URSA MAJOR

ELEMENTARY SCHOOL

ASCE 41-17 Collapse Prevention Evaluation

Findings Report

August 5, 2022

By: PND Engineers, Inc

In February 2022, PND Engineers (PND) completed a Tier 1 screening of Ursa Major Elementary school. The Tier 1 screening followed the performance-based design procedures found in ASCE 41-17 *Seismic Evaluation and Retrofit of Existing Buildings*. As part of this Tier 1 evaluation, PND completed a series of checklists. These checklists were designed to identify common deficiencies within existing buildings allowing engineers to discover and flag potential concerns. PND's Tier 1 evaluation of Ursa Major Elementary School identified 59 compliant items, 20 non-compliant items, and 1 unknown item.

Based on these findings, the Anchorage School Districted (ASD) engaged PND to investigate the seismic performance of the Ursa Major Elementary School further by completing Tier 2 evaluations of the steel framed and masonry additions and a Tier 3 evaluation of the original concrete structure. At this time, these evaluations are ongoing and have not yet been completed. The findings of these Tier 2 and Tier 3 evaluations are being used to create a report which will provide a more complete discussion of the seismic deficiencies present and provide conceptual repair and retrofitting options. The proposed options could be implemented to bring the structure up to the ASD performance goal of Life Safety (S-4) under a BSE-2N seismic hazard, and Damage Control (S-2) under a BSE-1N seismic hazard. The conceptual findings report will also identify specific elements and areas within the structure that are in need of retrofit so that the approximate repair and retrofitting costs may be estimated.

PND recently shared the preliminary findings of the Tier 2 & Tier 3 evaluations with Larry Morris and Dana Menendez of the ASD. Per their request, before publishing the final conceptual findings report, it was requested that PND compile information pertaining to the building performance in regards to the Collapse Prevention (S-5) performance level. This performance level was not part of the original evaluation effort so the building components reviewed for this additional performance level were limited to those components most likely to be identified as deficient. This partial review allowed for a more rapid evaluation while still providing an assessment of the buildings expected performance.

The ASCE 41-17 standard relies on *Basic Performance Objectives for Existing Buildings* (BPOE). This ASCE standard provides procedures and performance criteria for the evaluation of existing buildings. Using the ASCE 41-17 standard, a performance level for an existing structure can be determined. The desired level of performance is then evaluated for a particular magnitude of earthquake to determine if the building performance is adequate.

The magnitudes of earthquakes considered during the different evaluations are:

- BSE-2E is an earthquake with an expected probability of return of 5% in 50 years. This would result in an earthquake with a magnitude of roughly M8. After an earthquake of this magnitude major roadway damage, loss of utilities, and moderate building damage are expected.
- BSE-1E is a smaller earthquake with an expected probability of return of 20% in 50 years. This would result in an earthquake of a magnitude of approximately M6. This level of earthquake is not uncommon in the Anchorage bowl. Although it would likely cause alarm and cracks in shear walls and non-structural components, it is not likely for this magnitude of earthquake to cause significant damage to structures, roads, or utilities.
- BSE-2N is also known as the Maximum Considered Earthquake, this would be the largest earthquake expected for the area in question. This level of shaking in the Anchorage area is

capable of being M9+ earthquake. At this intensity one would likely see larges cracks in the ground, significant roadway damage, loss of utilities, and most buildings would be highly damaged.

- BSE-1N is a magnitude of earthquake that is two thirds of the maximum considered earthquake. This is the standard level used for new designs. This magnitude of earthquake would result in an approximately M7 level. After an earthquake of this magnitude, it would be expected to see roadway damage, ground cracking, damage to houses, utilities, and other buildings.

The typical target level of performance for Risk Category III structures chosen by the ASD is Damage Control (S-2) at a BSE-1N level seismic event and Limited Life Safety (S-4) at a BSE-2N seismic event. However, in this case, the level of performance selected for review is Collapse Prevention (S-5) at both the BSE-1E & BSE-2E seismic events.

Safety performance levels found in ASCE 41 are defined as follows:

- *Immediate Occupancy (S-1)* – The structure will retain the pre-earthquake strength and stiffness and can be utilized immediately. Few to no injuries should occur to persons within the structure due to building failure.

- *Damage Control (S-2)* – Some damage will occur to the building, with small permanent drift. Damage should be economical to repair. The building is capable of being occupied following shaking, but damage should be addressed as soon as feasible.

- *Life Safety (S-3)* – Moderate damage will be present in the building, with some residual strength left in the elements. Minor permanent drift will be present. The building may be beyond economical repair. Injuries due to structural failure should be few.

- *Limited Safety (S-4)* – Moderate to severe damage will be present. The building will have permanent drift, will be beyond economical repair, and should not be reused following an earthquake. Damage to the structure may cause injuries or obstacles to evacuation, but these should be minor.

- *Collapse Prevention (S-5)* – Severe damage will be present throughout the structure. Little residual strength and stiffness will remain, but load-bearing columns and walls should function. Large permanent drifts will exist in the structure and exits may be blocked. The building is near collapse. This structure will pose a hazard to human safety and will not be reusable. Risk of injury due to structural damage is high.

PND revised the analysis of the structure to include a review of the structure's lateral force resisting system (LFRS) at the lower performance level of Collapse prevention (S-5). The analysis found that the original 1950s portion of the Ursa Major LFRS, under the larger BSE-2E event, had concrete shear wall demand capacity ratios (DCRs) along almost every grid line that were greater than 100%. The DCRs exceeding 100% varied from 101% to 246%. Looking at Collapse Prevention (S-5) under the smaller BSE-1E event, half of the shear wall lines had at least one shear wall pier with a DCR greater than 100%. The overstressed piers had DCRs that ranged from 104% to 167%.

The attached spreadsheets shows the demand capacity ratios (DCRs) for shear wall piers in the lateral force resisting system. The demand capacity ratios at the Collapse Prevention (S-5) performance objective are given in APPENDIX A for the BSE-1E seismic hazard and in APPENDIX B for the BSE-2E seismic hazard. Both the 1E and 2E hazards show that the shear wall piers will experience loading demands beyond their capacities (>100%). Additionally, APPENDIX C (BSE-1E) and APPENDIX D (BSE-2E) provide contour maps that show a visual representation of the extent of deficiencies. In all cases, the dark blue zones represent areas where the material is being overstressed at the collapse prevention level of performance. The colored gradient contours are intended to be generally informative, showing where DCRs are anticipated to exceed 100%, but the graphics and do not represent a definite pass/fail criterion.

This structure is a reinforced concrete building built in the 1950s before significant revisions to the code were made to improve building performance. Therefore, it was not designed to meet the much higher ductility requirements of today's building codes. Many of the ductility requirements for concrete structures were not added to the code until the first code cycle after the Northridge Earthquake. After this major earthquake, the code incorporated the need for additional steel reinforcement and seismic detailing to increase ductility. Ductility is very important for a few reasons. First, ductile behavior helps the structure's elements dissipate and dampen the seismic forces the building experiences. Second, ductility prevents elements from fracturing and failing in a brittle manner. Brittle failure tends to be sudden with little to no prior warning or signs of distress and can trigger a rapid succession of failures in the surrounding structure. This structure is not reinforced in a way that would provide ductile failure modes as required and detailed for in today's building codes.

Deficiency of the lateral force resisting system in a building of this age, 70 years, is not surprising since many design improvements have been made to the building code in that time. The original drawings of the school show that the original building has little to no elements to provide resistance to lateral forces. It is assumed that the designers in the 1950s intended for the lateral forces to be transferred via rigid diaphragms to the shear walls in the art and multipurpose room (MPR) wings at each end of the building. Design tools in the 1950s, would not have included computer modeling and without the aid of mathematical modeling software, the designers would have assumed the diaphragm to be infinitely rigid to transfer lateral forces to the shear walls to simplify calculations. With the availability of modern analysis software, diaphragms can be fairly easily modeled as semi-rigid. If a semi-rigid diaphragm is used but the diaphragm could have in fact been defined as a rigid diaphragm, the modeled results between the two types of diaphragms will be nearly identical. ASCE 41 section 10.10.2.2 requires that diaphragm flexibility be considered in the model when the length to width ratio exceeds 2.0. The geometry of this structure exceeds the limit of 2. Therefore, PND used structural analysis software to model the diaphragm as semi-rigid so that the in-plane deformation of the diaphragms would be calculated. This model showed large discontinuities between the lateral force resisting system of the art room and multipurpose room, and in the lateral force resisting system elements as a whole. The large distance between shear walls lines resulted in large displacements in the middle of the diaphragm. This diaphragm displacement results in lateral forces being transmitted to non-lateral members such as columns and non-structural partition walls. These members were not designed to resist lateral forces and will likely be overstressed if loaded. PND evaluated the structure under purely gravity loading and the building gravity elements performed adequately and they did not exceed the DCR for the elements. However, once lateral loads are added to these elements, they no longer have sufficient member capacity to resist the forces being applied.

ASCE 41 provides an assessment procedure for individual components within in a building. The evaluation shows whether the individual component does or does not satisfy the criteria. However, the ASCE 41 standard does not provide a definitive metric for how many individual components must fail to result in a building collapse under the design event. Therefore, even though some of the building components do not meet the Collapse Prevention level of performance when evaluated under the ASCE 41-17 standard, this does not mean the building will absolutely collapse under these design seismic events. What exceedance of DCR under these events means is that building members can be, and likely have been overstressed and yielded. Once members yield, they no longer behave elastically, as they were designed to do, this results in nonlinear behavior. This nonlinear behavior results in the loss of member capacity and consequently a redistribution of forces to elements that were not intended to take lateral loads, such as gravity-only columns or non-structural partition walls. This redistribution of forces can result in a cascading path of partially or completely failed components. This is why a building may appear to be performing adequately but under a lateral force excitation a local failure or collapse may occur.

Since this concrete building was designed before ductility requirements were incorporated into the building code, it does not meet today's ductility requirements for concrete construction. Therefore, the failure of the concrete elements will likely be brittle and dynamic. Brittle failures occur quickly and with little to no warning. Brittle failure concerns are further compounded by the non-linear distribution of forces to members not intended to take lateral loads and the progressive failure of elements as described above. A local brittle failure has the potential to cause a cascading failure that could lead to a larger global brittle failure or collapse.

PND is currently working with the Anchorage School District to open up a few concealed areas to observe the condition of the concrete walls, columns and slabs. PND selected the areas for observations based on the results and findings from analysis software model. PND requested these areas be opened for further investigation since these particular areas were identified as being highly loaded elements in the building model. At this time, PND has not been able to fully observe these areas since they have not passed the hazardous material air clearances. However, PND has had some limited access and received some preliminary photos of the exposed areas from the hazardous materials abatement team. There is diagonal cracking at the shear wall near the art room. These cracks were filled with an unknown adhesive at some point. The repairs were likely completed after the 1964 earthquake but there are no records of these repairs so the PND can only infer when the repairs were completed. Many of the cracks are approximately 0.050 inches wide. However, some of the cracking was observed to be much wider at approximately 1 inch thick at its widest point. The epoxy at the wider cracks was installed over a black waterproofing which contains asbestos so the tan epoxy could not be removed by PND for further observation of the crack. The cracking could be seen on both sides of the shear wall so PND believes the cracking extends through the entire thickness of the wall. Engineering Health and Safety Consultants (EHS) also shared photos during a conference call on July 29th, 2022 which showed what appeared to be concrete crushing at the corners of a concrete shear wall near the administration offices at the front of the building. The crushing observed in the photos was likely due to tension and compression forces overstressing the shear wall panel. Once the abatement is complete, PND will fully observe and document the damage found in the exposed areas. For now, PND can only conclude that damage from past seismic events has been found and that the damage is not structurally insignificant.

