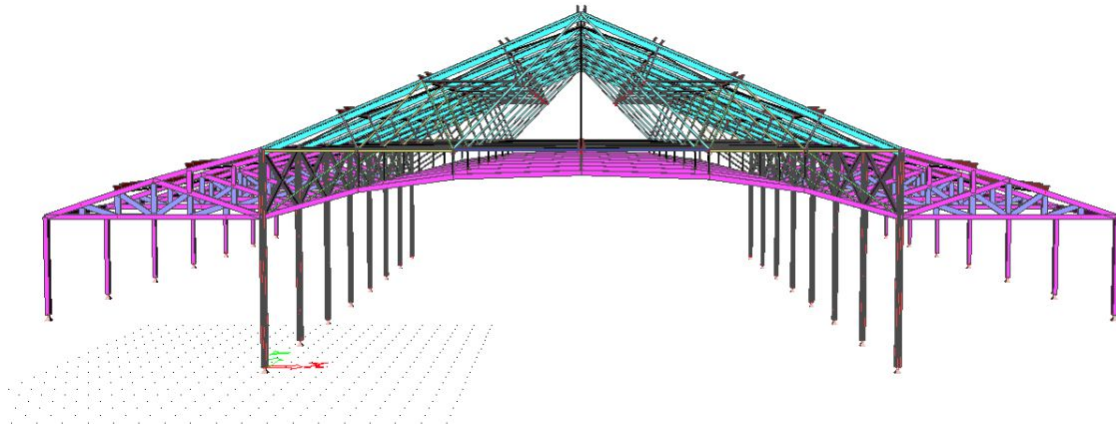




Structural Analysis – Chautauqua Institution Amphitheater

Structural Analysis of the Existing Building



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INTRODUCTION

Old Structures Engineering was retained by the Chautauqua Institution to analyze the existing structure of the amphitheater building relative to its present day structural performance and viability in the longer term. This analysis does not evaluate other factors such as accessibility issues, increased seating, site lines, comfort, acoustics, rigging needs, or back-of-house functions. This report provides our findings on how the building, in its present state, performs structurally, and offers recommendations for improving its future performance. Refer to Appendix D for a glossary of engineering terms used in the report.

EXISTING STRUCTURE

The existing amphitheater building was constructed circa 1893 using a combination of steel columns and roof trusses along with timber roof framing. The primary support for the roof is a series of eight steel trusses that sit on the steel columns. On the east and west there is a lower section of roof that is supported by timber trusses connected to the main steel columns and to timber columns at the exterior/perimeter of the building. The steel columns are built-up sections using zee shapes and a center plate connected with rivets to create an "H" section in plan. The trusses are also built-up from back-to-back angles and riveted plates as chords, with bottom chord tension tie rods. The building was subsequently reinforced in circa 1977 with the addition of tie rods along the longitudinal (east-west) axis, and reinforcement of the columns supporting the trusses.

The connections between the different steel and timber members can be classified as flexible or pinned connections. Due to the lack of rigidity in the connections, the structure is flexible under lateral forces. The existing structure has cross bracing to reduce the lateral displacement at the tops of the columns in the long direction (east-west) but in the short direction (north-south) the deflection relies entirely on the rigidity of the columns' cross section.

Per the archival drawings and supporting documentation we know that the structure was reinforced by an engineer presumably to address the movement at the tops of the columns; the first drift analysis was performed as part of that analysis.

CURRENT STRUCTURAL ANALYSIS

For structural analysis presented here, we have created a full scale model of the amphitheater structure using finite element software (SCIA Engineering). We reviewed existing drawings to understand the overall layout of the framing, and what modifications were made. We were assisted in obtaining undocumented framing sizes by John Shedd of the Chautauqua Institution. The sizes of some secondary, unmeasured, elements have been assumed based on photographs and comparison with archival drawings.

To begin the analysis, hand calculations were performed to generate the global loads and determine column reactions that were then used as a check of the computer model output for the initial set up.

The wind and snow loads, and the load combinations applicable to the building, have been calculated in agreement with the code *ASCE 7-10 Minimum Design Loading for Buildings and Other Structures*, and in compliance with the New York State code. A total of 13 different load combinations for dead, live, wind, and snow load were applied to the model (See Appendix C).

