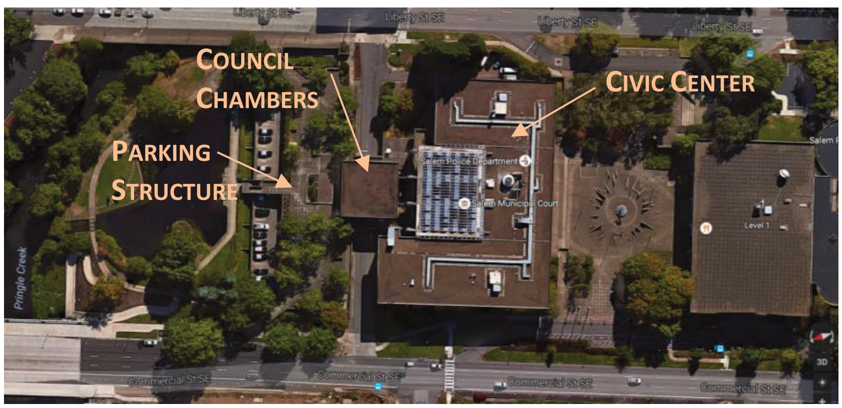

Figure 1. Salem City Hall

The intent of this evaluation was to analyze and assess the existing structure to develop an understanding of current capabilities to withstand a design seismic event as dictated by the current building code. Furthermore, the evaluation examined the viability of adding targeted owner elected structural seismic upgrades as part of a concurrent architectural renovation and programming of the building. This report presents the procedures and assumptions used in our analysis as well as findings and upgrade recommendations. Schematic drawings, detailed analysis and engineering calculations are also provided for reference. The recommended structural improvements were shared with Dalke Construction to provide preliminary construction estimates. This evaluation report is exclusive to the structural ramifications and excludes the costs and adequacy of existing Architectural, Electrical, Mechanical, Plumbing, ADA, Fire and Life System Analysis, and temporal relocation of offices during construction. We concur that City staff and Architectural consultants are best suited to update prior assessments of these service needs.

This evaluation is based on the idea that all critical emergency response functions will be relocated to a new Police Facility. This allows the Risk Category to be reduced from IV to III, and the performance objective reduced from "Immediate Occupancy" to "Life Safety" with and Importance Factor, I_e = 1.25.

1.2 Building Information

The Civic Center site is located at 555 Liberty Street SE in Salem, Oregon and was constructed in 1970 from design drawings dated March 16, 1970 prepared by Payne & Settecase, AIA, Charles E. Hawkes, AIA, Donald W. Richardson, AIA and Werner S. Storch, P.E. The original architectural, structural, civil, mechanical and electrical drawings were provided to MSC Engineers, Inc. by the City of Salem Public Works Department prior to commencement.

The structures on the site consist of three wings of City Hall joined by expansion joints forming a C-shaped footprint around a first level exterior plaza that is currently covered by a concrete and "glass" canopy roof approximately 40 feet above the plaza. The overall footprint of this conglomeration is approximately 200 feet by 230 feet. Further to the north, a council chambers and two-story parking structures exist. All of these elements are connected by pedestrian bridges and walkways. There is a covered parking level beneath the plaza slab which ties into the basement and mechanical sub-basement of the south City Hall wing. The City Hall wings are three stories in height and only the south wing has a full basement. At the east and west wings the first level is a slab on grade with limited basement level tunnels. Floor to floor heights are 9'-6", 13'-0", 15'-0" and 13'-0" for the parking, first, second and third stories, respectively. The council chambers floor slab is at the same elevation as the second floor of the wings, and it is connected to the wings by 12 ft. wide walk-ways. The walk-ways are supported by columns that run all the way up to the plaza roof.

The parking structure is comprised of a roof slab connected to the City Hall first floor by two pedestrian bridges. There are (18) planters 10'-0" wide x 14'-0" length x 3'-6" height. There is a platform at the center of the roof 36'-0" square and 1'-6" height covered by concrete pavers. This platform was designed as a water pool feature according to initial drawings. However, the existing condition shows that the feature has been filled with soil and topped with concrete pavers. Additionally, the roof slab has perimeter bench planters around the north edge of the slab. These planters are approximately 6'-0" wide x 1'-6" height. The main parking structure has one upper level with ramps sloping down and merging into the transition ramp which discharges vehicles at the basement level.

The primary structural system for all of the structures is cast-in-place concrete 2-way waffle slab floors, concrete columns, and concrete walls founded on shallow concrete spread footings. All concrete elements are reinforced with mild steel bars and are architecturally exposed. A large portion of the floor area on all levels falls outside the building envelope and therefore has a non-structural topping slab above waterproofing and insulation. The lateral force resisting system is comprised of a combination of rectangular and L-shaped concrete columns and concrete shear walls. A large majority of the exterior walls have ribbon window glazing, precast concrete cladding or spandrel panels and poured concrete architectural walls.

1.3 Site Key Plan

An overall site key plan is presented in Figure 2 for reference only.

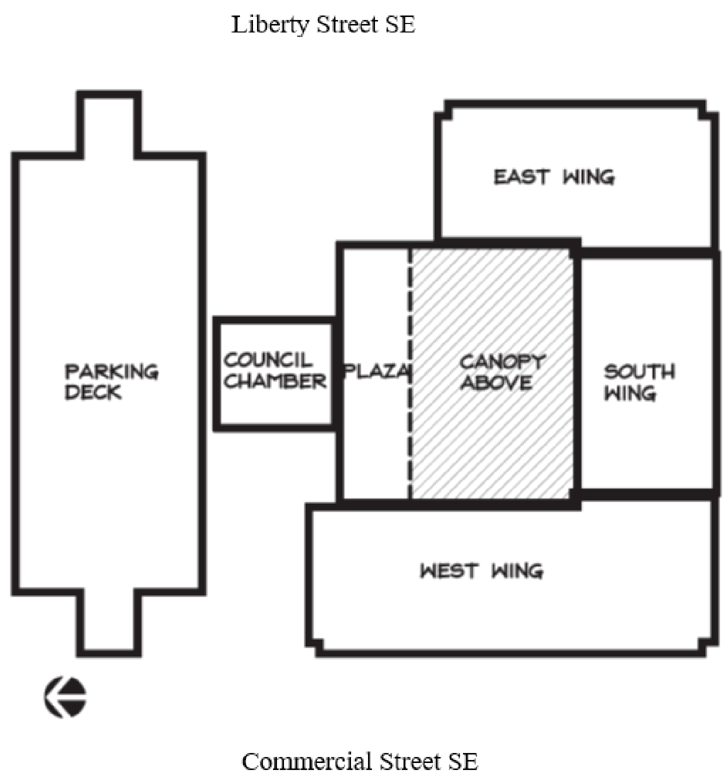

Figure 2 Site Key Plan

1.4 Structural and Non-Structural Deficiencies Findings

MSC Engineers, Inc. has completed a seismic evaluation of the City of Salem City Center in Salem, Oregon. This evaluation is based on the agreement that all critical emergency response functions will be relocated to a new Police Facility. This allows the Risk Category to be reduced from IV to III, and the performance objective reduced from "Immediate Occupancy" to "Life Safety" with and Importance Factor, $I_e = 1.25$. Analysis of the structure considered seismic loads required by the current building code. The use of the term deficiency intends to highlight possible structural instability during an earthquake of significant magnitude, not as a result of gravity loading in the absence of such an event.

As described below, four cases have been modeled. Case 1 represents the as-built condition as shown in the construction drawings (Ref. 2.1), Case 2 shows the as-built configuration with the canopy roof removed. Case 3 illustrates the recommended shearwall upgrades retaining the existing expansion joints. Finally, Case 4 includes the recommended shearwall upgrades including tying the building wings together across the existing expansion joints.

A brief summary of our findings and recommendations are listed below:

1.4.1 Non-Structural Deficiencies

1.4.1.1 Building Wings, Canopy Roof and Council Chambers

- Partition walls constructed of unreinforced or inadequately braced masonry are present in areas around record rooms, corridors, mechanical rooms and stairs that pose out-of plane collapse potential. Approximately 50% of the light gage framed partition walls terminate at the ceiling grid elevation or are not adequately braced to resist out-of-plane lateral forces.
- Suspended ceiling systems in office areas are not adequately braced to resist seismic loads. In addition, lay-in tiles are not properly secured with clips at corridors and egress paths.
- Emergency power equipment is not adequately anchored to prevent overturning or sliding.
- HVAC mechanical equipment such as pumps and ducts may not be adequately anchored or braced to prevent sliding or overturning leading to a loss of function during and after an earthquake.
- Fire suppression piping may not be properly anchored and braced in accordance with current code provisions. In addition, mechanical piping and electrical conduit do not have flexible couplings at expansion joints and other areas where differential movement is expected.
- Exterior window glazing that extends to the floor is not tempered and could pose a hazard.