PND has also had eight concrete cores cut from the basement walls to be tested to verify the concrete compressive strength specified on the design drawings and to permit the use of a knowledge factor, *k* of

1.0. PND assumed a knowledge factor of 1.0 for all of the calculations completed pending the concrete testing results. PND used the procedures in ASCE 41 Chapter 10 to adjust the lower bound compressive strength of 2500 psi to an expected compressive strength of 3750 psi. The results included in this report were completed using this expected concrete compressive strength of 3750 psi. Today, PND received the concrete core break results. The average concrete compressive strength for the 8 cores was 7135 psi with a standard deviation of 1257 psi. That would set the lower bound of the concrete compressive strength based on testing at approximately 5878 psi. This increase in compressive strength could be used in the analysis of the shear walls but PND would recommend additional sampling of the walls throughout the building before using the higher compressive strength for all concrete elements. This testing was the minimum required per ASCE 41 to permit the use of a 1.0 for the knowledge factor. Increasing the concrete compressive strength would reduce the DCRs but would not bring many of the piers below the 100% threshold.

The two building additions were also evaluated as part of the original tier 1 evaluation. Those additions include the 1989 Instructional Materials Center expansion and the 1997 East and West wing expansions. The Instructional Materials Center expansion is a concrete masonry unit (CMU) addition with a light wood sheathed, flexible, diaphragm. The 1989 addition was not seismically isolated from the original 1950s concrete building. However, due to the geometry of the addition, as well as the flexible diaphragm, this addition will primarily act independently. The 1997 addition included a gym at the plan east wing, which is seismically isolated from the original 1950s structure. The East gym addition has its own independent CMU shear wall system with a flexible metal deck diaphragm. The West end of the 1997 addition consists of the expansion of the multipurpose room to include a music room, stage, and additional classrooms. The West wing expansion is seismically isolated, and has a combination of CMU shear walls supporting the classrooms, and steel braced frames, supporting the music room and hallway between the music room and classrooms. Both the CMU and braced framed systems have flexible metal deck diaphragms.

No deficiencies were found in the CMU walls of the 1989 concrete masonry unit (CMU) addition or the 1997 additions under the Tier 1 evaluation. However, several deficiencies were found during the Tier 1 evaluation for the steel braced frames. The Tier 1 deficiencies triggered the need for a higher level of analysis. Since the brace frame additions are seismically isolated from the main 1950s structure, and the surrounding CMU was not found to be deficient, PND was able to perform a Tier 2 evaluation of the steel braced frames separate from the Tier 3 evaluation of the original concrete structure.

At this time point of our analysis, PND can show that the braced frame portion of the building is deficient under the Tier 2 analysis and will require a retrofit to meet the Life Safety and Damage Control performance levels. However, this portion of the building was not checked under collapse prevention, as the deficiency of the braced frames does not appear to be as extreme as the original concrete portion of the building, and the attached non-deficient CMU system can provide redundancy to the braced frames. This redundancy makes collapse of the steel framed additions unlikely.

The purpose of this document is to inform the Anchorage School District of the expected level of performance of the current structure under the Collapse Prevention performance level using the procedures of the ASCE 41-17 standard. PND's evaluation did not include a review of all building elements. The evaluation was focused on the building's most concerning lateral force resisting elements which are the concrete shear walls. Therefore, it should not be assumed that the deficiencies are limited to the elements discussed in this report. PND's analysis of the structure found that the original concrete portion of the school does not meet the Collapse Prevention Level of Performance as defined in ASCE 41. Numerous shear wall piers were found to be beyond their demand capacity ratios. Due to the quantity of deficient shear wall piers discovered and the lack of redundancy, PND believes that there is a significant potential for a partial or complete building collapse during a BSE-2E seismic event. There is also a likely possibility of a partial building collapse during the smaller BSN-1E seismic event. The building has performed satisfactory in past seismic events but when analyzed using the procedures found in ASCE 41-17, many of the structure's shear wall piers have been found to be insufficient to meet the Collapse Prevention level of performance. Due to the potential for building collapse, PND strongly recommends that the original concrete portion of the building be immediately strengthened via retrofitting or replaced by a new structure to increase the building's expected level of performance.

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APPENDIX A — BSE-1E DEMAND CAPACITY RATIOS

P | N | D
ENGINEERS, INC.

221048 Ursa Major Colapse Prevention

BSE-1E

Calculated By: JJL on 08/03/2022
Reviewed By: DNB

Grid D	Grid 1
Grid Fa	Grid 8
Grid C	Grid 12
Grid A	Grid 14
Grid J	Grid 18
Grid 2	
Grid 5	
Grid 6	
Grid 20	
Grid 21	
Grid 23	
Grid 24	

Note:

1.) Colors indicate the line shear pier may be found on.
2.) Shear piers are evaluated locally.

LABEL	EVENT	LC	TYPE	BNDRY ELE	COMP TYPE	DCR_V	DCR_M	DCR_C	DCR_T	DCR_{CM}	DCR	FAILURE TYPE	
						Red indicates Demand Capcity Ratio (DCR) >100%						Columns identiy what area/s DCR is exceeded.	
P1	BSE-1E	LONG		NO	PRIMARY	46.14%	13.05%	17.26%	14.57%	5.03%	46.14%		
P2	BSE-1E	LONG		NO	PRIMARY	38.87%	19.75%	18.53%	142.39%	18.52%	142.39%	AXIAL TENSION	
P3	BSE-1E	LONG		NO	PRIMARY	80.68%	14.67%	18.41%	35.06%	21.18%	80.68%		
P4	BSE-1E	LONG		NO	PRIMARY	90.58%	14.08%	19.47%	8.70%	21.50%	90.58%		
P5	BSE-1E	LONG		NO	PRIMARY	82.46%	12.09%	10.88%	37.91%	11.81%	82.46%		
P6	BSE-1E	LONG		NO	PRIMARY	65.21%	10.80%	8.88%	95.25%	12.10%	95.25%		
P7	BSE-1E	LONG		NO	PRIMARY	138.72%	55.71%	11.31%	32.47%	13.71%	138.72%	SHEAR	
P30	BSE-1E	LONG		NO	PRIMARY	107.08%	2.56%	11.06%	71.44%	8.38%	107.08%	SHEAR	
P31	BSE-1E	LONG		NO	PRIMARY	51.12%	30.36%	6.91%	37.96%	9.33%	51.12%		
P8	BSE-1E	LONG		NO	PRIMARY	10.64%	1.38%	3.78%	23.78%	4.48%	23.78%		
P9	BSE-1E	LONG		NO	PRIMARY	11.21%	2.06%	2.98%	6.87%	3.31%	11.21%		
P10	BSE-1E	LONG		NO	PRIMARY	101.37%	10.49%	19.80%	193.69%	23.96%	193.69%	AXIAL TENSION	SHEAR
P11	BSE-1E	LONG		NO	PRIMARY	122.73%	14.21%	11.33%	85.33%	10.37%	122.73%	SHEAR	
P12	BSE-1E	LONG		NO	PRIMARY	167.03%	18.10%	12.81%	119.03%	17.45%	167.03%	SHEAR	AXIAL TENSION
P13	BSE-1E	LONG		NO	PRIMARY	6.41%	0.83%	2.41%	0.00%	2.30%	6.41%		
P14	BSE-1E	TRANS		NO	PRIMARY	71.43%	1.98%	6.08%	26.32%	3.75%	71.43%		
P21	BSE-1E	TRANS		NO	PRIMARY	11.95%	0.90%	1.26%	6.13%	0.95%	11.95%		
P15	BSE-1E	TRANS		NO	PRIMARY	103.89%	4.01%	7.90%	0.00%	8.22%	103.89%	SHEAR	
P16	BSE-1E	TRANS		NO	PRIMARY	46.97%	14.35%	12.88%	33.65%	15.99%	46.97%		
P17	BSE-1E	TRANS		NO	PRIMARY	17.27%	2.07%	4.95%	19.28%	5.36%	19.28%		
P18	BSE-1E	TRANS		NO	PRIMARY	7.91%	1.48%	3.17%	7.80%	3.40%	7.91%		
P19	BSE-1E	TRANS		NO	PRIMARY	140.00%	15.28%	7.64%	38.13%	9.52%	140.00%	SHEAR	
P20	BSE-1E	TRANS		NO	PRIMARY	162.45%	17.08%	9.60%	45.19%	10.57%	162.45%	SHEAR	
P22	BSE-1E	TRANS		NO	PRIMARY	52.91%	5.59%	7.44%	45.66%	9.47%	52.91%		
P23	BSE-1E	TRANS		NO	PRIMARY	7.28%	0.77%	1.00%	7.86%	0.79%	7.86%		
P24	BSE-1E	TRANS		NO	PRIMARY	52.88%	9.59%	4.57%	49.96%	6.76%	52.88%		
P25	BSE-1E	TRANS		NO	PRIMARY	43.99%	2.00%	5.27%	3.39%	5.15%	43.99%		
P26	BSE-1E	TRANS		NO	PRIMARY	53.60%	0.58%	8.56%	109.08%	12.33%	109.08%	AXIAL TENSION	
P27	BSE-1E	TRANS		NO	PRIMARY	15.86%	0.68%	2.14%	20.12%	0.87%	20.12%		

APPENDIX B — BSE-2E DEMAND CAPACITY RATIOS

221048 Ursa Major Colapse Prevention

Calculated By: JJL on 08/03/2022
Reviewed By: DNB

BSE-2E

P | N | D ENGINEERS, INC.

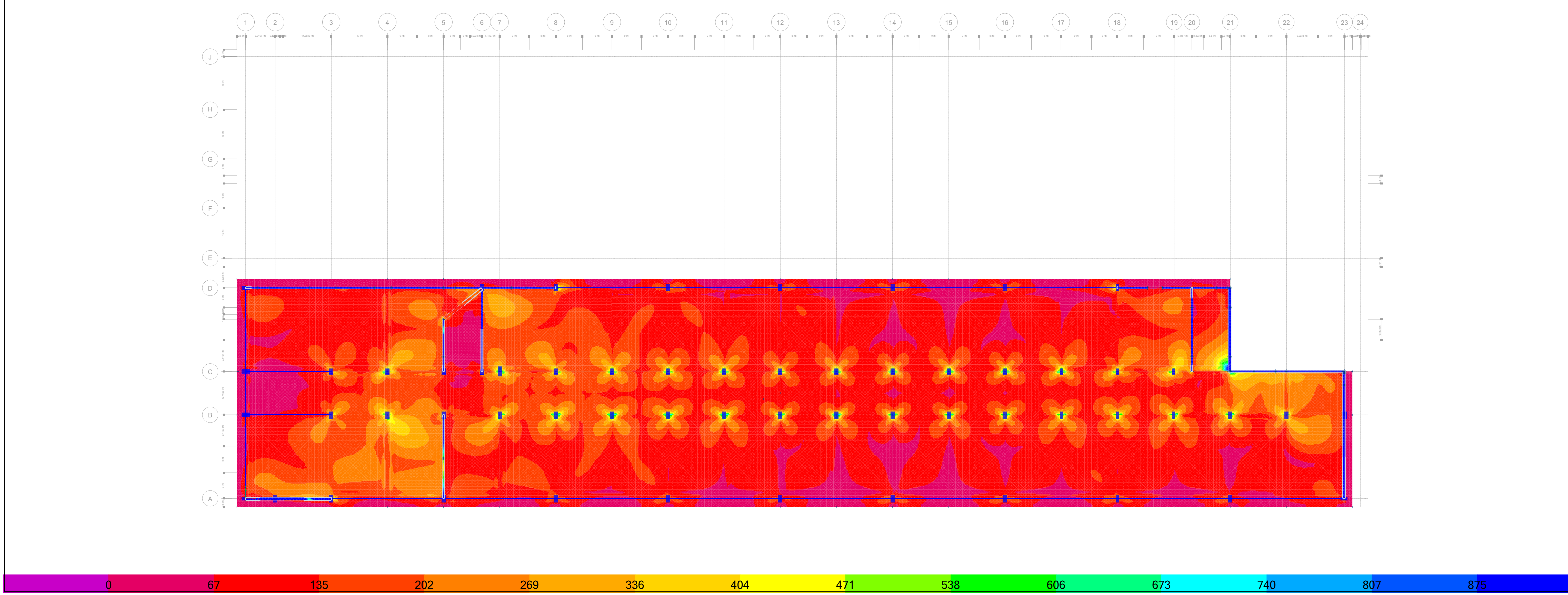
Grid D	Grid 1
Grid Fa	Grid 8
Grid C	Grid 12
Grid A	Grid 14
Grid J	Grid 18
Grid 2	
Grid 5	
Grid 6	
Grid 20	
Grid 21	
Grid 23	
Grid 24	