The structural analysis focused on the central "front-of-house" building and the steel elements: trusses, columns, and cross bracing. The timber structure on both sides of the main building does not contribute to the support of the main roof other than providing limited bracing against drift; the dead and snow load from the side roofs were accounted for in the analysis. The organ structure and the "back-of-house" portion of the building were not included in the model.

The following assumptions were made.

- The steel has a yield strength of at least 30ksi based on the American Institute of Steel Construction historical steel shapes reference book.
- The slenderness ratio of the members in compression were taken to be $K*L/r \leq 200$.
- The column and truss connections are adequate for the load from the connecting members based on the fact that there is no empirical evidence that the connections have failed.
- The condition of the steel, rivets, and bolts are in good condition¹.

RESULTS

STRUCTURAL CAPACITY

In general the existing steel truss members do not show over-stress in most of the load conditions analyzed. However, under wind loads and snow loads the top chords of the trusses show slightly higher values than the allowable values (See Image 1 & 2). The over-stresses do not exceed 10% of the yield strength. This temporary (transient loading) over-stress is considered to be within acceptable limits.

The column drift surveys performed in 1977 and regularly since 1999 show a permanent drift between 0.5 inches and 4.5 inches under no lateral or snow load. This permanent drift, or eccentricity at the tops of the columns, has to be taken into account because it creates bending moments in the columns even without lateral loads being applied. If a column is perfectly vertical, an applied vertical load results in compression only in the steel.

Under gravity and uniform, vertical loads, such as snow, the stresses in the columns are below the allowable capacities (acceptable). However, when the wind load or the uneven snow load is applied, the columns have to withstand the axial compressive load, the bending mo-

1. A visual review of the amphitheater by OSE was done on April 15th. It was not an exhaustive survey, however, the condition of the steel that was directly observed appeared to be good without deformation or significant corrosion. Only exposed elements were reviewed.

ment due to the lateral force (wind for example), and the bending moment due to the eccentricity of the column. The model shows that when lateral load is applied, the columns are overstressed at the portion of the column where the side timber truss meets (See Image 3 & 4)².

STRUCTURAL STABILITY

Due to the lack of rigid connections or sufficient bracing in the short direction (north-south), the existing building is very sensitive to lateral loads. In the longitudinal direction (east-west) there is steel cross bracing aligned with the columns and in the plane of the roof which contributes to control the deflection of the building along the longitudinal axis.

Under a uniform load condition the model shows active drift in the top of the columns of 0.3 inches. Under an uneven snow load the model shows an outwards drift at top of the column of 6.5 inches in the short direction, and less than 1 inch in the long direction. Under wind load the drift goes up to 8.5 inches in the short direction, and 4.5 inches in the long direction.

The structural model has been run with and without the column reinforcement from 1977, to study the affect of the intervention on the drift phenomenon. Taking into account the original column cross section, the wind drift was 12 inches. Taking into account the existing reinforcement, the drift drops to 6.5 inches. A comparison between the 1977 and the latest (1999 - 2015) column drift surveys (See Table A in appendix B) shows that in a period of 38 years the columns have continued drifting, albeit by small increments. The drift has increased on average just under 1/2 inches, however, the trend is increasing every year.

CONCLUSIONS

1. The existing steel trusses were analyzed to be adequate to withstand the code required loads. Under worst case loading (wind they show slight over-stress which is considered to be within an acceptable range for temporary loading.
2. The existing columns are over-stressed in the vicinity of the connection with the side roofs under lateral load/uneven snow load. The 1977 column reinforcement implies the engineer was aware of this over-stress because the cross-section of the column was increased at this point (See Image 5).
3. The existing column displacement, plus the induced drift caused by wind or uneven snow load can magnify the stresses in the steel columns. As seen in the drift survey by Foit-Albert Associates, since 1999 the columns have moved incrementally year after year. The last drift survey from May 2015 shows that 12 of the 16 steel columns exceed what can be considered the allowable drift ($L/500$)³.

2. Refer to Image 5. The location of the noted column over-stress, at the junction with the steel trusses and side roof, was reinforced in the 1970's. We did not incorporate the increased column cross section in the model, therefore, the over-stress may be less than calculated or possibly zero.

3. $L/500$ is the drift limit for enclosed buildings. It could be argued that the allowable drift could be increased for a structure with no cladding.