1.4.1.2 Parking Structure

- Deterioration of concrete due to water intrusion through the top surface. Moderate select instances of concrete cover spalling and scaling of rebar pose falling hazard and eventual degradation of strength. Definite impact on durability and life of structure if unabated.
- Exposed concrete surfaces have not received a seal coat in decades. This renders the rough aggregate vertical and horizontal surfaces vulnerable to undeterred water intrusion and weathering erosion of exposed aggregate.
- Concrete overlay panels approximate 3-in. thickness and supported on shims atop a sand bed on the surface of the structural waffle slab have “walked” over time rendering portions of the overlay slab unsupported. This has resulted in “rocking” of the overlay panels under moving vehicle tire loads. This distress has manifested as cracking and spalling of the overlay panels and will evolve quickly into major damage to these overlay

panels. It does not directly threaten the underlying structural integrity of the waffle slab or severe damage to vehicles but will lead to deterioration of the overlay panels and greater water retention on the surface.

- The planters located at the parking top slab impose a potential for water intrusion and leakage resulting in deterioration of the waffle slab. There is a total of eighteen rectangular planters located around the middle area of the slab. There are also bench planters in the North, East and West sides of the slab.

1.4.2 Structural Deficiencies

- Strength demand to capacity ratios are insufficient for 37% of the existing columns and 69% of the existing shear walls.
- Expected lateral displacements approach 7.59 inches at the west wing roof relative to the base. This deflection exceeds the 2 in. expansion joints and could result in impact of adjacent floor plates leading to localized damage. In addition, the large displacements between floors exceeds the current code requirements for allowable drifts.
- Precast concrete spandrel panels are not anchored to roof slab edges with adequate ductility and strength to prevent spalling or failure at connection points during a design earthquake.
- Six lateral resisting columns at the perimeter of the plaza are insufficiently connected to the floor slabs to provide sufficient seismic strength. These slender columns are also prone to torsional buckling under combined compression and flexural seismic loads.
- Plaza stair towers are not adequately tied to the primary building to prevent displacement that could disrupt the egress path.
- Several architectural concrete walls are not properly isolated from the structure to allow for a minimum 2% story displacement. Three of these walls do not extend to the foundation and will impart high vertical forces on inadequately designed supporting beams and columns.

1.4.3 Recommended Non-Structural Upgrades

These recommended non-structural upgrades are intended to provide an adequate level of safety with regards to preventing falling hazards during a seismic event. Partition walls should be properly attached to the top and bottom to resist out-of-plane loads. Suspended ceilings, HVAC mechanical equipment and fire suppression piping should be braced to resist seismic loads during an earthquake. Emergency power systems should be properly anchored to the supporting structure to prevent overturning and sliding during dynamic loads. Exterior glazing should be tempered to avoid falling hazards over egress paths.

As previously discussed, the parking structure is in need of intervention to prevent further deterioration of the structural waffle slab. The following recommendations are based on our visual inspection of the existing conditions and the findings reported by the URS Corporation in their repair and seismic evaluation report (Ref. 3.2).

- The URS Corporation report (Ref. 3.2) showed that roughly 10,900 sq.ft. of parking surface is in need of intervention. Ideally all concrete overlay panels would be removed from the elevated parking surfaces including ramps and driveways approximately 36,000 sq.ft. This would allow for the complete removal of the sand base and shims. A new liquid applied water proofing barrier would be applied directly atop the structural concrete integrated with the floor drainage system. The sand would be replaced with new clean mason sand and the shims replaced. Presumably, the overlay slab would be poured new, but salvage may be feasible if the contractor deems it cost effective. An alternate scheme, consistent with the URS report, would entail employing the same restoration process but limit the scope of work to the middle drive lane where the damage is the most extensive. This is not an ideal approach, but it is a feasible scope that could be explored if cost reductions are necessary. A final process would involve applying a water barrier membrane on the entirety of the horizontal exposed surfaces and re-sealing the construction joints between overlay panels.
- Flood coat all concrete building surfaces with an impervious coating to provide a protective barrier against moisture and humidity.
- Localize and repair all areas under the waffle slab that show concrete spalling and rebar delamination by wire-cleaning the rebar and applying non-shrink grout to cover the exposed reinforcement. Where necessary inject and fill all cracks or fissures over 1/8" in width.
- The top slab of the parking structure should be waterproofed in a similar way as presented for the parking ramps and driveways. Although the ideal condition is to remove all overlay panels and replace them with a new system, there are other possibilities in case of budgetary constraints. For instance, applying a liquid water barrier membrane on the entirety of the horizontal surfaces and re-sealing the construction joints between overlay panels. This approach should temporary prevent additional water from entering the structural waffle slab; however, without replacing the overlay panels and applying the proper waterproofing, the continuous movement at panel joints may cause cracking of the joint sealant resulting in water intrusion through the joints. Regardless of the approach used to waterproof the top slab surface, it is essential to complete waterproofing of the planter's base and perimeter walls. Waterproofing of the planters is addressed in the URS Corporation report (Ref. 3.2.) and schematic details presented therein. The process begins with removing the existing soil and cleaning the planter to expose the concrete waffle slab, a waterproofing membrane is installed atop the slab and planter side walls ensuring proper drainage at the base. A bed of drainage rock with a soil barrier on top is proposed followed by landscaping work. In addition, all drains should be inspected and properly sealed to prevent leakage and filtration around the pipe inlets.

Upon removal of the roof canopy, the plaza at the first floor level between the building wings and parking structure will be exposed to the full brunt of all weather conditions with more intensity and severity than experienced over its current history. This will entail a complete re-conditioning of the plaza slab including proper slab waterproofing and surface drainage. In order to achieve the waterproofing expectations, it is anticipated that the overlay concrete panels will need to be removed to the extent of exposing the top of the concrete waffle slab. One option to restore the topping surface and provide an adequate drainage is to employ concrete pavers as the future walking surface. Pavers would allow for adequate surface drainage and their relative maintenance cost is less than conventional concrete toppings. This solution requires removing the existing overlay concrete panels and cleaning the concrete waffle slab surface followed by application of a new liquid water barrier, a waterproofing membrane and a composite hydroduct drainage. Finally, a clean layer of sand can be applied for leveling and installation of concrete pavers. It is important to note that adequate drainage is necessary to prevent water stagnation and possible filtration into the concrete waffle slab. As an alternative to the concrete pavers, new overlay concrete panels can be installed following the same process as described for the parking structure ramps and driveways. The amount of rain water that will be captured by the plaza surface must be designed, filtered and treated so it can be disposed. This new volume of water, as stated previously, will be larger than the volume at which the current system is subjected. Therefore, upgrades to the drainage system are necessary to accommodate the new volume of rain water.

1.4.4 Recommended Structural Upgrades

The proposed course of action for owner elective seismic upgrading of the building is presented on schematic plan drawings (Appendix B). The primary recommendation is to relocate essential services such as police and emergency communications into a new separate facility to allow the Risk Category to be reduced from IV to III. The reduction in Risk Category results in a 25% reduction in code required seismic forces while maintaining a conservative Life-Safety performance objective. This reduced seismic force is reflected in all of our attached modeling and analysis. Highlights of the recommended structural upgrades are listed below:

- Addition of reinforced concrete shear walls at the Civic Center, Council Chambers and parking structure.
- New concrete grade beams and footings to support new shear walls and augment the foundation elements under existing columns and shear walls.
- Reinforce precast spandrel panel anchorage at the roof.
- Augment collector reinforcement at stairs and plaza perimeter columns at connection points to the floor slab.
- Laterally brace the plaza perimeter columns with gusset plates at the floor plate level to reduce the unbraced length of the columns from three story to one story.
- Removal of discontinuous shear walls or detachment and re-support of existing concrete non-structural walls to accommodate the expected displacement between building floors.
- Removal of canopy roof and its supporting columns above the second level walkway.

1.5 Conclusions

Our report strives to highlight the seismic vulnerabilities of the City Hall building and parking structure at the City of Salem Civic Center campus. Recommendations were provided based on a comprehensive structural analysis using computer modeling as well as a qualitative assessment from several on-site building walk-throughs, careful review of the original architectural and structural drawings and previous repair and seismic evaluation report provided by URS Corporation. Our evaluation was concentrated on the structural aspects of the building's potential seismic performance, and we realize there will be further input from other disciplines that will influence the assumptions that affect this work. We anticipate the next step in the process will include a study conducted by the City and Architect to assess the feasibility of the proposed upgrades. As stated throughout this report, the elective proposed seismic upgrades part of this evaluation report are based on the consideration that all critical emergency response functions will be relocated to a new Police Facility. This allows the Risk Category to be reduced from IV to III, and the performance objective reduced from “Immediate Occupancy” to “Life Safety” with an Importance Factor, $I_e = 1.25$. This assumption results in an immediate reduction of the seismic load by 25 percent, which in conjunction with the proposed upgrades results in a structure with an adequate performance level as related to current code seismic and serviceability requirements.