Note:

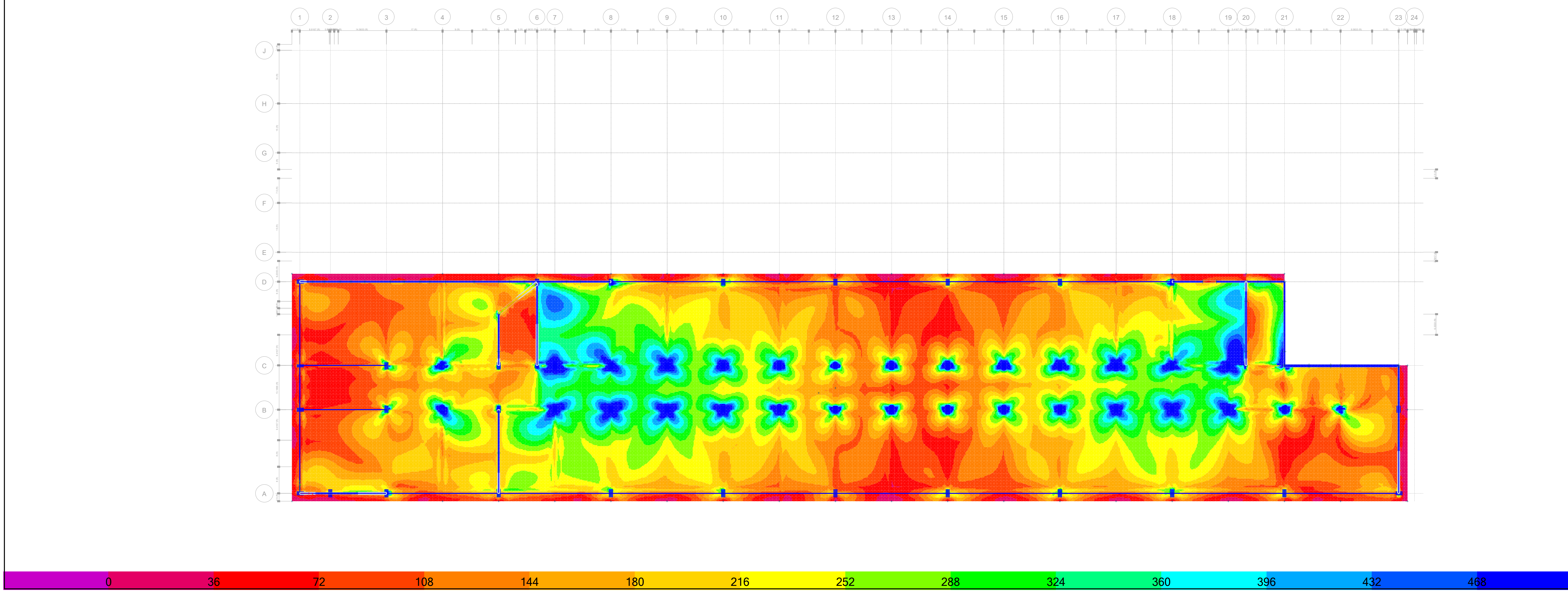
1.) Colors indicate the line shear pier may be found on.
2.) Shear peirs are evaluated locally.

LABEL	EVENT	LC	TYPE	BNDRY ELE	COMP TYPE	DCR_V	DCR_M	DCR_C	DCR_T	DCR_{CM}	DCR	FAILURE TYPE	
						Red indicates Demand Capcity Ratio (DCR) >100%						Columns identiy what area/s DCR is exceeded.	
P1	BSE-2E	LONG		NO	PRIMARY	62.29%	11.40%	21.03%	37.65%	28.03%	62.29%		
P2	BSE-2E	LONG		NO	PRIMARY	53.49%	27.06%	22.88%	206.44%	22.93%	206.44%	AXIAL TENSION	
P3	BSE-2E	LONG		NO	PRIMARY	112.01%	20.73%	20.49%	65.65%	24.34%	112.01%	SHEAR	
P4	BSE-2E	LONG		NO	PRIMARY	125.81%	28.61%	20.95%	30.42%	23.85%	125.81%	SHEAR	
P5	BSE-2E	LONG		NO	PRIMARY	126.32%	17.04%	12.38%	60.45%	4.80%	126.32%	SHEAR	
P6	BSE-2E	LONG		NO	PRIMARY	90.74%	14.75%	11.44%	133.50%	15.93%	133.50%	AXIAL TENSION	
P7	BSE-2E	LONG		NO	PRIMARY	191.44%	76.95%	12.83%	53.17%	17.19%	191.44%	SHEAR	
P30	BSE-2E	LONG		NO	PRIMARY	149.47%	3.72%	13.39%	106.36%	9.89%	149.47%	SHEAR	AXIAL TENSION
P31	BSE-2E	LONG		NO	PRIMARY	70.73%	42.04%	8.14%	56.06%	11.30%	70.73%		
P8	BSE-2E	LONG		NO	PRIMARY	13.99%	1.86%	4.56%	35.45%	5.39%	35.45%		
P9	BSE-2E	LONG		NO	PRIMARY	15.50%	2.81%	3.33%	11.79%	3.86%	15.50%		
P10	BSE-2E	LONG		NO	PRIMARY	140.80%	14.88%	25.42%	277.59%	31.18%	277.59%	AXIAL TENSION	SHEAR
P11	BSE-2E	LONG		NO	PRIMARY	171.45%	19.94%	13.96%	124.79%	13.58%	171.45%	SHEAR	AXIAL TENSION
P12	BSE-2E	LONG		NO	PRIMARY	246.43%	26.98%	16.31%	184.09%	22.76%	246.43%	SHEAR	AXIAL TENSION
P13	BSE-2E	LONG		NO	PRIMARY	9.01%	1.20%	2.51%	0.00%	2.48%	9.01%		
P14	BSE-2E	TRANS		NO	PRIMARY	100.81%	2.74%	7.09%	41.39%	4.58%	100.81%	SHEAR	
P21	BSE-2E	TRANS		NO	PRIMARY	16.40%	0.95%	1.49%	9.56%	1.03%	16.40%		
P15	BSE-2E	TRANS		NO	PRIMARY	144.47%	5.43%	8.01%	0.00%	8.45%	144.47%	SHEAR	
P16	BSE-2E	TRANS		NO	PRIMARY	65.04%	19.94%	14.96%	51.32%	19.16%	65.04%		
P17	BSE-2E	TRANS		NO	PRIMARY	23.64%	2.66%	5.72%	30.86%	6.15%	30.86%		
P18	BSE-2E	TRANS		NO	PRIMARY	10.99%	2.03%	3.54%	13.43%	3.99%	13.43%		
P19	BSE-2E	TRANS		NO	PRIMARY	194.95%	21.29%	9.13%	57.82%	12.33%	194.95%	SHEAR	
P20	BSE-2E	TRANS		NO	PRIMARY	226.83%	23.89%	12.00%	65.15%	13.07%	226.83%	SHEAR	
P22	BSE-2E	TRANS		NO	PRIMARY	73.71%	7.86%	8.91%	67.88%	11.70%	73.71%		
P23	BSE-2E	TRANS		NO	PRIMARY	10.01%	0.87%	1.25%	11.05%	0.92%	11.05%		
P24	BSE-2E	TRANS		NO	PRIMARY	73.12%	13.18%	5.97%	71.01%	8.82%	73.12%		
P25	BSE-2E	TRANS		NO	PRIMARY	61.39%	3.42%	5.71%	10.03%	5.78%	61.39%		
P26	BSE-2E	TRANS		NO	PRIMARY	74.71%	0.78%	11.57%	153.90%	16.88%	153.90%	AXIAL TENSION	
P27	BSE-2E	TRANS		NO	PRIMARY	17.74%	2.43%	2.71%	28.73%	3.95%	28.73%		

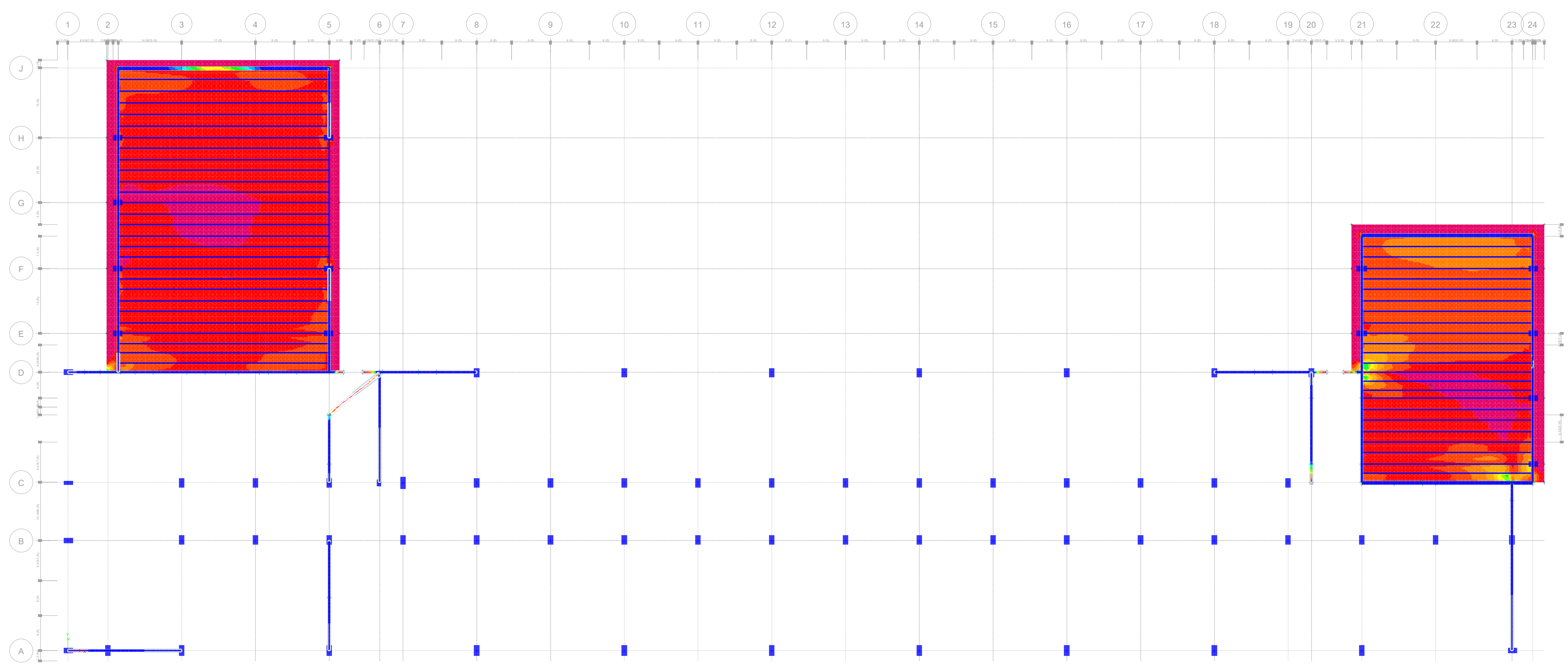
APPENDIX C — BSE-1E STRESS MAPS



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - ROOF - Z = 28.0833 (ft) Stress S12 Diagram Max (1E LONG COMP) [lb/in²]