4. The effect of the permanent eccentricity, plus wind, with additional gravity load from snow, may cause an over-stress beyond the capacity of the columns resulting in failure. This risk will increase over time due to ongoing drift and the resulting column eccentricity.
5. Under wind load without snow, the eccentricity is considered a short term effect, and the columns are found to be slightly overstressed. A coupon sample of the steel would provide the exact strength of the steel versus the allowable yield stress assumed based on historical references. Exact measurement of the circa 1977 reinforcement can be incorporated into the model as well, resulting in a reduction in the calculated column stress.

RECOMMENDATIONS

Near Term: Old Structures Engineering did not perform a truss by truss, column by column survey; during a walk-through of the building we visually observed the columns from ground level, the trusses from the attic catwalk, and the overall roof from an adjacent building. There were no signs of structural over-stress or serious corrosion observed. Prior to making modifications to the existing structure in order to keep it in use, we recommend that a more detailed condition survey be performed to ensure there are no weakened areas that were not accounted for in this analysis. It should be noted that safe access in the attic is limited; a consultant should review conditions and make recommendations to improve access (e.g., fall protection, harness tie-off points, etc.).

Near Term: Temporarily brace the steel columns at each bay in the short direction, and at least in several bays in the long direction, to restrain the structure against further drift during the snow season. We recommend the temporary bracing be installed before winter to prevent further drift under unbalanced snow and wind loads. This can be achieved using crossed tension cables from the top of one column to the bottom of the adjacent column. The rods can be spliced using turnbuckles to allow tensioning post installation; refer to Image 6. Tie rods could be removed in the spring (post snow season) and update the drift study. An alternate configuration would reduce the lengths of the tie rods, but would require concrete blocks to counter the loads induced by wind and unbalanced snow on the roof; refer to Image 7.

Long Term: Reinforce steel columns, connections with the trusses, some truss chord sections, and, if needed, column foundations, to withstand the lateral loads and reduce drift. The reinforcement of the columns may be accomplished with new steel plates welded to the existing. Part of this work would require further investigation of the column bases to evaluate the capacity of the footing or timber pile support relative to proposed column reinforcement.

APPENDIX A: IMAGES

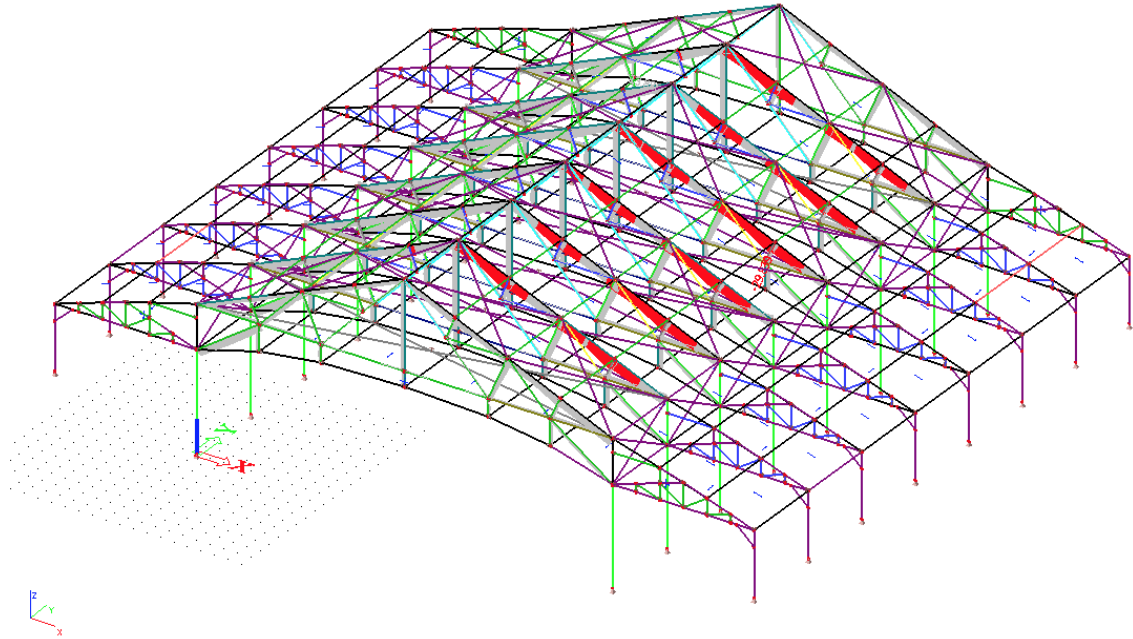


Image 1. 3D structural model. In red are the sections of the trusses over-stressed due to compressive forces under wind loads (Load Case 6).