MSC Engineers, Inc. has attached the contractor provided cost estimate in Appendix L. This estimate was based on our schematic drawings, an on-site walk-through with a representative of Dalke Construction, and verbal description of the work. A market driven cost estimate is seen as the most accurate way to assign an appropriate dollar figure to the proposed upgrades that can be used in conjunction with future cost estimates provided by the Architect and their agents. This portion of the evaluation should be used with discretion for determining the feasibility of the proposed work. It is important to note that implementation of the proposed recommendations described herein in tandem with intervention from required work from other disciplines will disrupt the daily functioning and operation of the building. We believe that the upgrades can be performed in stages eliminating the need for seeking complete relocation of offices and services operating in the building. Therefore, coordination between all entities involved during the various stages of the project will be essential to reduce the impact of partial or staged occupancy on the continuous operation of the facility.

In addition to the seismic evaluation of the City of Salem Civic Center, MSC Engineers, Inc. was retained to provide peer review to the evaluation report provided by BergerABAM with respect to the City of Salem Central Public Library. This review focuses on the information provided by BergerABAM in the evaluation report (Ref. 3.3) as it relates to the proposed seismic upgrades for structural and non-structural components. A detailed review of calculations and computer model are not included in this report. This review is intended to assist the City's representative in the determination of whether or not the proposed upgrades, recommendations and cost estimate are consistent with standard engineering practice. Upon review of the City of Salem

Central Public Library evaluation report provided by BergerABAM (Ref. 3.3), MSC Engineers, Inc. has determined that the analysis was conducted using adequate code references and seismic parameters. The proposed seismic upgrades for the library, auditorium and parking structure correlate with the need to provide additional stiffness and strengthening to the existing structural systems. We recommend that an additional modal response spectrum analysis be performed to the library and auditorium including current code seismic loading levels. The modal analysis will provide a more representative idealization of the response of the structure. In our opinion, the proposed non-structural recommendations and cost estimate appear to be suitable for the scope of work presented in the report. We recommend that an adjustment factor for inflation be added to the given cost to achieve a more representative value to the current market price.

Section 2. Inputs and References

This section outlines the various inputs and references that were used in the development of this seismic evaluation report.

2.1 Applicable Codes, Standards and References

The following codes, standards and references were used in the development of this report.

1. Codes and Standards
   1.1. 2014 Oregon Structural Specialty Code (OSSC)
   1.2. ASCE/SEI Standard 7-10, "*Minimum Design Loads for Buildings and Other Structures*"
   1.3. ASCE/SEI Standard 31-03, "*Seismic Evaluation of Existing Buildings*"
   1.4. ASCE/SEI Standard 41-13, "*Seismic Evaluation and Retrofit of Existing Buildings*"
   1.5. ACI 308-11, "*Building Code Requirements for Structural Concrete*"
   1.6. FEMA 273, "*NEHRP Guidelines for the Seismic Rehabilitation of Buildings*"
2. Engineering Drawings
   2.1. *Drawing package per Architects A.I.A*, Dated 3/16/1970
3. Engineering Reports
   All existing referenced reports are posted to and available at MSC Engineers FTP Host link: *173.164.104.173*
   3.1. *"City Hall Seismic Evaluation Salem"*, Oregon, MSC Engineers, Dated 11/14/2011
   3.2. *"City of Salem Civic Center – Repair and Seismic Evaluation Reports"*, URS Corporation, Dated 06/30/2005
   3.3. *"Evaluation Report - Salem Central Public Library", BergerABAM, Dated 02/14/2014*

2.2 Inputs

The following are inputs to this report:

2.2.1 Material Properties

Concrete
All existing strength values taken from AIA Drawing CS-1 (Ref. 2.1) unless noted otherwise.

▪ Unit Weight	150 pcf (Assumed)
▪ Existing Structural Concrete Footings	f'c=3,000 psi
▪ Existing Structural Concrete Wall, Slabs, and Beam	f'c=4,000 psi
▪ Existing Precast Concrete	f'c=5,000 psi

Reinforcing Steel

- Existing Reinforcing Steel ASTM A615 — Fy = 60,000 psi
- Existing Ties and Stirrups — Fy = 40,000 psi

Soils

- Safe Bearing Capacity (qa) — 5,000 psf (Ref. 2.1)
- Active Equivalent Fluid Pressure — 40 pcf (Assumed)
- At-Rest Equivalent Fluid Pressure — 60 pcf (Assumed)
- Passive Equivalent Fluid Pressure — 200 pcf (Assumed)
- Friction Coefficient — 0.40 (Assumed)
- Unit Weight — 110 pcf (Assumed)

2.2.2 Loading

Loadings used are noted throughout the calculations; however, a general summary is provided below.

2.2.2.1 Dead Load

Dead loads are presented and applied as shown in Appendix E.

2.2.2.2 Live Load

Live loads are divided and applied as shown in Appendix E.

Floor Live Loads

- Common Areas & Corridors: 100 psf
- Restaurant: 100 psf
- Office: 50 psf
- Classroom: 40 psf
- Mechanical/Storage: 125 psf
- Jail Cell Blocks: 40 psf
- Exterior Yards & Terraces: 100 psf
- Assembly Areas: 60 psf (Council Chambers)

Note: Floor areas are divided into areas of 50 psf and 100 psf for the purpose of our analysis on the 1st, 2nd, and 3rd levels. Jail cell block areas are assumed to be transformed into office space after relocation of police.

2.2.2.3 Wind Load

Wind loading was not considered for the purposes of this analysis as it was deemed irrelevant compared to seismic loading.

2.2.2.4 Seismic Load

Seismic load is developed using current USGS seismic ground motion values. See Appendix A for specific seismic data.

S_s:	0.928g	S_{M1}:	0.683g
S_1:	0.437g	S_{DS}:	0.698g
S_{MS}:	1.047g	S_{D1}:	0.456g

Importance Factor (I_e):	1.25
Site Class:	D
Seismic Design Category:	D
Lateral Force Resisting System:	Ordinary Reinforced Shearwalls
Response Modification Factor (R):	5.0
Deflection Amplification Factor (C_d):	4.5
Seismic Overstrength Factor (Ω_o):	2.5
Analysis Procedure:	Linear Response Spectrum Analysis

2.2.2.5 Load Combinations

The following load combinations are considered for strength design per 2014 OSSC (Ref. 1.1) and ASCE 7-10 (Ref. 1.2):

LRFDEQ161: 1.4D	Where:
LRFDEQ162: 1.2D + 1.6L	D: Dead Load
LRFDEQ165: 1.2D + 0.5L + 1.0E	L: Live Load
LRFDEQ167: 0.9D + 1.0E	E: Seismic Load

2.3 Assumptions

The following are assumptions to this seismic evaluation report:

- Essential services such as police and emergency communications will be relocated to a new separate facility to allow the Risk Category to be reduced from IV to III. The reduction in occupancy category results in a 25% reduction in code required seismic forces while maintaining a conservative Life-Safety performance objective.
- Existing material properties comply with construction documents per reference 2.1.
- Canopy roof would be removed and other recommendations will be implemented to accomplish a structure as presented in Case 4 (Section 4.1).

Section 3. Acceptance Criteria

The resultant demand values in each structural system will be compared with their associate code based allowable capacities. Elements of the structure corresponding will be considered acceptable if the demand values for each member is less than their corresponding capacity values (i.e., D/C ratio of less than 1.0). In cases, where the D/C ratio of a given member or component exceeds 1.0, a recommendation or justification is provided to ensure compliance with the acceptance criteria.

The drift and deformation of the structure should comply with 2014 OSCC (Ref. 1.1) allowable drifts. The calculated drift should be determined at critical locations of the structure with considerations of translational and torsional displacements as described in the ASCE7-10 (Ref. 1.2).

Section 4. Methodology

4.1 Evaluation Approach

The 2014 Oregon Structural Specialty Code (Ref. 1.1), and to a limited extent of the ASCE/SEI 31-03 Seismic Evaluation (Ref. 1.4) were used to evaluate the seismic resistance of the existing structure with and without seismic upgrades. A 3D finite element computer model ETABS® was used to analyze the entire structure and design check of individual components for strength and serviceability requirements. Design capacities were calculated using the integrated design tool incorporated in ETABS® per the ACI 318-11 (Ref. 1.5). It should be noted that this building was not originally designed according to the current building codes that tend to be more rigorous than legacy codes. Therefore, full compliance with the current code is not the objective of this evaluation. Current code requirements are only meant, for this case of study, as a benchmark for assessing the condition of the as-built structure, and evaluate the benefits of the proposed elective structural seismic upgrades as well as collapse prevention and life-safety objectives. Four cases were evaluated during this analysis to compare the overall performance of the structure with the proposed elective upgrades. These cases are described below and listed in Table 1:

- **Case 1** represents the as-built condition as documented in the drawings (Ref. 2.1)
- **Case 2** represents the as-built condition with the canopy roof removed. The supporting columns at the North side are removed above the walk-ways.
- **Case 3** represents the proposed elective upgrades for shear resisting elements without tying the building wings at the expansion joints.
- **Case 4** represents the proposed elective upgrades of Case 3 and includes tying the building wings across the expansion joints.