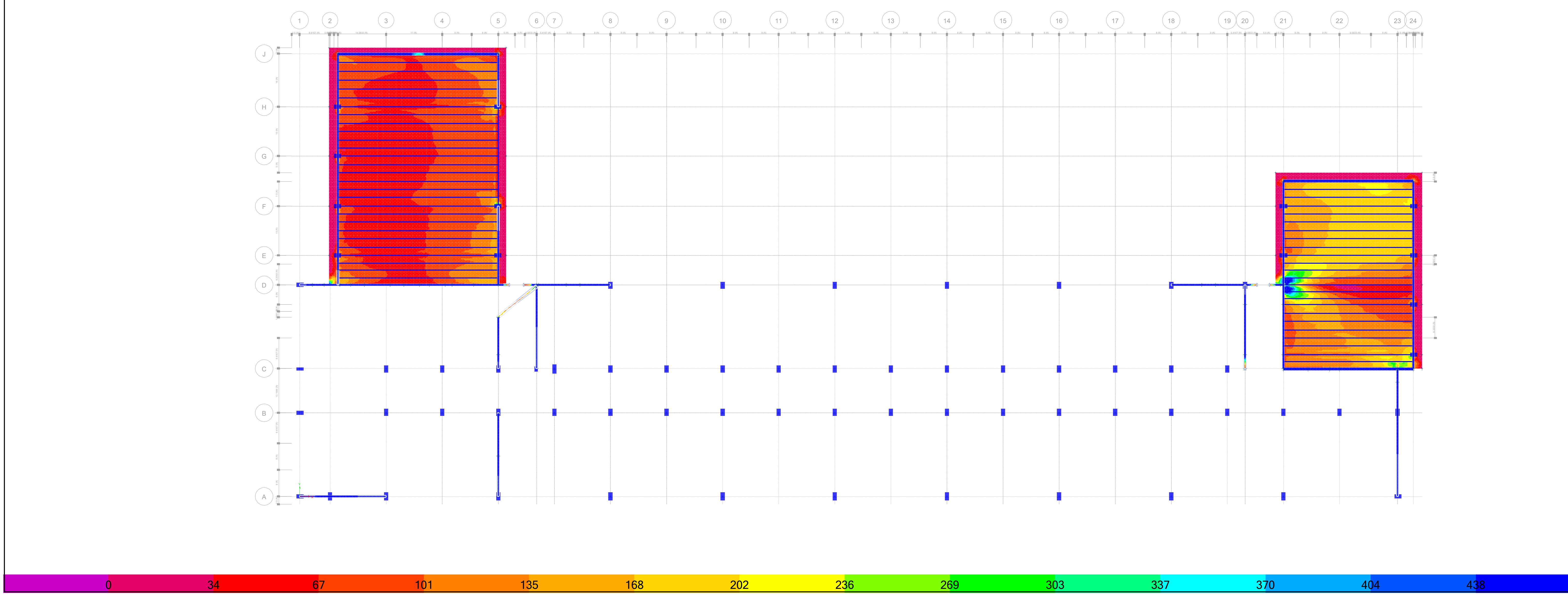


Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - ROOF - Z = 28.0833 (ft) Stress S12 Diagram Max (1E TRANS COMP) [lb/in²]

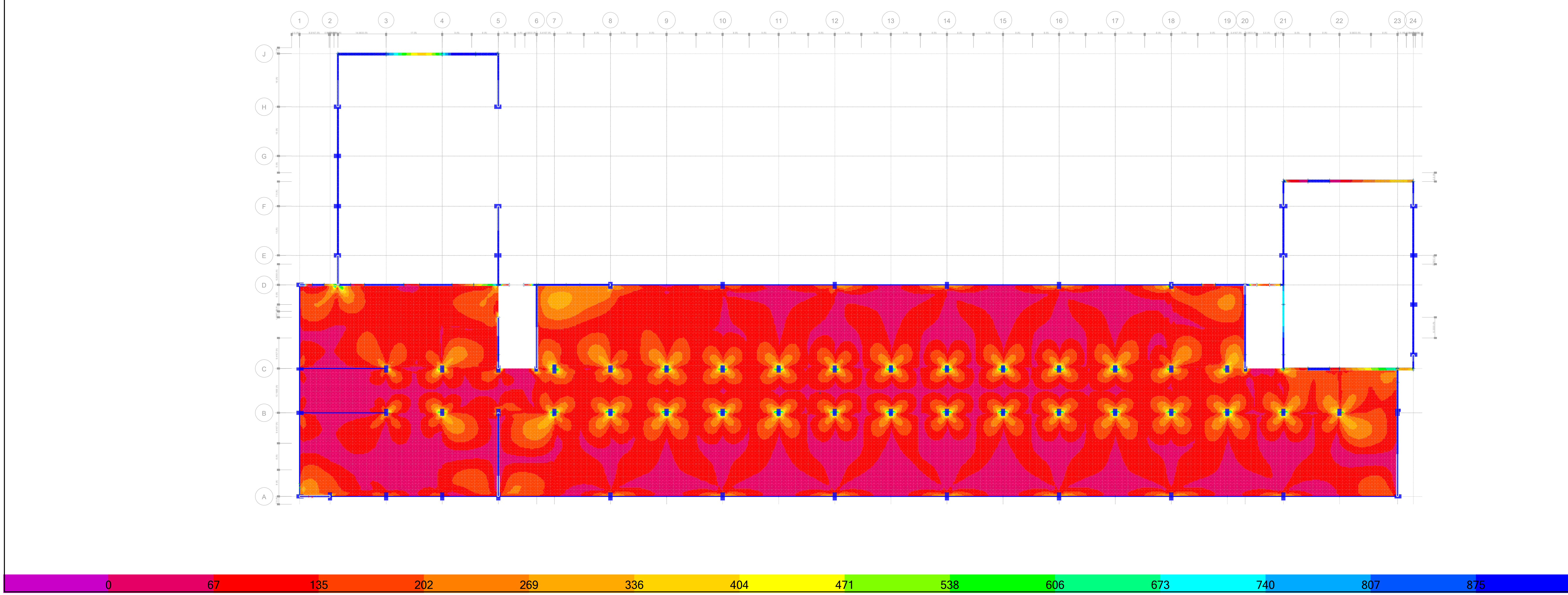


0	53	106	159	213	266	319	372	425	478	532	585	638	691

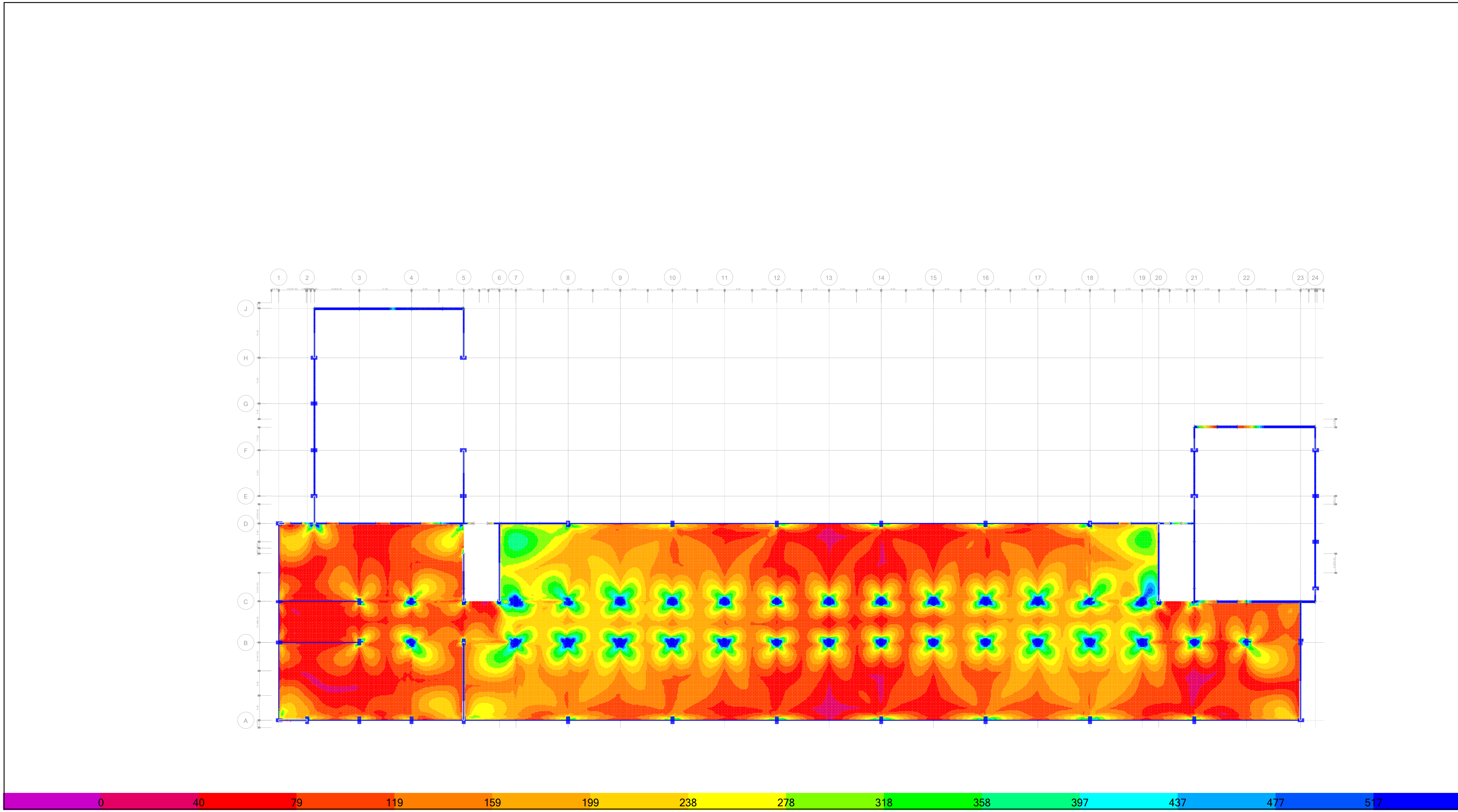
Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDBPlan View - MPR & ART ROOM - Z = 16.8333 (ft) Stress S12 Diagram Max (1E LONG COMP) [lb/in²]