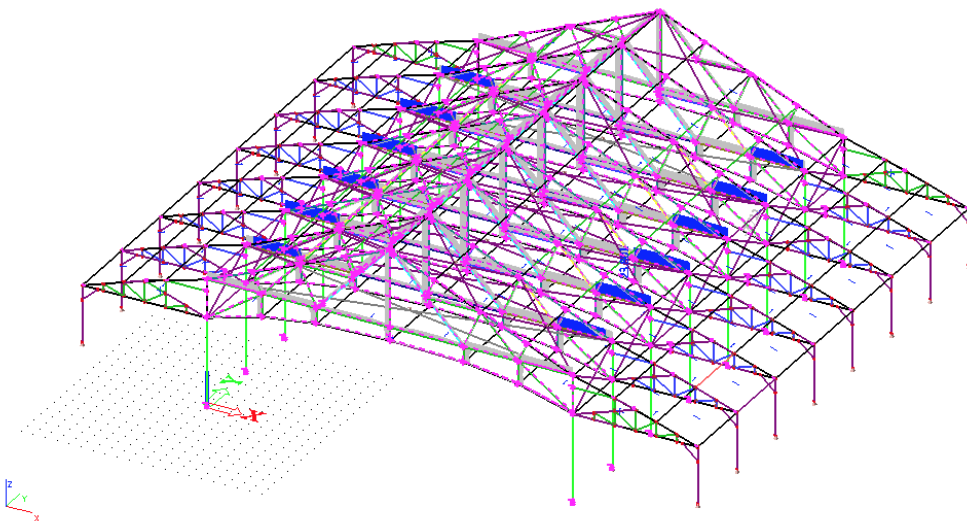


Image 2. 3D structural model. In blue are sections of the truss over-stressed due to tensile forces under an even/uniform snow load condition (Load Case 2).

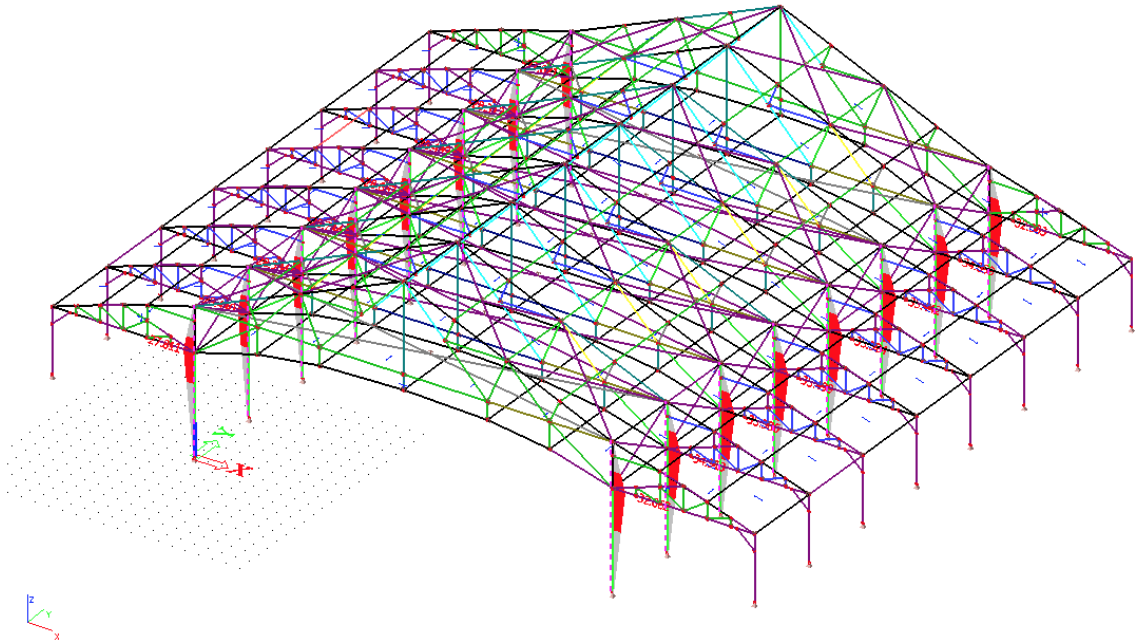


Image 3. 3D structural model. In red sections of the columns overstressed due to lateral load conditions

