BSC	Design Calculation or Analysis Cover Sheet *Complete only applicable items.*	1. QA: QA 2. Page 1

3. System Initial Handling Facility	4. Document Identifier 51A-DBC-IH00-00200-000-00B
5. Title Initial Handling Facility (IHF) Foundation Design	
6. Group Civil/Structural/Architectural	
7. Document Status Designation ☐ Preliminary ☒ Committed ☐ Confirmed ☐ Cancelled/Superseded	

8. Notes/Comments

Rev 00A of this calculation supersedes 51A-SSC-IH00-00400-000-00A in its entirety.

List of Originators and Checkers for the contents:

Contents	**Originators**	**Checkers**
Body of the calculation	R. Chou, M. Lin & D. Croft	J. Solowey and D. Croft
Attachments A, C, E, I	R. Chou	J. Solowey
Attachment B, D	M. Lin	R. Chou
Attachment F	M. Lin / D. Croft	R. Chou / J. Solowey
Attachment G	R. Chou /D. Croft	J. Paredes/J. Solowey
Attachment H	J. Paredes	M. Lin
Attachment J	R. Chou	M. Lin
Attachment K	J. Paredes	J. Solowey

Attachments	Total Number of Pages
See Section 5	134+ 2 DVDs

RECORD OF REVISIONS

9. No.	10. Reason For Revision	11. Total # of Pgs.	12. Last Pg. #	13. Originator (Print/Sign/Date)	14. Checker (Print/Sign/Date)	15. EGS (Print/Sign/Date)	16. Approved/Accepted (Print/Sign/Date)
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00B ~~A~~ SM 2/20/08	This calculation is being revised as follows to address Condition Report (CR) #11723: - Total Revision of Attachments B, D, F & G - Revised pages 3, 5, 9, 10, 11, 12, 17, 19, 38, 40, 41, 48-55, 59-76 & 79-81, 77 SM 2/20/08 - Revised pages B-1, C-1, F-1, G-1	~~215~~ ~~214~~ ~~SFC~~ ~~02/20/08~~ 215 SFC 02/22/08	K-19	Ray Chou 2/20/08 Michael Lin 2/20/2008 Ray Chou 2/20/08 Dustin Croft 2/20/2008 Jason Paredes 2/20/2008	J. Solowey 2/20/2008 Dustin Croft 2/20/2008 Jason Paredes 2/20/2008 Michael Lin 2/20/2008 Ray Chou 2/20/08	Sal Macias 2/20/08	R. Rajagopal 2/20/08

DISCLAIMER

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ACRONYMS AND ABBREVIATIONS

Acronyms

BSC	Bechtel SAIC Company
DBGM-2	Design Basis Ground Motion 2 (2,000-year Return Period Seismic Event)
DL	Dead Load
D/C	Demand/Capacity Ratio
FEM	Finite Element Model
ID	Identification Designation
IHF	Initial Handling Facility
ITS	Important to Safety
ISRS	In-Structure Response Spectra
NRC	U.S. Nuclear Regulatory Commission
OTM	Overturning Moment
PDC	Project Design Criteria
QA	Quality Assurance
RSA	Response Spectrum Analysis
SADA	Seismic Analysis and Design Approach
SAP2000	SAP2000 Structural Analysis Software Version 9.1.4
SASSI	System for Analysis of Soil-Structure Interaction
SRSS	Square Root of the Sum of the Squares
SSC	Structures, Systems and Components
TOS	Top Of Steel
TOC	Top Of Concrete
ZPA	Zero Period Acceleration

Abbreviations

ft	Feet
ft^2	Square feet
in	Inch
kips	Kilo pounds
ksf	Kilo pound per square foot
min.	Minimum

ACRONYMS AND ABBREVIATIONS (Continued)

psf	Pound per square foot
psi	Pound per square inch
pcf	Pound per cubic foot
Ref.	Reference
Sec.	Section

1. PURPOSE

The primary purpose of this calculation is to provide the design for two individual mats (the large mat foundation and the small mat foundation) for the Initial Handling Facility (IHF). This calculation includes the following:

- Design of shear and flexural reinforcement for each of the foundation mats
- Stability evaluation of the foundation mats against overturning and sliding when subjected to seismic loads
- The allowable soil bearing pressure check for each of the foundation mats

In addition, this calculation includes a model of the IHF structures, which combines the individual steel structure, concrete structures and the foundation mat structure models, as a combined finite element model (FEM). This combined steel and concrete finite element model will be used for the Tier-2 analysis using SASSI (Ref. 2.2.23). The combined model also addresses a coupled model that includes the major cranes modeled along with the IHF steel structure.

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2.3 DESIGN CONSTRAINTS

None

2.4 DESIGN OUTPUTS

Results of this calculation will be used in developing the IHF concrete foundation drawings.

The combined model will be used in the Tier-2 SASSI (Reference 2.2.23) analysis.

3. ASSUMPTIONS

3.1 ASSUMPTIONS REQUIRING VERIFICATION

3.1.1 The IHF layouts are taken from the General Arrangement drawings (References 2.2.14 to 2.2.18). This calculation does not address local reinforcement or modifications needed to accommodate heavy equipment mounting or installation details. Such local modifications and/or additional reinforcing for the mat foundations shall be included during the detail design phase, based on design information provided by the equipment manufacturer(s).

Rationale–Pursuant to this current design of the overall foundation mats (consistent with Sections 6.4.3 and 6.4.6 of *Design Loads for the Steel and Concrete Structures*, Reference 2.2.21), the superimposed dead loads and live loads will provide adequate loading to account for most of the heavy equipment loads on the mats. In the final analysis, local modifications or additional reinforcement of the mat foundations may be necessary based on the Equipment Manufacturer’s design information. The currently identified major equipment includes, but is not limited to the items listed below. For a more detailed list of the equipment, see Attachment U of Reference 2.2.21.

For Small (West) Mat Foundation:

- Waste Package Position Room Shield Doors 1 and 2 (Type 3)
- Cask Unloading Room Shield Door (Type 1)

For Large (East) Mat Foundation:

- Waste Package Position Room Shield Doors 1 and 2 (Type 3)
- Waste Package Loadout Room Shield Door (Type 2)

This assumption is being tracked in CalcTrac.

Assumption 3.1.1 is used in Section 6.3.

3. 1.2 Floor Superimposed Live Load: Uniform 100 psf

Uniform superimposed live load of 100 psf will be assumed over both mats.

Rationale— The majority of live loads on the mat come from the routine heavy moving equipment for transferring a canister to a waste package, which have well designated traveling patterns. Therefore, a uniform live-load assumption of 100 psf is used. This superimposed floor live load will be sufficient to account for most moving equipment, cargo loads, or both, except when on any specific zone with higher moving equipment loads.

Areas with heavily loaded equipment exceeding 100 psf will be further analyzed during final design and reinforced as necessary.

This assumption is being tracked in CalcTrac.

Assumption 3.1.2 is used in Sections 4.3.2.1, 6.3 and SAP2000 models in Attachments B and C.

3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

3.2.1 Soil Spring of Strain compatible soil properties for the DBGM-2 lower bound 100 ft alluvium case (Reference 2.2.4) will be used as the controlling soil spring stiffness.

Rationale–The soil springs from lower bound 100 ft alluvium, DBGM-2 seismic load have the least stiffness (the smallest value), in the vertical direction and horizontal direction (Reference 2.2.4, Section 7.1). This will generate maximum bending moments and shear in the foundation mat due to greater deformation. Similarly, the horizontal soil value with the least stiffness will result in larger translational movements of the mat when subjected to earthquake forces.

Assumption 3.2.1 is used in Attachments B and C.

3.2.2 Stress contour plots generated by SAP2000 (Reference 2.2.19) using nodal averaging will be used in the design of the required reinforcing steel.