Table 1. Analysis Case Matrix

Analysis Case	Structure Self-Weight	Load Combos	Redundancy Factor (ρ)	Analysis Goal
Case 1 (As-built)	Existing	1.2D+1.0E+0.5L 0.9D+1.0E	1.3	Check number of overstressed LRE and adequacy of existing columns
Case 2	Existing with canopy roof removed	1.2D+1.0E+0.5L 0.9D+1.0E	1.3	Check improvement of LRE with removal of mall canopy roof
Case 3	Existing + new walls	1.2D+1.0E+0.5L 0.9D+1.0E	1.3	Check improvement of LRE with additional new shearwalls
Case 4	Existing + new walls with tied wings	1.2D+1.0E+0.5L 0.9D+1.0E	1.0	Check improvement of LRE with additional new shearwalls and tied wings

4.2 Performance Objective Definition

Rehabilitation structural performance levels are defined in the ASCE 41-13 (Ref. 1.4) and FEMA 273 (Ref. 1.6). They are explained below in increasing order of performance, and difficulty.

Collapse Prevention: means the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force-resisting system, large permanent lateral deformation of the structure, and to a more limited extent, degradation in vertical-load carrying capacity. However, all significant components of the gravity load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. Under this performance objective, the structure is assumed to be impaired beyond salvage and is not safe for re-occupancy, as aftershock activity could induce collapse.

Life Safety: means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this would not result in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons, this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

Immediate Occupancy: means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical-, and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

4.3 Dynamic Analysis Procedure

A linear response spectrum analysis (RSA) procedure was used along with a linear static equivalent lateral force (ELF) procedure. The design response spectrum for the dynamic analysis was scaled to 85% of the base shear given by the use of an ELF procedure with a predetermined response factor (R) and using mapped ground accelerations from the design basis earthquake with a 500-year return period. The dynamic analysis was performed using Ritz Vectors and all modal responses were combined with a complete quadratic combination (CQC) method. A minimum of 90% modal mass participation was required.

Note: an additional 0.75 reduction of seismic forces for existing buildings as explained in ASCE 31-03 (Ref. 1.3), 5.2.1 was NOT used in this analysis. The reason for this is the purpose of the analysis was only to show “betterment” of LRE performance by making elective seismic upgrades. Therefore, the unmodified seismic loading required by the current building code for new buildings was used as the basis for this evaluation.

4.4 Warning on Inelastic Capacity

The procedure previously described measures the demand to capacity (D/C) ratios of individual components according to the OSSC (Ref. 1.1) for new buildings. A lateral resisting system response factor (R) of 5.0 was selected for the existing and new concrete shear walls. Ordinary reinforced shear walls according to the current code do not require special seismic detailing for inelastic capacity which is generally how this building was originally constructed. However, current codes prohibit this type of structural system for new construction in regions of high seismic activity such as Salem, Oregon.

The D/C ratios in the report are for the design basis earthquake (DBE) using this inelastic response factor. It should be cautioned that existing shear walls not originally detailed for inelastic capacity that have D/C ratios less than 1.0 are not guaranteed to perform without brittle failure during a large earthquake. This can be made clear by explaining that if this building were built with today’s codes “Special Reinforced Shear Walls” would be required. These walls contain extensive prescriptive detailing to maintain inelastic reserve capacity in the event of a large earthquake. The existing walls do not provide this level of durability and ductility. To maintain current code required inelastic capacity, all new walls should be designed as “Special Reinforced Shear Walls.” If possible, critical original shear walls should be reinforced to comply with special wall requirements. If this is not feasible, then new additional concrete walls should be introduced near the original walls to create a redundant ductile element to laterally support that portion of the floor diaphragm. This is up to the discretion of the responsible engineer and owner.

The other option to fully evaluate inelastic capacity is a rigorous inelastic model executed per ASCE 41-13 (Ref. 1.4) and FEMA 273 (Ref. 1.6) Performance Based Seismic Design (PBSD) methodology. This is an option if the building is ever planned to undergo a comprehensive seismic retrofit and it will provide further information on whether original inelastic elements are able to resist the maximum considered earthquake (MCE). This approach is not warranted for the current scope of seismic upgrades.

4.5 Design Parameters

4.5.1 Irregularities

The following vertical and horizontal structural irregularities were noted that required the use of a modal analysis procedure. All references are from the ASCE 7-10 (Ref. 1.2):

Horizontal Structural Irregularities (Table 12.3-1)
Torsional Irregularity (1a)
Extreme Torsional Irregularity (1b)*
Reentrant Corner Irregularity (2)

Vertical Structural Irregularities (Table 12.3-2)
In-plane Discontinuity (4)* – various shear wall lines, in-plane shift

*This irregularity will be resolved after implementation of proposed elective upgrades part of analysis Case 4 and presented in Appendix B.

4.5.2 Lateral Resisting Elements

The structure was broken up into two classifications of lateral components: primary lateral resisting elements (PLRE) and secondary lateral resisting elements (SLRE). The reinforced concrete walls were designated as PLRE and serve to resist the majority (>75%) of seismic generated lateral forces while the reinforced concrete columns and beams are designated as SLRE and resist less than 25% of these loads. The SLREs are not required for the building to reach desired performance, but they will attract a relatively small amount of lateral load through their inherent stiffness.

4.5.3 Redundancy Factor

The redundancy factor (ρ) is set equal to 1.3 for the original building because of the extreme torsional irregularity. For the upgraded building, Case 4, ρ is set equal to 1.0. ASCE 31-03 recommends that each horizontal diaphragm have greater than 2 lines of shear walls in each direction for the Life Safety performance objective. After tying the expansion joints together and adding the new shear resisting lines, the extreme torsional irregularity will be reduced to a torsional irregularity and the redundancy factor decreased to 1.0. This will reduce the seismic loads by 30% and further decrease D/C ratios in Case 4. Therefore, the redundancy factor is set to 1.3 for cases 1 through 3, and 1.0 for case 4.

4.5.4 Importance Factor

This evaluation is based on the assumption that all critical emergency response functions will be relocated to a new Police Facility. This allows the Risk Category to be reduced from IV to III, and the performance objective reduced from “Immediate Occupancy” to “Life Safety” with and Importance Factor, Ie = 1.25.

4.6 Structural Model

4.6.1 P-delta Analysis

An elastic non-iterative P-delta method was used to account for geometric non-linearity. The results of the P-delta analysis on building response were assessed to be minimal, but they were included. Additionally, the code only requires the second order analysis for slender columns which will not exist in this building after implementation of the upgrades.

4.6.2 Stiffness Modifiers

Stiffness modifiers were utilized to account for beam, column, wall and slab cracking and openings. Detailed descriptions of assigned modifiers are found within the ETABS output in the Appendix.

4.6.3 Foundation Boundary Conditions

All column boundary conditions at the foundation were assigned as fully fixed. This assumption was made given the approximate 2.5 ft. depth of the bottom of footing below the slab on grade as well as the average 2 ft. depth of the footings. The footings and slab were presumed to generate sufficient resistance to address applied moments at the base of the footing. All shearwall boundary conditions were assigned as pinned.

4.6.4 Diaphragms

All diaphragms were modeled with 2D shell elements to represent approximate waffle slab in-plane and out-of-plane rigidity instead of attempting to model the actual two-way waffle slab. Modeling the waffle slab was considered convoluted and unnecessary since in-plane rigidity is the preeminent structural property pertinent for lateral analysis. Therefore, gravity load carrying capacity of the structural floor slabs cannot be assessed using this lateral model. The strength of the two-way waffle slab is evaluated following guidelines presented in the ACI 318-11 (Ref. 1.5).

4.7 Site Specific Seismic Evaluation

There is an option to perform a risk-targeted site specific seismic evaluation for quantifying MCE_R. These are performed by qualified professional geotechnical engineers and can result in seismic forces less than the mapped spectral values provided by the code. The evaluation accounts for tectonic setting, geology, and seismicity for a given site. This approach should be considered prior to any large comprehensive upgrade project.

4.8 Further Recommendations

In addition to the rigorous analysis described above, a cursory examination of non-structural components such as precast panels, cladding, stairs towers and partitions were made and documented in the URS report (Ref. 3.2) and MSC Engineering report (Ref. 3.1). Items that appeared to be inadequately connected to the primary structure absent sufficient strength and ductility to resist a design earthquake were recommended for examination in further detail by a local engineer. Deficient components such as ceiling systems, HVAC equipment and fire suppression systems were listed for further consideration by the building owner. Specific recommendations to remedy possible deficiencies related to these non-structural components should be assessed in the future under a discipline specific seismic evaluation.

Section 5. Results and Conclusions

5.1 Findings

This section highlights the structural deficiencies of the existing as-built structure when subjected to current seismic lateral design provisions. These results are contrasted with the results derived for incremental improvements culminating with the results for the final recommended improvements. Demand to capacity ratios (D/C) of existing columns and shearwalls, as well as expected displacements, were examined by considering seismic forces for analysis cases 1-4.