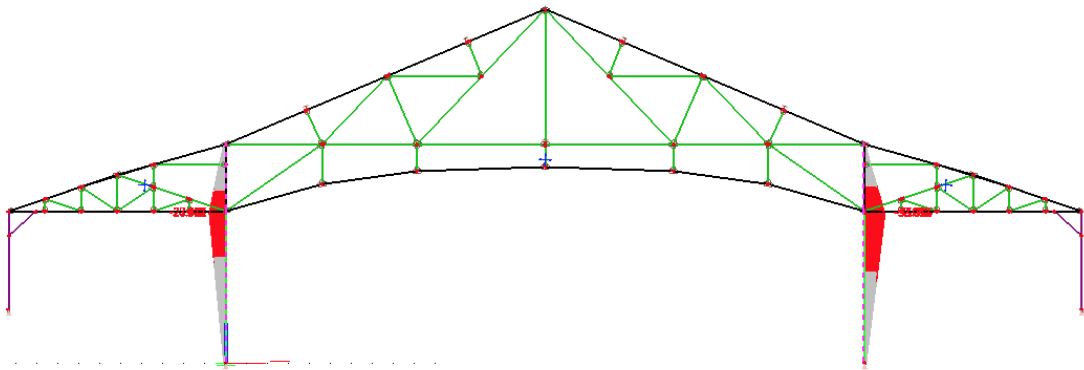


Image 4. Front view of the structural model. In red sections of the columns overstressed due to lateral load condition



Image 5. Existing columns. Circled in red increase of reinforcement at the point of hishest stresses.

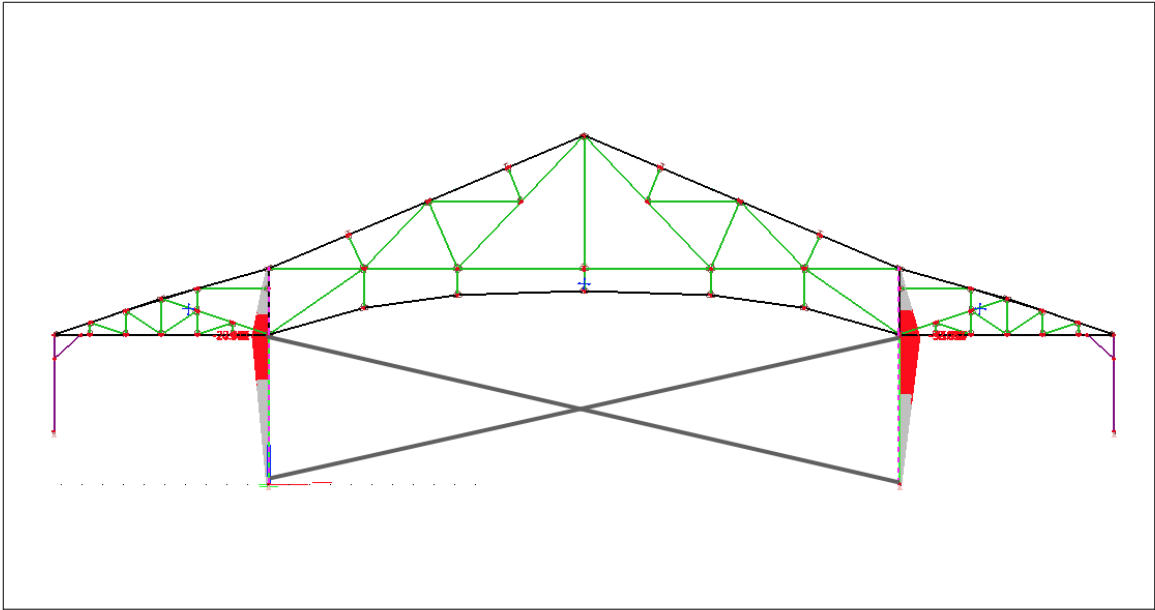


Image 6. Diagram illustrating the temporary cross bracing with spliced tension rods to resist further drift during the winter months.