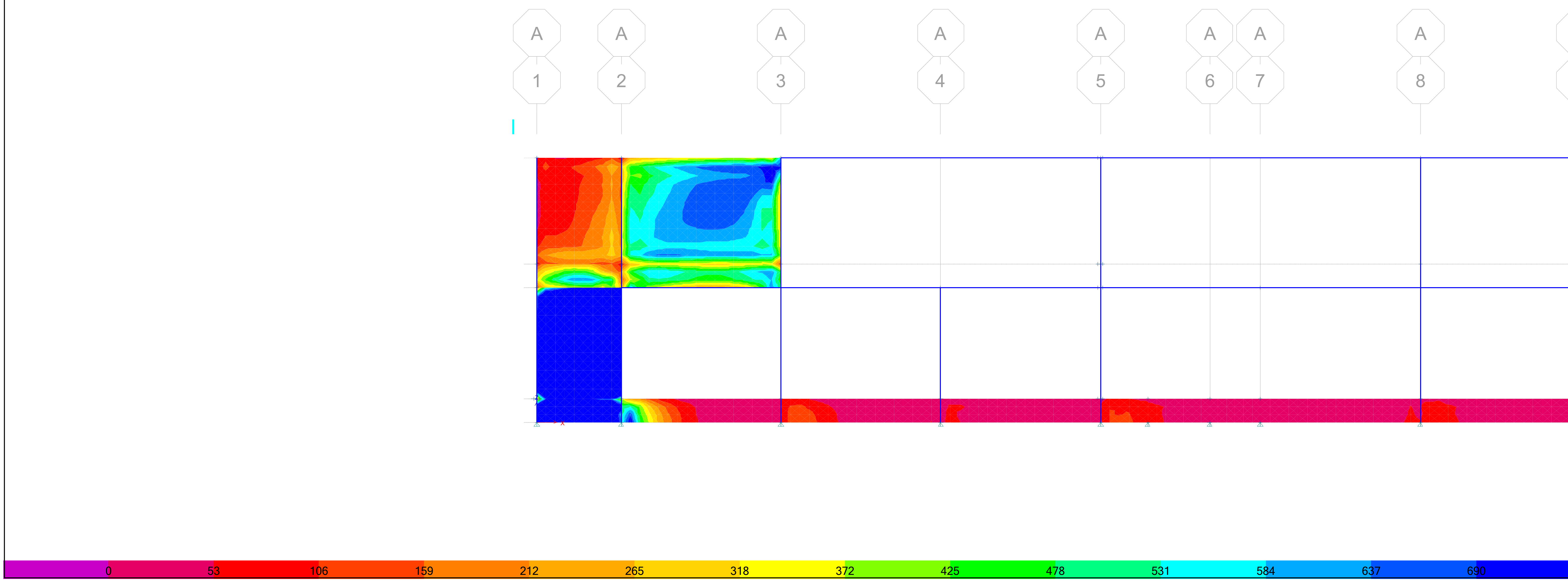
Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - MPR & ART ROOM - Z = 16.8333 (ft) Stress S12 Diagram Max (1E TRANS COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - SECOND FLOOR - Z = 14.3333 (ft) Stress S12 Diagram Max (1E LONG COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - SECOND FLOOR - Z = 14.3333 (ft) Stress S12 Diagram Max (1E TRANS COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Elevation View - A Stress S12 Diagram Max (1E LONG COMP) [lb/in²]

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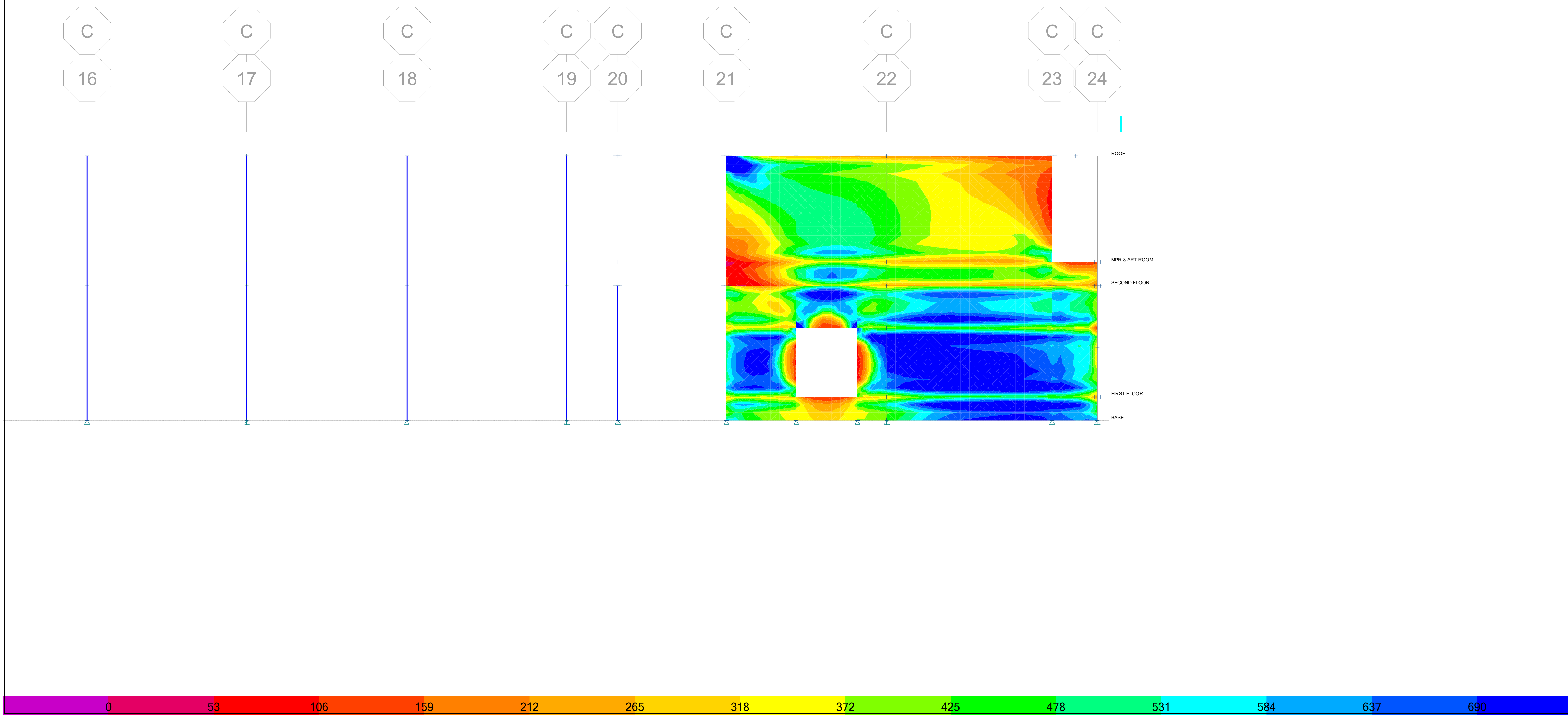
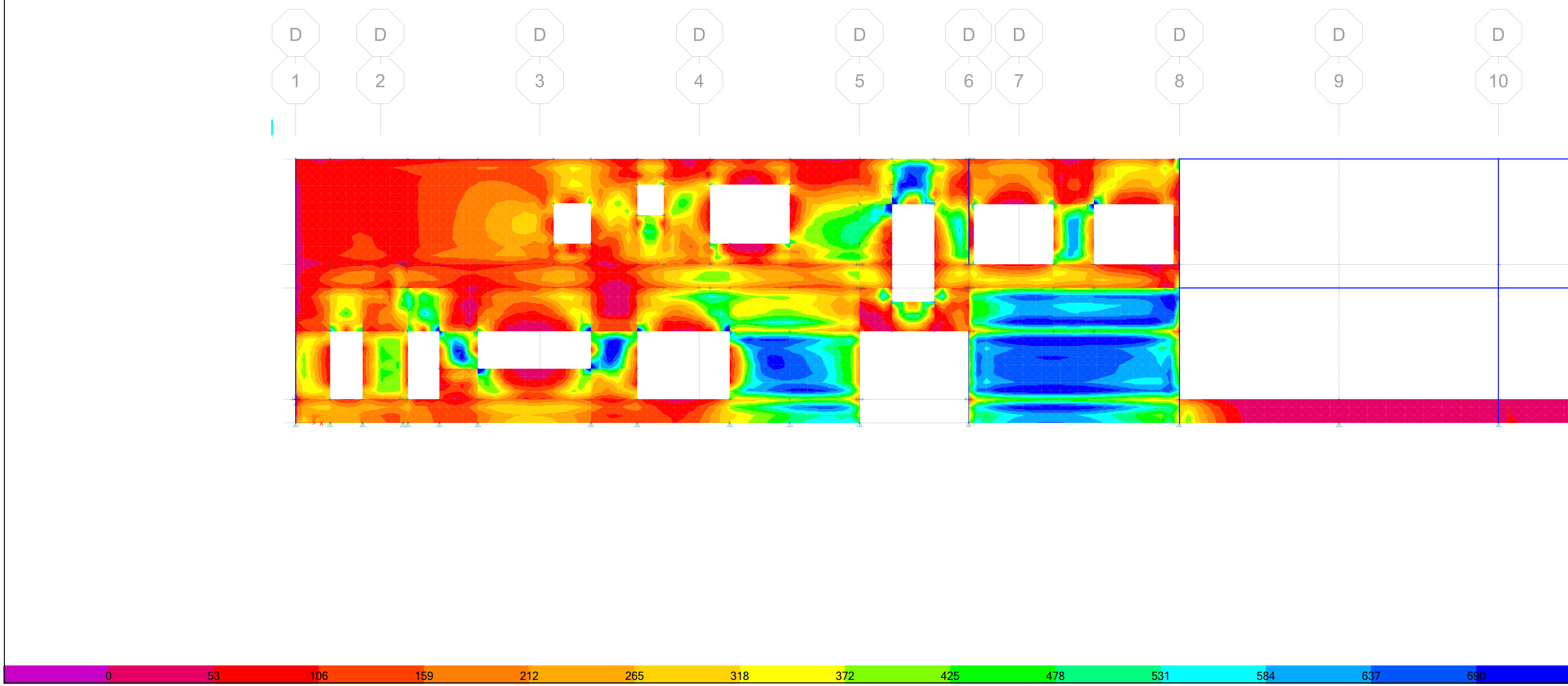
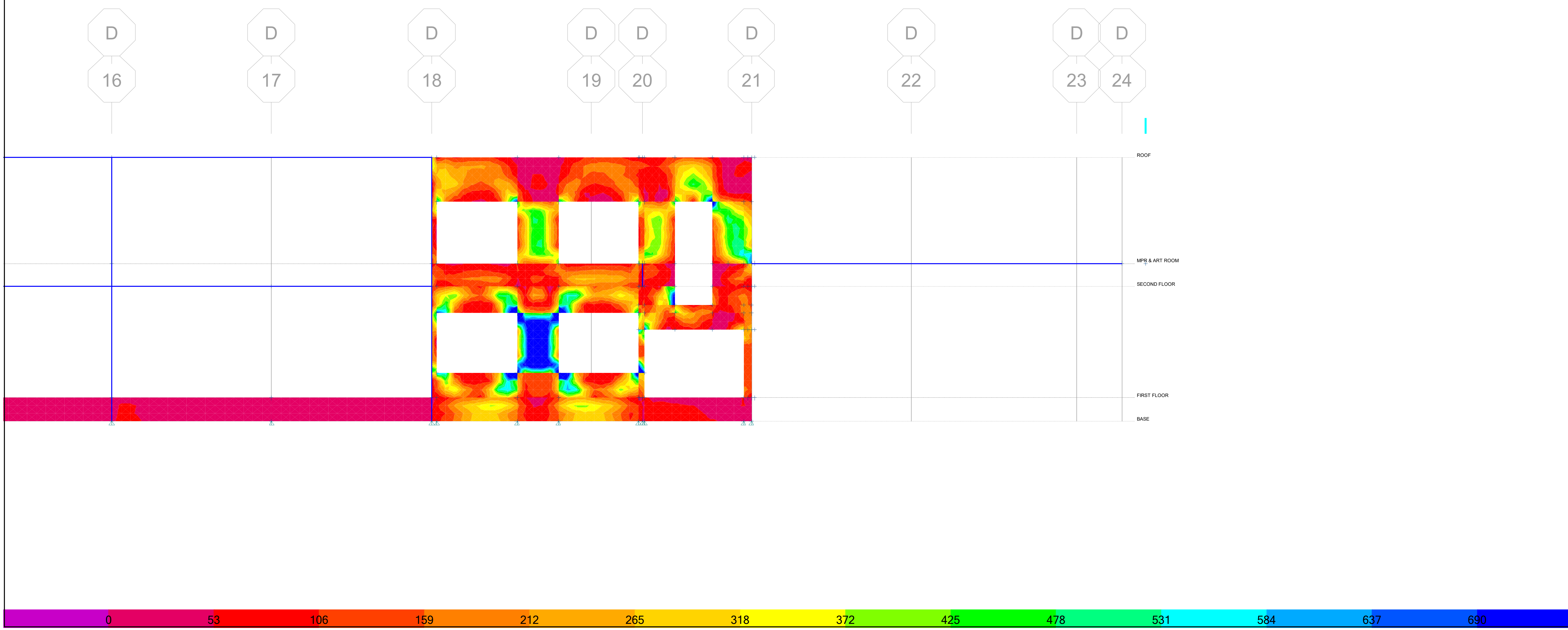


Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Elevation View - C Stress S12 Diagram Max (1E LONG COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Elevation View - D Stress S12 Diagram Max (1E LONG COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Elevation View - D Stress S12 Diagram Max (1E LONG COMP) [lb/in²]

APPENDIX D — BSE-2E STRESS MAPS

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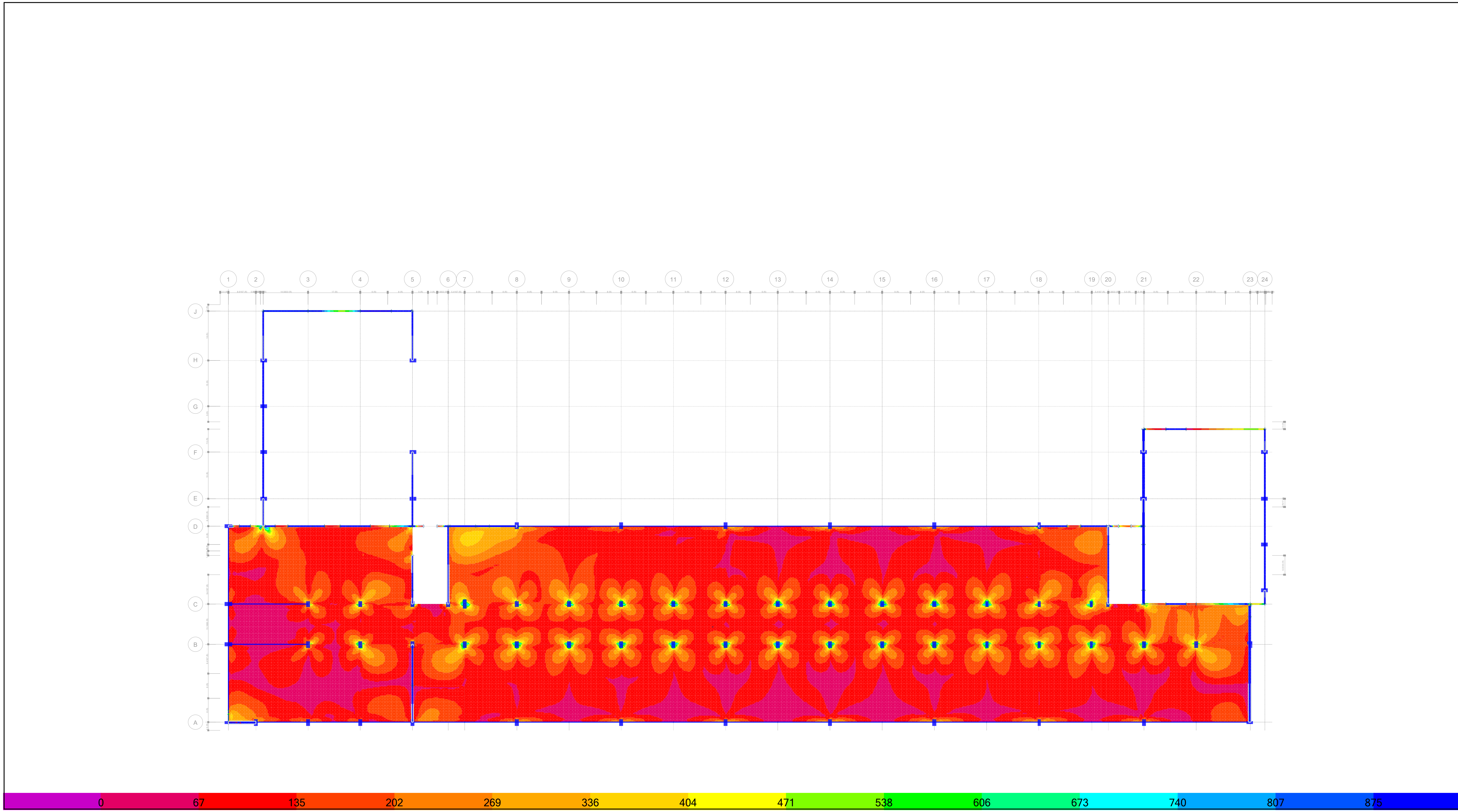
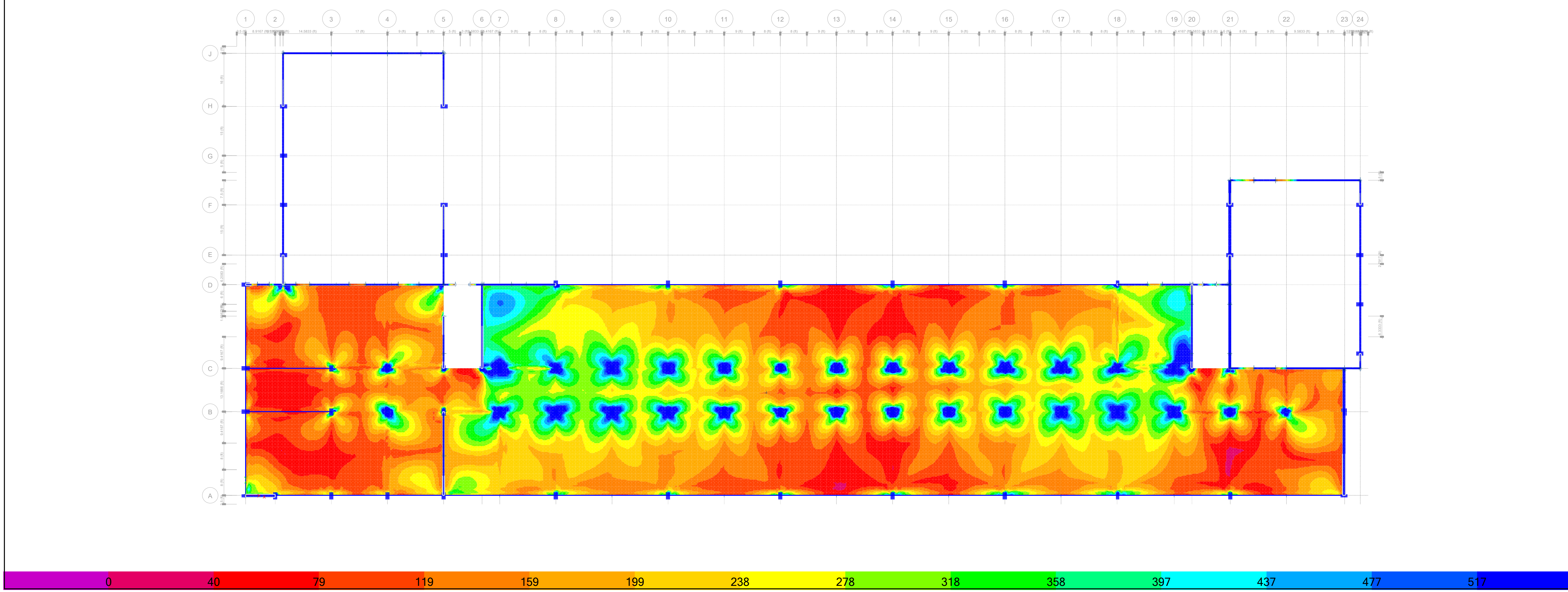
8/3/2022

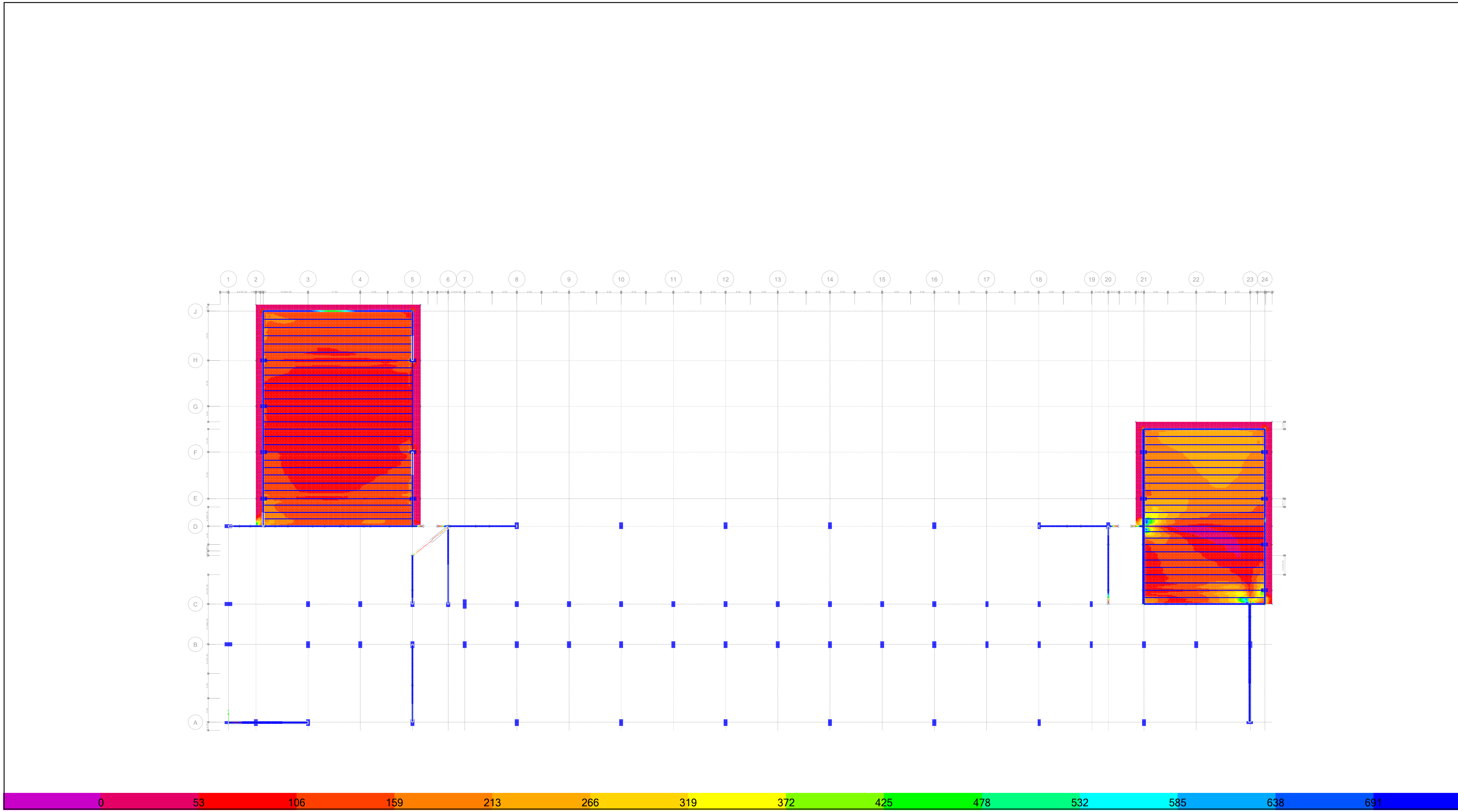


Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - SECOND FLOOR - Z = 14.3333 (ft) Stress S12 Diagram Max (2E LONG COMP) [lb/in²]