Strength Capacity of Lateral Resisting Elements and Expected Lateral Displacements of Floor Plates

5.1.1 Case 1: As-Built Existing Condition

Table 2 summarizes strength D/C ratio for existing lateral resisting elements (LRE) - concrete shearwall piers and columns. Deficient elements were divided up into three categories of D/C ratio to depict the degree a critical element is overloaded. A total of 580 existing shearwall piers and 248 existing columns were considered in our analysis. As represented, 14% of shearwalls and 42% of columns exhibit D/C ratios in excess of building code requirements for Case 1. However, a significant number of the elements evaluated had D/C ratios greater than 125% of their design capacities which is deemed a nominal degree of overstress.

For a detailed list of D/C ratios see Appendix F.

Table 2 Summary of Deficient LRE **(Case 1)**

Level	Shearwall Piers			Columns			
	1.0<D/C<1.25	1.25<D/C<1.5	D/C>1.5	1.0<D/C<1.25	1.25<D/C<1.5	1.5<D/C<2	D/C>2
Basement	9	5	4	0	0	2	0
1st Level	5	5	10	2	2	4	3
2nd Level	7	6	16	12	4	13	8
3rd Level	2	1	8	8	12	4	4
Roof	3	0	1	15	7	5	0
Total	**26**	**17**	**39**	**37**	**25**	**28**	**15**

Expected Lateral Displacements of Original Floor Plates

The computer model provided an approximate magnitude of horizontal lateral displacement to be expected at the building floor plates during a design seismic event. Elastic values for cracked concrete lateral resisting elements were calculated directly by the program by the use of stiffness modifiers. A prescribed deflection amplification factor (C_d) was then applied to approximate the inelastic lateral displacement after the concrete elements have cracked and the reinforcement has undergone significant yielding. Since a torsional irregularity exists, the displacements were taken at the corners of the diaphragms as shown in Figure 4 - Figure 7. The existing as-built structure has been determined to have extreme torsional irregularities, and therefore, a torsional amplification factor (A_x) is applicable and should be applied to the computed story displacements to obtained the maximum expected drifts. However, for comparison purposes, the amplification factor has NOT been included in the lateral displacements presented in Table 3. The maximum inelastic drift at the canopy roof (joint 456 - Figure 7) is calculated be 10-in.

Figure 3 presents the deformed shape of the structure under seismic loading after combining the displacements using the Square Root of the Sum of the Squares (SRSS) directional combination procedure.

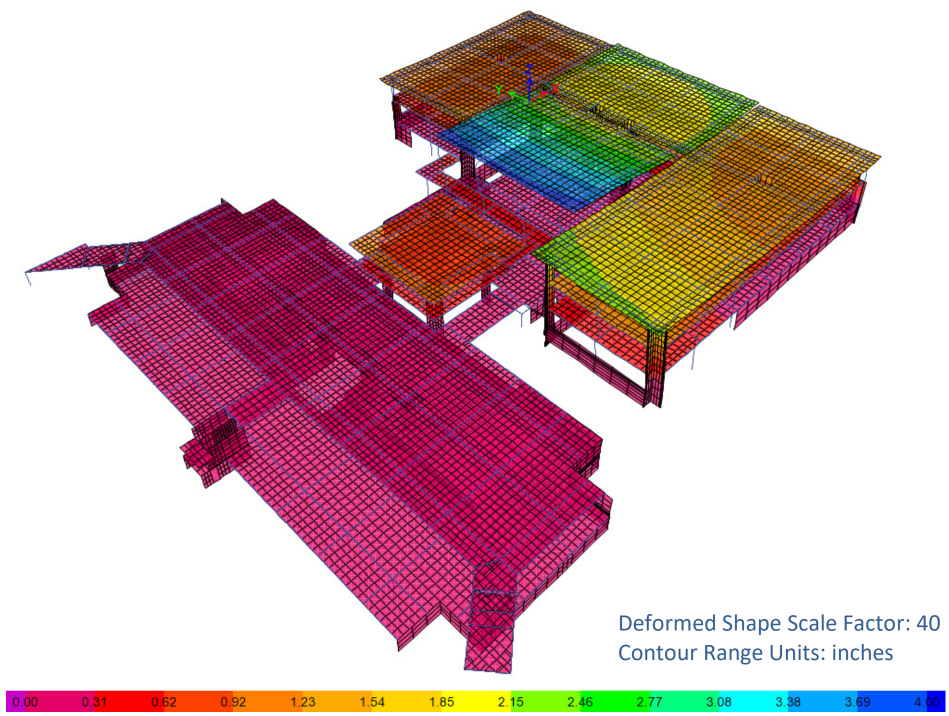

Figure 3 SRSS Directional Combination Displaced Shape (Case 1)