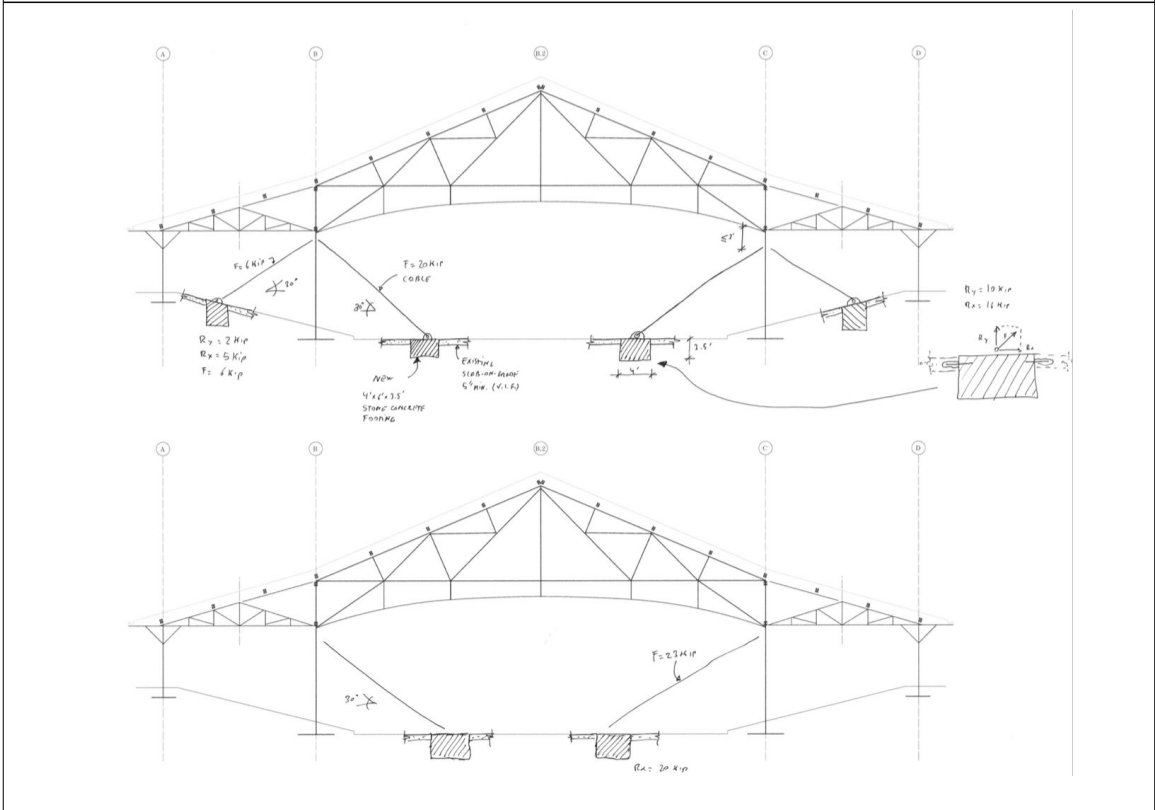


Image 7. Diagram showing an alternate temporary bracing scheme with shorter tie rods and concrete anchorage blocks.

APPENDIX B

TABLE A. 1977 – 2005 DRIFT SURVEY COMPARISON

Column	Measured column drift at ceiling elevation (in)				Difference in 38 years (in)	
	1977		2015		X	Y
	X	Y	X	Y		
D1	0.12	0.48	-0.15	0.61	-0.27	0.13
D2	1.2	-0.24		0.59		0.83
D3	-0.24	0.48		0.75		0.27
D4	-0.24	0.6		0.59		-0.01
D5	-1.2	0.48	-1.6	0.83	-0.4	0.35
D6	-0.6	3.48		3.8		0.32
D7	-1.2	2.76		3.05		0.29
D8	-0.96	4.32	-0.5	4.1	0.46	-0.22
A1	0	1.08	-0.5	0.95	-0.5	-0.13
A2	-1.2	1.68		1.6		-0.08
A3	-3.48	0.84		1.05		0.21
A4	-2.16	0.24		0.11		-0.13
A5	-2.76	-1.32	-3.18	-1.26	-0.42	0.06
A6	-3.36	-0.12		-1.01		-0.89
A7	-2.4	0.24		0.34		0.1
A8	-0.96	-0.12	-1.1	-0.32	-0.14	-0.2

* Column name corresponds to names used in the column monitoring drawings TS-100 & TS-101 of Foit Albert Associates.

xxx Denotes drifts above L/500, second order analysis in columns should be taken into account
xxx Denotes an increase in deflection above 1/2" between the 2015 and 1977 readings.