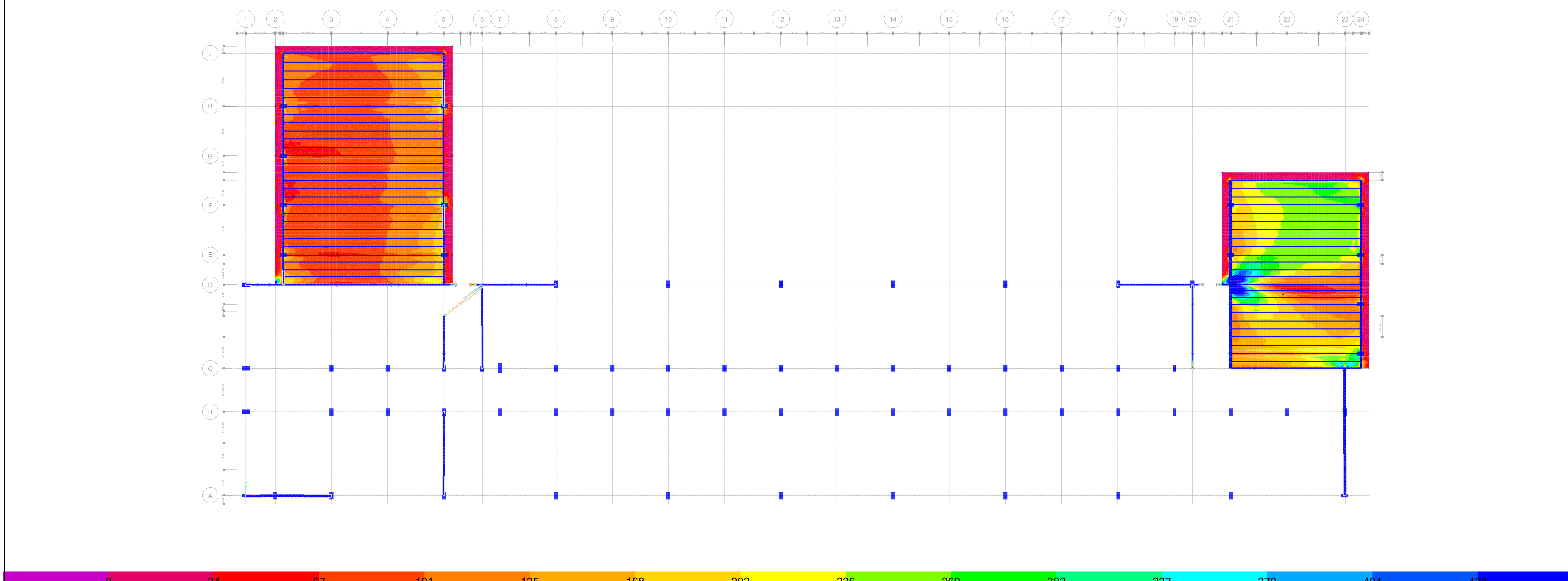
ETABS 17.0.1 8/3/2022



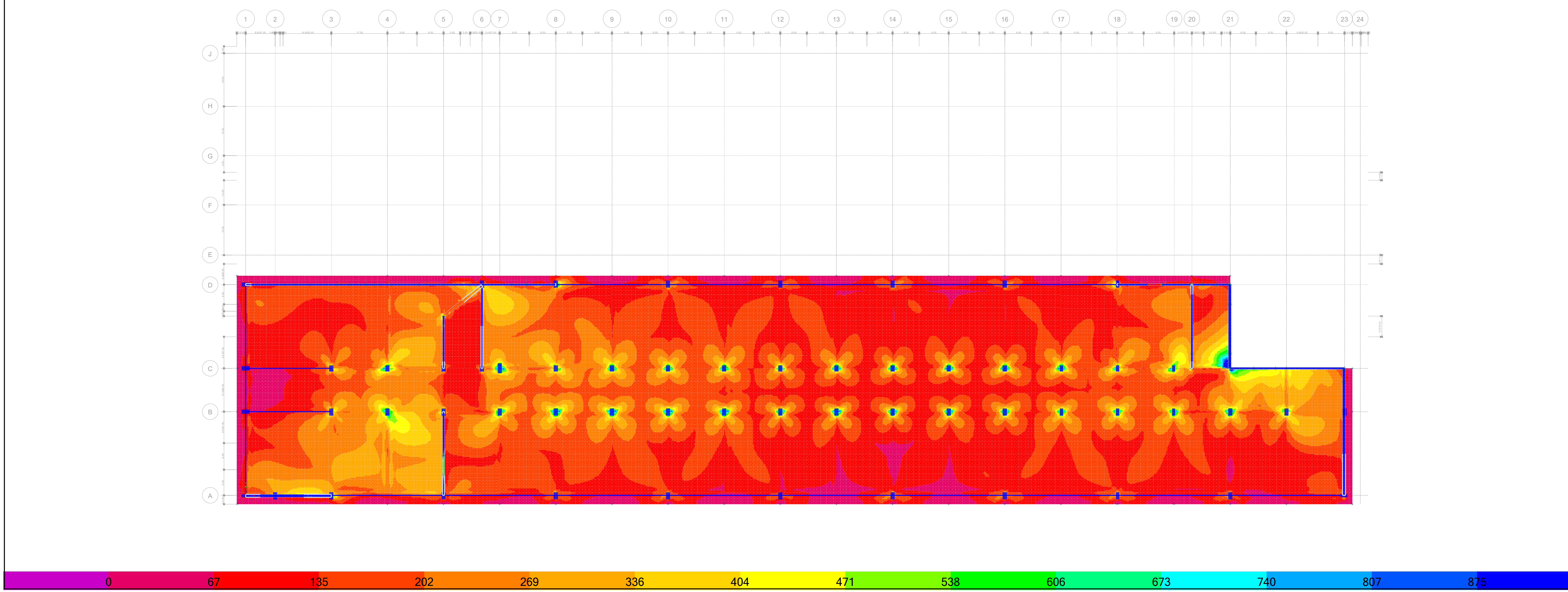
Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - SECOND FLOOR - Z = 14.3333 (ft) Stress S12 Diagram Max (2E TRANS COMP) [lb/in²]



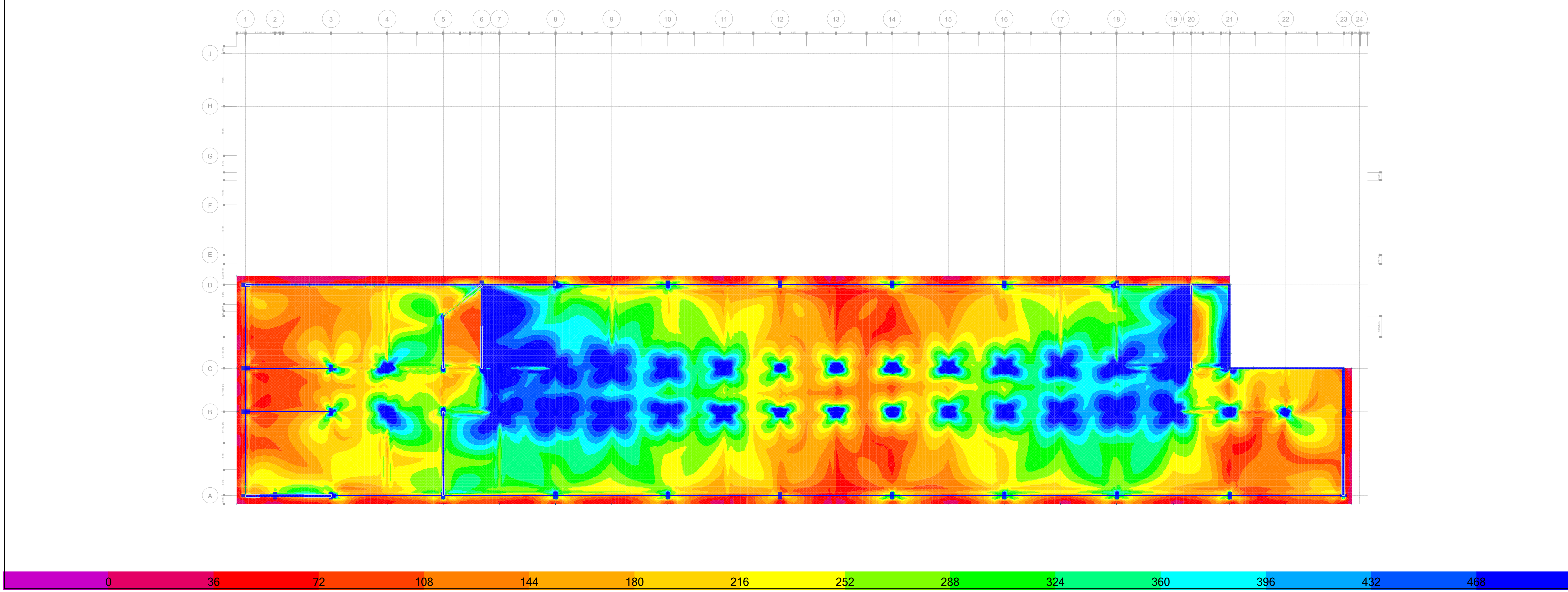
Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDBPlan View - MPR & ART ROOM - Z = 16.8333 (ft) Stress S12 Diagram Max (2E LONG COMP) [lb/in²]



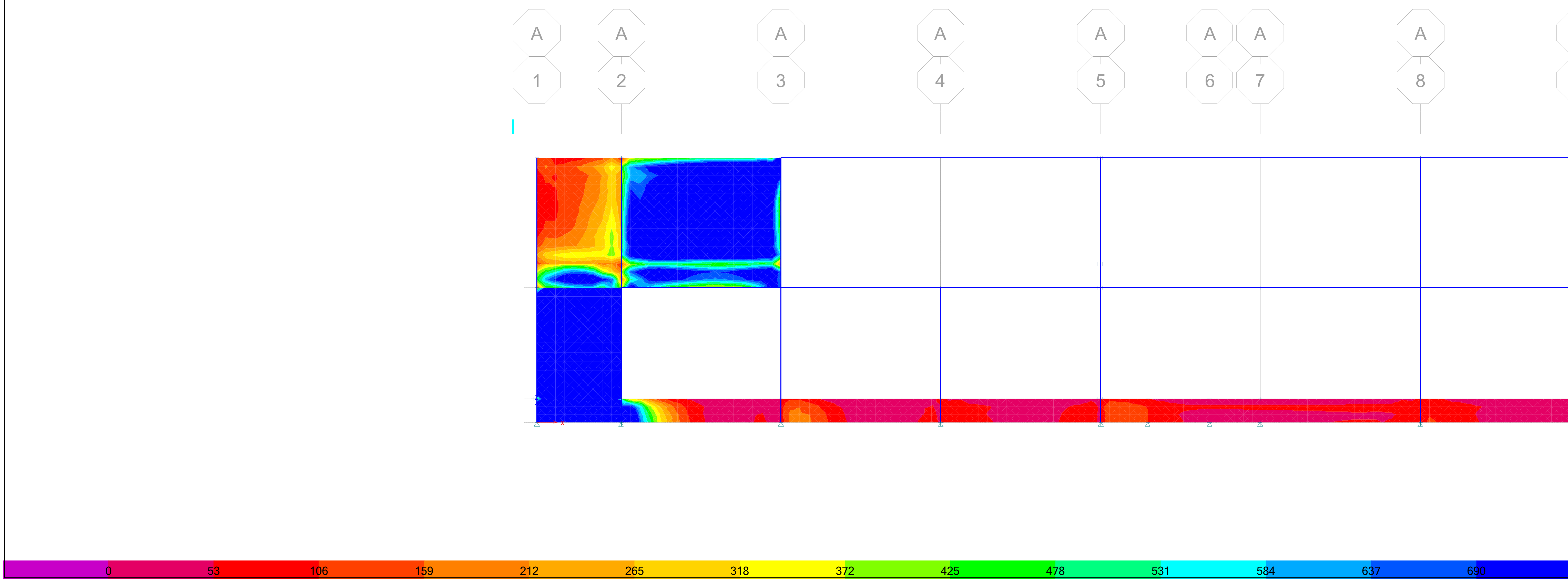
Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - MPR & ART ROOM - Z = 16.8333 (ft) Stress S12 Diagram Max (2E TRANS COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB   Plan View - ROOF - Z = 28.0833 (ft)   Stress S12 Diagram Max   (2E LONG COMP) [lb/in²]