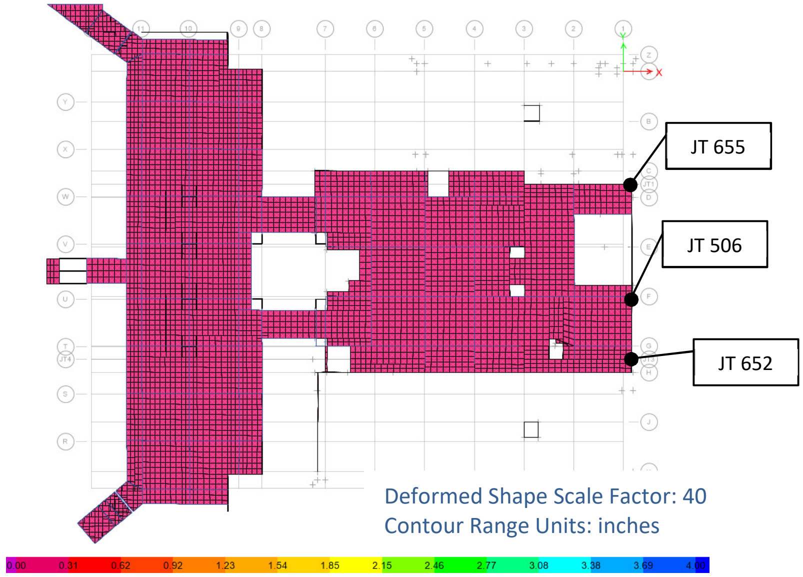

Figure 4 Level 1 Diaphragm Displacement

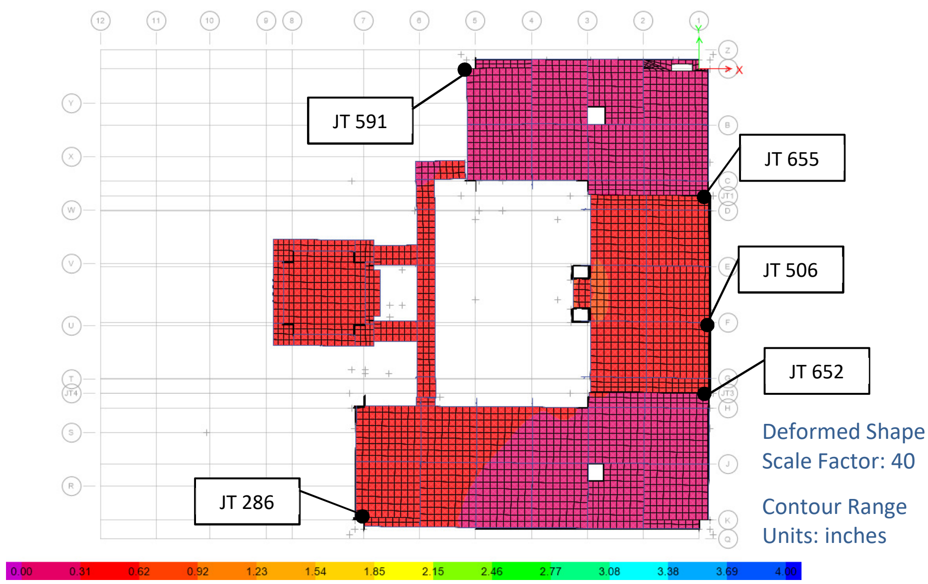

Figure 5 Level 2 Diaphragm Displacement

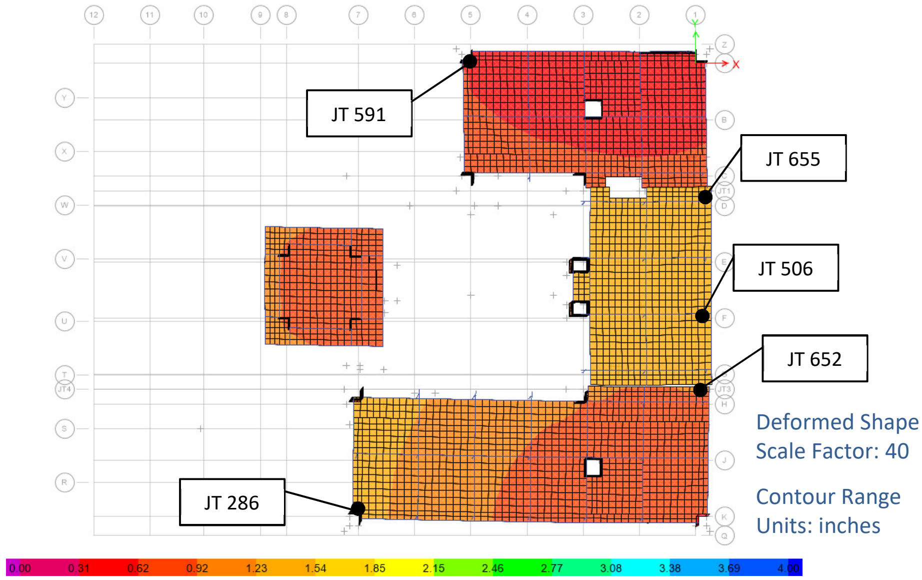

Figure 6 Level 3 Diaphragm Displacement

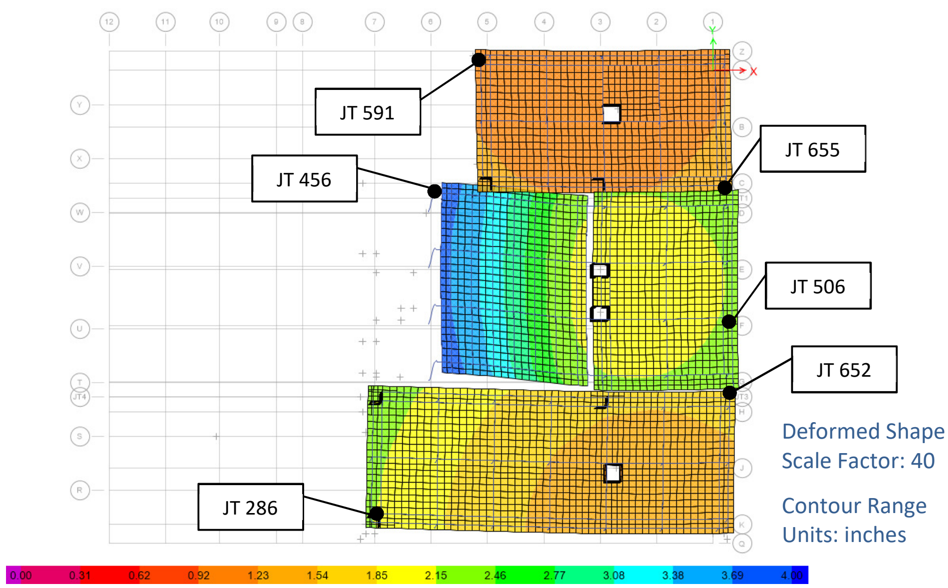

Figure 7 Roof Level Diaphragm Displacement

Table 3 Summary of Maximum Lateral Displacements **(Case 1)**

Location		Level	Floor Height (ft.)	Relative to Level Below				Relative to Base			
				Magnitude (in) Elastic (Inelastic)		h/d ratio Elastic (Inelastic)		Magnitude (in) Elastic (Inelastic)		h/d ratio Elastic (Inelastic)	
EAST WINGS	JOINT 591	2nd Level	13.0	0.10	(0.37)	1527	(424)	0.10	(0.37)	1527	(424)
		3rd Level	13.0	0.49	(1.77)	318	(88)	0.59	(2.14)	263	(73)
		Roof	13.0	0.66	(2.38)	236	(66)	1.15	(4.15)	135	(38)
	JOINT 655	2nd Level	13.0	0.13	(0.46)	1221	(339)	0.13	(0.46)	1221	(339)
		3rd Level	13.0	0.56	(2.01)	279	(78)	0.69	(2.47)	227	(63)
		Roof	13.0	0.85	(3.04)	185	(51)	1.40	(5.06)	111	(31)
SOUTH WINGS	JOINT 506	1st Level	9.5	0.13	(0.49)	845	(235)	0.13	(0.49)	845	(235)
		2nd Level	13.0	0.32	(1.14)	493	(137)	0.45	(1.63)	345	(96)
		3rd Level	13.0	0.83	(2.99)	188	(52)	1.15	(4.13)	136	(38)
		Roof	13.0	0.95	(3.42)	164	(46)	1.78	(6.41)	88	(24)
WEST WINGS	JOINT 286	2nd Level	13.0	0.48	(1.72)	327	(91)	0.48	(1.72)	327	(91)
		3rd Level	13.0	0.82	(2.96)	190	(53)	1.30	(4.68)	120	(33)
		Roof	13.0	1.29	(4.63)	121	(34)	2.11	(7.59)	74	(21)
	JOINT 652	1st Level	9.5	0.06	(0.23)	1816	(504)	0.06	(0.23)	1816	(504)
		2nd Level	13.0	0.15	(0.56)	1008	(280)	0.22	(0.78)	717	(199)
		3rd Level	13.0	0.76	(2.74)	205	(57)	0.92	(3.30)	170	(47)
		Roof	13.0	0.91	(3.28)	171	(48)	1.67	(6.02)	93	(26)
MALL CANOPY ROOF	JOINT 456	1st Level	9.5	0.06	(0.23)	1770	(492)	0.06	(0.23)	1770	(492)
		2nd Level	13.0	0.42	(1.52)	370	(103)	0.49	(1.75)	321	(89)
		Roof	26.0	2.78	(10.0)	112	(31)	3.20	(11.5)	98	(27)
Note: Elastic and inelastic displacements are not cumulative.											

5.1.2 Case 2: As-Built Condition with Canopy Roof Removed

Table 4 summarizes strength D/C ratio for existing lateral resisting elements (LRE), concrete shearwall piers and columns after removing the canopy roof. A total of 580 existing shearwall piers and 244 existing columns were considered in our analysis. As represented, 14% of shearwalls and 33% of columns exhibit D/C ratios in excess of building code requirements for Case 2. However, a significant number of the elements evaluated had D/C ratios between 100% and 125% of their design capacities which is deemed a nominal degree of overstress.

The removal of the canopy roof reduces the effective seismic weight and stress level in other lateral force resisting elements. However, the amount of deformation that is computed at the expansion joints between wings could lead to localized failure at the floor plates. Table 5 presents a summary of maximum displacements at different locations. The maximum inelastic drift at the roof is 4.71-in. (Table 5 - Joint 286), which exceeds the 2-in. gap provided at expansion joints. As a result, the building wings and bridges will pound one another inducing localized damage as a consequence of inelastic deformations.

Table 4 Summary of Deficient LRE **(Case 2)**

Level	Shearwall Piers			Columns			
	1.0<D/C<1.25	1.25<D/C<1.5	D/C>1.5	1.0<D/C<1.25	1.25<D/C<1.5	D/C>1.5	D/C>2
Basement	10	6	3	0	2	0	0
1st Level	6	3	10	2	4	1	2
2nd Level	8	5	16	7	2	7	7
3rd Level	5	1	8	11	9	3	4
Roof	3	0	1	16	3	1	0
Total	**32**	**15**	**38**	**36**	**20**	**12**	**13**

Table 5 Summary of Maximum Lateral Displacements **(Case 2)**

Location		Level	Floor Height (ft.)	Relative to Level Below: Magnitude (in) Elastic	(Inelastic)	Relative to Level Below: h/d ratio Elastic	(Inelastic)	Relative to Base: Magnitude (in) Elastic	(Inelastic)	Relative to Base: h/d ratio Elastic	(Inelastic)
EAST WINGS	JOINT 591	2nd Level	13.0	0.10	(0.37)	1513	(420)	0.10	(0.37)	1513	(420)
		3rd Level	13.0	0.50	(1.78)	315	(88)	0.60	(2.15)	261	(72)
		Roof	13.0	0.66	(2.39)	235	(65)	1.16	(4.18)	134	(37)
	JOINT 655	2nd Level	13.0	0.13	(0.46)	1211	(336)	0.13	(0.46)	1211	(336)
		3rd Level	13.0	0.56	(2.01)	279	(77)	0.69	(2.48)	227	(63)
		Roof	13.0	0.85	(3.05)	184	(51)	1.41	(5.06)	111	(31)
SOUTH WINGS	JOINT 506	1st Level	9.5	0.12	(0.45)	916	(254)	0.12	(0.45)	916	(254)
		2nd Level	13.0	0.31	(1.10)	510	(142)	0.43	(1.55)	363	(101)
		3rd Level	13.0	0.94	(3.38)	166	(46)	1.25	(4.48)	125	(35)
		Roof	13.0	1.08	(3.88)	145	(40)	2.02	(7.27)	77	(21)
WEST WINGS	JOINT 286	2nd Level	13.0	0.49	(1.75)	321	(89)	0.49	(1.75)	321	(89)
		3rd Level	13.0	0.84	(3.01)	186	(52)	1.32	(4.76)	118	(33)
		Roof	13.0	1.31	(4.71)	119	(33)	2.14	(7.72)	73	(20)
	JOINT 652	1st Level	9.5	0.06	(0.20)	2061	(572)	0.06	(0.20)	2061	(572)
		2nd Level	13.0	0.17	(0.60)	938	(261)	0.22	(0.80)	704	(196)
		3rd Level	13.0	0.76	(2.73)	206	(57)	0.93	(3.33)	169	(47)
		Roof	13.0	0.93	(3.34)	168	(47)	1.69	(6.07)	92	(26)

Note: Elastic and inelastic displacements are not cumulative.