APPENDIX C

LOAD COMBINATIONS

ASCE 7-10 Load Combinations:

1. **LC1** - Dead + Live Load (roof and attic)
2. **LC2** - Dead + Snow Load (Even)
3. **LC3** - Dead + Snow Load (Uneven)
4. **LC4** - Dead + 0.75 Live Load + 0.75 Snow Load (Even)
5. **LC5** - Dead + 0.75 Live Load + 0.75 Snow Load (Uneven)
6. **LC6** - Dead + 0.6 Wind Load (case A, downward load)
7. **LC7** - Dead + 0.6 Wind Load (case B, uplift load)
8. **LC8** - Dead + 0.6 Wind Load (Y direction) (in wall and roof acting inwards)
9. **LC9** - Dead + 0.6 Wind Load (Y direction, wall in suction and roof inwards)
10. **LC10** - Dead + 0.6 Wind Load (Y direction, wall and roof in suction)
11. **LC11** - Dead + 0.75 Live Load + 0.45 Wind Load (case A) + 0.75 Snow (Even)
12. **LC12** - Dead + 0.75 Live Load + 0.45 Wind Load (Y direction) + 0.75 Snow (Even)
13. **LC13** - Dead + 0.75 Live Load + 0.45 Wind Load (Y direction) + 0.75 Snow (Uneven)

APPENDIX D

Glossary of Engineering Terms

- **Allowable Stress Design (ASD):** The calculations to evaluate the existing structure of the amphitheater utilize ASD. Defined by American Society of Civil Engineers - 7 (ASCE-7), *Minimum Design Loads for Buildings and Other Structures* as “A method of proportioning structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called working stress design)”. This design method compares allowable stress to actual stress. The factor to define allowable stress is a percentage of the material's yield stress; keeping stresses below the allowable ensures a "built in" factor of safety. Modern steel formulas were used in conjunction with assumed historical steel yield strength.
- **Axial Load:** A load applied parallel with and along a member's longitudinal axis. A typical column experiences axial load, a hanger has a tensile load. An axial load may be either compressive (squeezing) or tensile (stretching).
- **Dead Load:** Dead load describes the loads from the weight of the permanent components of the structure.
- **Deflection:** Deflection is the displacement of a structural member under a load.
- **Drift:** In traditional engineering terms, lateral deflection is the predicted movement of a structure under lateral loads; and story drift is defined as the difference in lateral deflection between two adjacent stories. For the purposes of this report, beginning with the first drift study in circa 1977, the cumulative, permanent lateral deflection at the tops of the columns is referred to as drift.
- **Dynamic Load:** This type of load varies over time such as wind or seismic.
- **Footing:** A footing is a slab of concrete under a column, wall, or other structural to transfer the loads of the member into the surrounding soil.
- **Gravity Loads:** Loads applied to a structure in the vertical, downward direction, such as self weight, occupancy load, snow.
- **Lateral Loads:** Loads applied to a structure horizontally such as wind, seismic.
- **Live Load:** Gravity load from occupancy of a building.
- **Modulus of Elasticity (E):** The value is usually 29,000,000 psi for structural steels and is also called Young's Modulus; it relates to the stiffness of a material. The modulus calculates the slope of the straight-line portion of the stress-strain curve in the elastic range of a material.
- **Moment:** Moment is the tendency of a force to cause a rotation about a point or axis which in turn produces bending stresses.

- **Slenderness:** Compression members, such as columns, are mainly subjected to axial forces. The principal stress in a compression member is therefore the normal stress (force/area). When a compression member becomes longer, the role of the geometry and stiffness (modulus of elasticity) becomes more and more important. For a long (slender) column, buckling occurs well before the normal stress reaches the strength of the column material. For example, pushing on the ends of a business card or bookmark can easily reproduce the buckling.
- **Yield Strength: Denoted F_y , it is the stress a material, such as steel, can withstand without permanent deformation.** Below the yield strength the material is in the elastic range, where deformation is not permanent. Allowable Stress Design applies reduction factors to the yield strength to obtain allowable stress levels for various loading conditions.