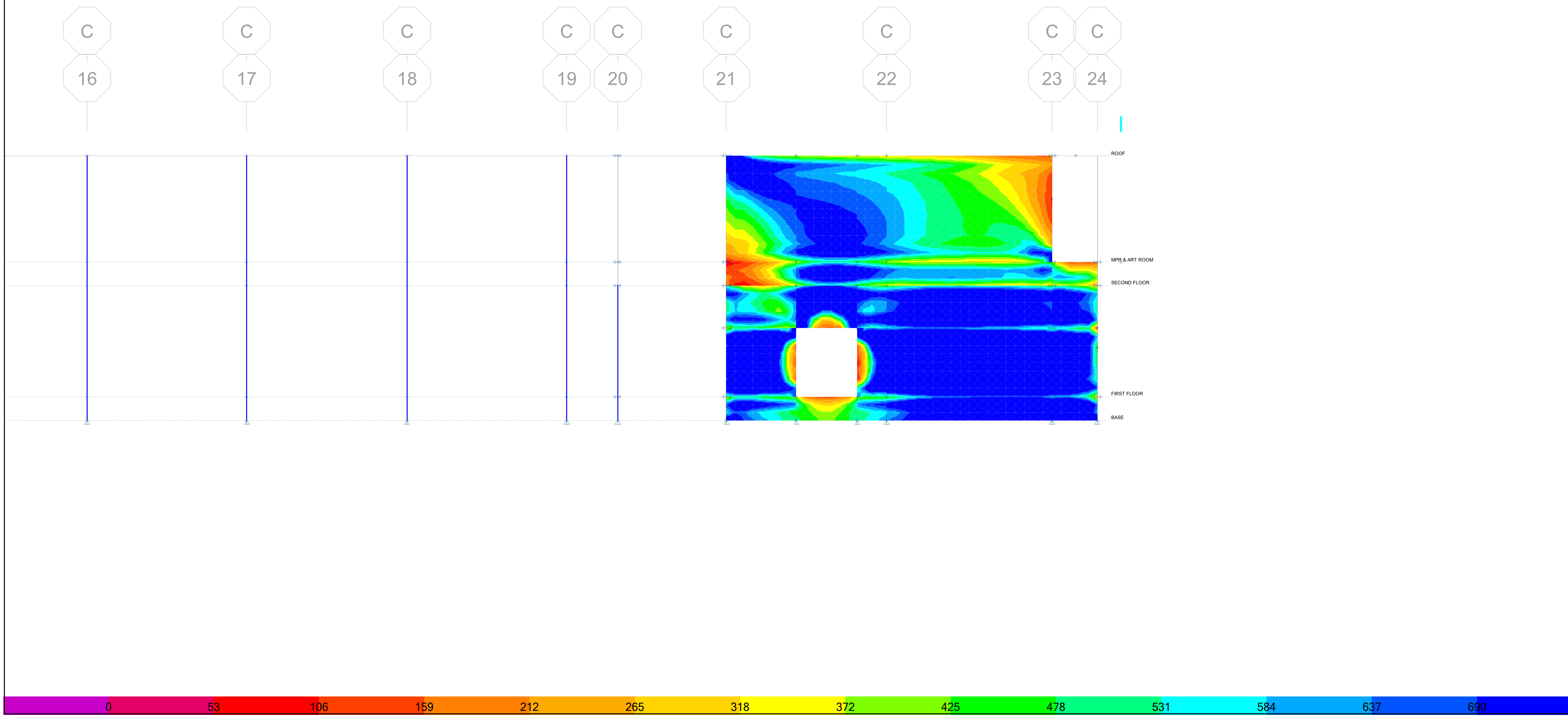
Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Plan View - ROOF - Z = 28.0833 (ft) Stress S12 Diagram Max (2E TRANS COMP) [lb/in²]



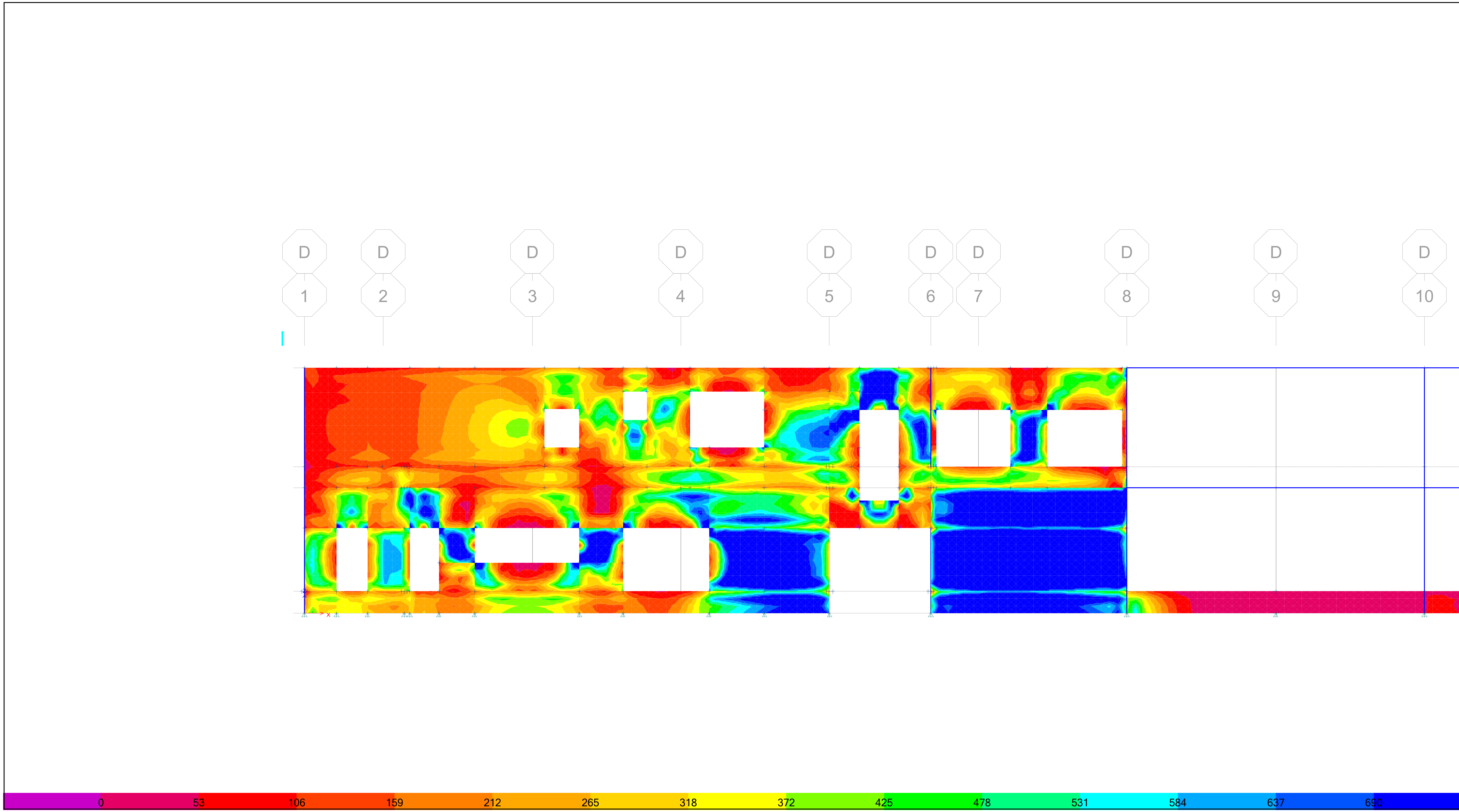
Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Elevation View - A Stress S12 Diagram Max (2E LONG COMP) [lb/in²]

ETABS 17.0.1

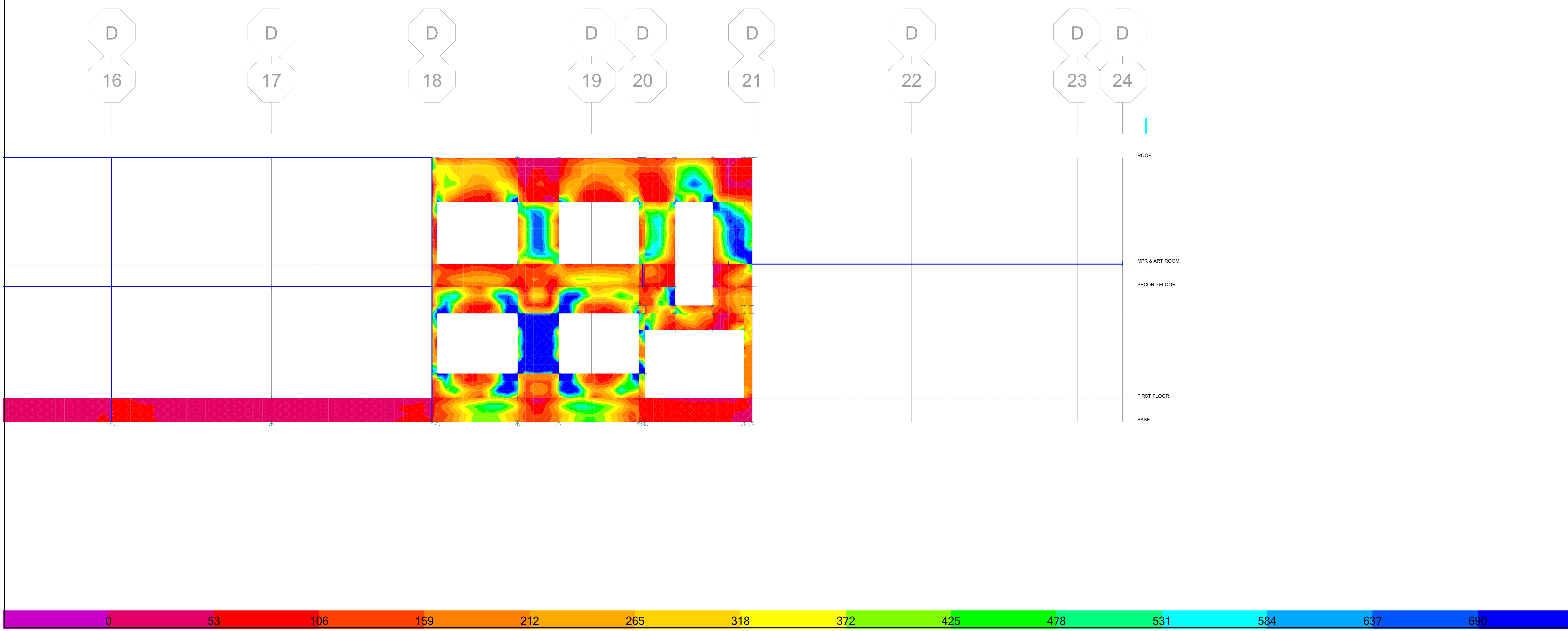
8/3/2022



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB   Elevation View - C   Stress S12 Diagram Max   (2E LONG COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Elevation View - D Stress S12 Diagram Max (2E LONG COMP) [lb/in²]



Ursa Major Orginal 1952 (GRAVITY COLUMNS 2.0) BSE-1E & BSE-2E.EDB Elevation View - D Stress S12 Diagram Max (2E LONG COMP) [lb/in²]