5.1.3 Case 3: Proposed Elective Upgrades without Tying the Building Wings at the Expansion Joints

The ETABS® model was used to examine the effect of adding new concrete shearwalls in the parking structure, council chamber and City Hall. These changes are shown in Figure 10 and Figure 9. These upgrades are presented in more detail in Appendix B.

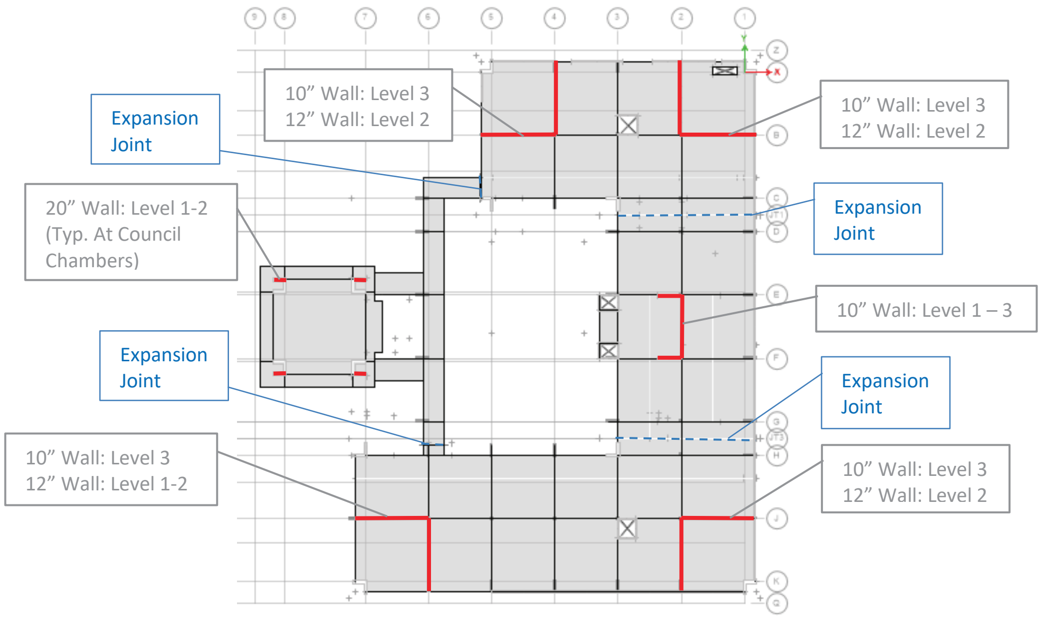

Figure 8 Upgraded Shearwall Locations - City Hall (Level 1 - 3)

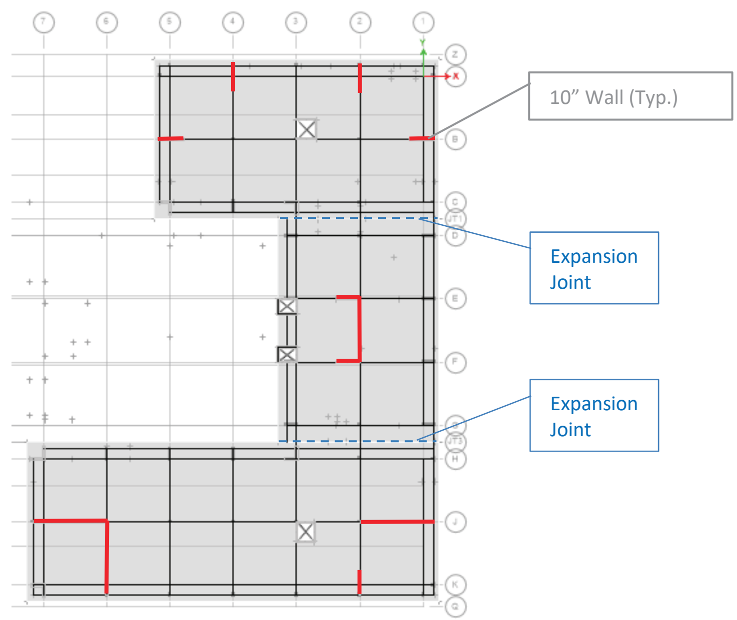

Figure 9 Upgraded Shearwall Locations - City Hall (Roof Level)

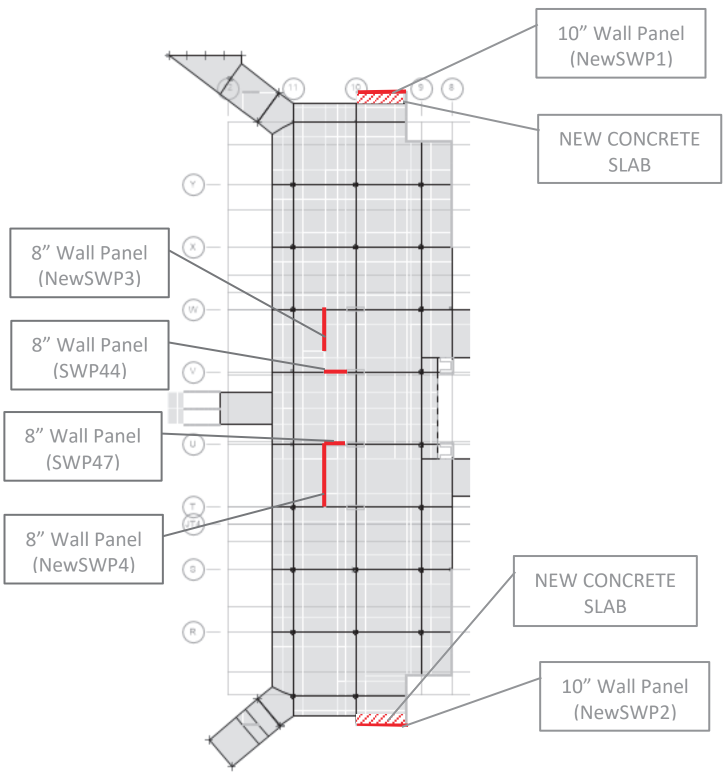

Figure 10 Upgraded Shearwall Locations – Parking Structure

Table 6 summarizes strength D/C ratios for the lateral resisting elements (LRE), concrete shearwall piers and columns after adding the proposed upgrades without tying the building at the expansion joints. A total of 730 shearwall piers and 244 columns were considered in our analysis. The total deficient elements decrease from 166 for Case 2 to 46 for Case 3, where 5% of shearwalls and 4% of columns exhibit D/C ratios in excess of 1.0. This is a noticeable improvement for adding the shearwalls at the various locations. The proposed upgrades significantly reduce the stress levels within the existing shearwalls and columns.

Table 6 Summary of Deficient LRE **(Case 3)**

Level	Shearwall Piers			Columns		
	1.0<D/C<1.25	1.25<D/C<1.5	D/C>1.5	1.0<D/C<1.25	1.25<D/C<1.5	D/C>1.5
Basement	1	2	2	0	0	0
1st Level	3	0	2	3	0	0
2nd Level	6	3	2	6	1	0
3rd Level	6	3	2	0	0	0
Roof	2	2	0	0	0	0
Total	18	10	8	9	1	0

However, the drift projected at each level of each wing are too large to prevent pounding at the original 2-in. wide expansion joints between each of the wings and at each end of the walkway bridges. Expected lateral displacements decrease with the proposed shearwall upgrades. Table 7 shows a summary of the maximum lateral displacement taken for the same slab joints. For example, the magnitude of the inelastic displacement at the roof level (joint 286 - west wing) is reduced 23%. Furthermore, results show that pounding between wings will most likely occur at the 2-in. expansion joints.

Additional shear walls or stouter proposed new walls is one possibility to resolve this problem, but the cost and feasibility of implementing more walls could create collateral restrictions such as space planning. It should be noted that impact damage is not likely to cause failure of the structural members and will most-likely result in damage such as localized spalling and crushing of concrete edges at the joints. Concrete cladding, glazing and partition walls could also be damaged as a result of large deformations, potentially posing a falling hazard to occupants.

Therefore, one more case of study, Case 4, was devised which employs tying the building wings together at the expansion joints. Tying the wings will reduce the torsional behavior of the structure reducing the redundancy factor from 1.3 to 1.0. See section 4.5.3 for discussion about torsional irregularity and redundancy factor.

Table 7 Summary of Maximum Lateral Displacement **(Case 3)**

Location		Level	Floor Height (ft.)	Relative to Level Below				Relative to Base			
				Magnitude (in)		h/d ratio		Magnitude (in)		h/d ratio	
				Elastic (Inelastic)		Elastic (Inelastic)		Elastic (Inelastic)		Elastic (Inelastic)	
EAST WINGS	JOINT 591	2nd Level	13.0	0.03	(0.10)	5681	(1578)	0.03	(0.10)	5681	(1578)
		3rd Level	13.0	0.14	(0.51)	1101	(306)	0.17	(0.61)	922	(256)
		Roof	13.0	0.28	(0.99)	566	(157)	0.42	(1.50)	374	(104)
	JOINT 655	2nd Level	13.0	0.04	(0.13)	4307	(1196)	0.04	(0.13)	4307	(1196)
		3rd Level	13.0	0.14	(0.51)	1093	(304)	0.18	(0.64)	872	(242)
		Roof	13.0	0.27	(0.96)	585	(163)	0.41	(1.47)	381	(106)
SOUTH WINGS	JOINT 506	1st Level	9.5	0.09	(0.32)	1289	(358)	0.09	(0.32)	1289	(358)
		2nd Level	13.0	0.20	(0.71)	795	(221)	0.28	(1.03)	548	(152)
		3rd Level	13.0	0.48	(1.72)	326	(90)	0.68	(2.43)	231	(64)
		Roof	13.0	0.54	(1.96)	287	(80)	1.02	(3.68)	153	(42)
WEST WINGS	JOINT 286	2nd Level	13.0	0.14	(0.49)	1139	(316)	0.14	(0.49)	1139	(316)
		3rd Level	13.0	0.20	(0.71)	795	(221)	0.33	(1.20)	468	(130)
		Roof	13.0	0.30	(1.09)	513	(143)	0.50	(1.80)	312	(87)
	JOINT 652	1st Level	9.5	0.04	(0.14)	2836	(788)	0.04	(0.14)	2836	(788)
		2nd Level	13.0	0.02	(0.09)	6296	(1749)	0.06	(0.23)	2401	(667)
		3rd Level	13.0	0.22	(0.79)	711	(198)	0.24	(0.88)	639	(178)
		Roof	13.0	0.29	(1.05)	536	(149)	0.51	(1.84)	306	(85)
Note: Elastic and inelastic displacements are not cumulative.											

5.1.4 Case 4: Proposed Elective Upgrades of Case 3 and Tying the Building Across the Expansion Joints

Expected lateral displacements decrease with the shearwall additions made in Case 3. It has been discussed that another method of limiting relative displacement of the floor plates and therefore limit the impact damage at the existing expansion joints, involves structurally tying the floor plates together thus abandoning the existing expansion joints. We propose to connect the floor plates across the entire length of the expansion joints by installing epoxy set reinforcing bars drilled through the existing edge beams to stitch the wings together at the joints.

The benefit of unitizing the three wings would presumably limit localized damage at the joints, especially at the critical egress paths. It is anticipated that forces due to volume change will be reasonable since nearly all elastic shrinkage and plastic creep of the structure has taken place. Another benefit to this proposal could be realized in omitting the future required flexible conduit, piping and mechanical connections if the expansion joints were to remain.

Table 8 shows the benefits of tying the wings. Two shearwall piers have a D/C ratio less than 1.25, and two columns have D/C ratio less than 1.5. Because of the redundancy and safety factors incorporated within the analysis, these structural elements are deemed acceptable and meeting the acceptance criteria per Section 3.

Table 8 Summary of Deficient LRE **(Case 4)**

Level	Shearwall Piers			Columns		
	1.0<D/C<1.25	1.25<D/C<1.5	D/C>1.5	1.0<D/C<1.25	1.25<D/C<1.5	D/C>1.5
Basement	0	0	0	0	0	0
1st Level	0	0	0	0	0	0
2nd Level	2	0	0	1	1	0
3rd Level	0	0	0	0	0	0
Roof	0	0	0	0	0	0
Total	**2**	**0**	**0**	**1**	**1**	**0**

Table 9 shows that the magnitude of inelastic displacement at the roof level in the East wing decreased by 27% and by 41% at the South wing. Tying the wings together result in a substantial reduction in the level of stress at shearwall piers and columns. Also, reducing the deformation eliminates the possibility of impact and the falling of hazards such as spalling concrete between floor plate interfaces. Figure 11 illustrates the deformed shape of the structures under seismic loading after combining the displacements using the Square Root of the Sum of the Squares (SRSS) directional combination procedure.

Table 9 Summary of Maximum Lateral Displacement **(Case 4)**

Location		Level	Floor Height (ft.)	Relative to Level Below				Relative to Base			
				Magnitude (in)		h/d ratio		Magnitude (in)		h/d ratio	
				Elastic (Inelastic)		Elastic (Inelastic)		Elastic (Inelastic)		Elastic (Inelastic)	
EAST WINGS	JOINT 591	2nd Level	13.0	0.03	(0.10)	5608	(1558)	0.03	(0.10)	5608	(1558)
		3rd Level	13.0	0.13	(0.47)	1185	(329)	0.16	(0.57)	979	(272)
		Roof	13.0	0.20	(0.72)	782	(217)	0.33	(1.19)	471	(131)
	JOINT 655	2nd Level	13.0	0.05	(0.19)	2946	(818)	0.05	(0.19)	2946	(818)
		3rd Level	13.0	0.14	(0.50)	1125	(312)	0.19	(0.69)	814	(226)
		Roof	13.0	0.21	(0.75)	747	(208)	0.35	(1.25)	449	(125)
SOUTH WINGS	JOINT 506	1st Level	9.5	0.04	(0.13)	3057	(849)	0.04	(0.13)	3057	(849)
		2nd Level	13.0	0.08	(0.28)	1997	(555)	0.12	(0.42)	1351	(375)
		3rd Level	13.0	0.20	(0.72)	784	(218)	0.28	(1.00)	563	(156)
		Roof	13.0	0.23	(0.82)	684	(190)	0.43	(1.54)	365	(101)
WEST WINGS	JOINT 286	2nd Level	13.0	0.13	(0.47)	1194	(332)	0.13	(0.47)	1194	(332)
		3rd Level	13.0	0.18	(0.64)	883	(245)	0.31	(1.11)	508	(141)
		Roof	13.0	0.28	(1.00)	564	(157)	0.45	(1.63)	344	(96)
	JOINT 652	1st Level	9.5	0.02	(0.06)	7070	(1964)	0.02	(0.06)	7070	(1964)
		2nd Level	13.0	0.06	(0.20)	2787	(774)	0.07	(0.26)	2164	(601)
		3rd Level	13.0	0.17	(0.62)	912	(253)	0.23	(0.82)	687	(191)
		Roof	13.0	0.20	(0.72)	783	(217)	0.37	(1.33)	421	(117)

Note: Elastic and inelastic displacements are not cumulative.

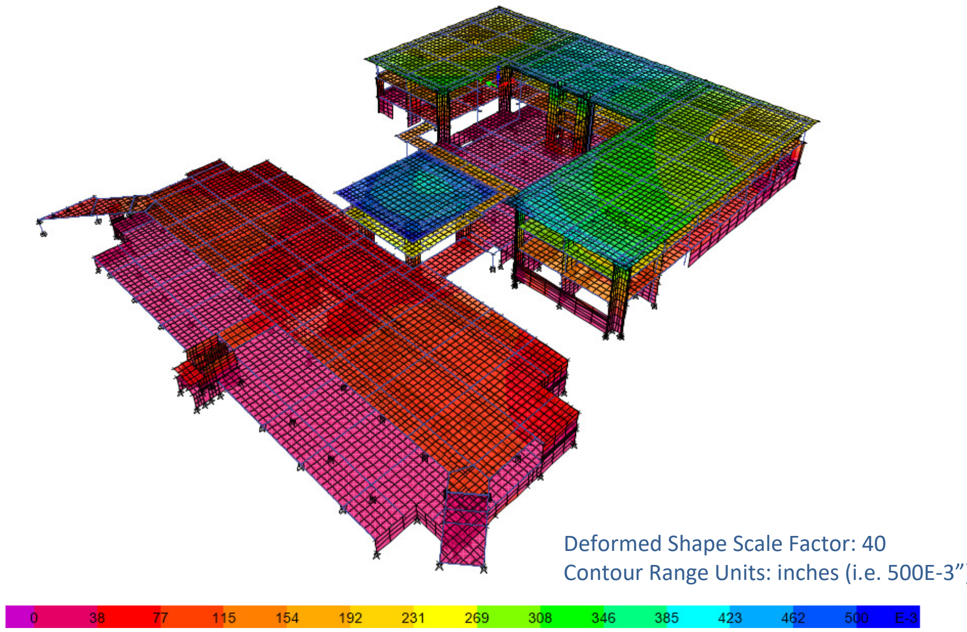

Figure 11 SRSS Directional Combination Displaced Shape

Table 10 presents a general summary of deficient lateral resisting elements for each case studied. The benefits of implementing recommended upgrades presented in Case 4 is substantial in terms of strength requirements and serviceability performance.

Table 10 General Summary of Deficient LRE

Element	D/C Ratio	Summary of Deficient LRE			
		Case 1	Case 2	Case 3	Case 4
Shearwall Piers	1.0<D/C<1.25	33	32	18	2
	1.25<D/C<1.5	16	15	10	0
	D/C>1.5	38	38	8	0
Columns	1.0<D/C<1.25	65	36	9	1
	1.25<D/C<1.5	31	20	1	1
	D/C>1.5	31	25	0	0