Rationale–Reinforced concrete is a composite material comprised of concrete and embedded reinforcing bars. When peak element forces exceed the average values shown on the contour plots (Attachments D & E) it is recognized that, as concrete cracks and as reinforcing bars yield, peak resultants are redistributed over adjacent elements. Utilizing force resultants based on nodal averaging accounts for the redistribution and is appropriate for use in reinforced concrete design.

Assumption 3.2.2 is used in Attachments D and E.

3.2.3 10% of estimated dead loads from the uniform platform dead load are included in the dead load (DL) load application.

Rationale–In addition to the standard dead load of structural steel and concrete, uniform platform dead load has been applied to the large mat. Ten percent of this dead load is considered a permanently attached load on the platforms. Included in this uniform load is HVAC equipment, platform steel and miscellaneous equipment supports. The load combinations which address 0.9DL, includes 0.9 of the weights of structural steel and concrete, as well as 10% of the uniform platform DL stated above. Attachment U of Reference 2.2.21 also includes the list of heavy equipment locations and weight information.

Assumption 3.2.2 is used in Section 6.3.

4. METHODOLOGY

4.1 QUALITY ASSURANCE

This calculation was prepared in accordance with EG-PRO-3DP-G04B-00037 (Reference 2.1.1). Section 3.1.2 of the *Basis of Design for the TAD Canister–Based Repository Design Concept* (Reference 2.2.2) classifies the IHF structure as ITS. Therefore, the approved record version of this calculation has a record designation of QA: QA.

4.2 USE OF SOFTWARE

The commercially available Microsoft Office Excel 2003 spreadsheet code, which is a component of Microsoft Office 2003 Professional, is used to perform computations in the Tables and graphing of the Figures shown in Sections 6 and 7 and Attachment I. Computation formulas and data are manually input in Excel 2003 based on the geometry of the SAP2000 model and the soil spring properties (Reference 2.2.4). The computation results were verified by checks using hand calculations. Figures are checked though visual inspection. The commercially available Microsoft Office Word 2003, which is also a component of Microsoft Office 2003 Professional, was used to compose this calculation report. Usage of Microsoft Office 2003 Professional in this calculation constitutes Level 2 software usage, as defined in IT-PRO-0011 (Ref. 2.1.2). Microsoft Office 2003 Professional (STN 610236-2003-SPISP2-00) is listed in the current Level 2 Usage Controlled Software Report as defined in IT-PRO-0011 (Ref. 2.1.2). Microsoft Office Excel 2003 (11.8105.8108 Service Pack 2) and Word ((11.8106.8107 Service Pack 2) were executed on a PC running the Microsoft Windows XP Professional (5.1.2600 Service Pack 2 Build. 2600) operating system. The calculation process and equations are documented in Section 6 for checking by manual calculation.

The structural engineering software program SAP2000, Version 9.1.4, (STN 11198-9.1.4-00) (Ref. 2.2.19) is used in this calculation to perform the non-linear static analysis of the mat foundation models. The SAP2000 Version 9.1.4 is classified as Level 1 software usage as defined in IT-PRO-0011 (Reference 2.1.2). SAP2000 input was based on manual input. SAP2000 output stress contours (Attachments D & E) are used to design the concrete mat foundations (Section 6.5 of this calculation) and link element force outputs (Attachments B & C) are used in checking the soil bearing pressure (Section 6.7 of this calculation). The software program was run on a PC with Windows XP operating platform. The SAP2000 evaluation performed for this calculation is fully within the range of the validation performed for SAP2000 (Reference 2.2.11).

4.3 DESIGN METHODOLOGY

4.3.1 Seismic loading

Two seismic analysis methods are given in the NRC Regulatory Guide 1.92 (Reference 2.2.24, page 7), the *SRSS combination* and its alternative, the *100-40-40 percent combination*. The SRSS approach cannot fully address the local uplifting movement of these mats (as the result of seismic overturning), which results in losing local support of the soil. Therefore, the *100-40-40 percent combination* method is used in this calculation. As stated in the NRC Regulatory Guide

1.92 (Reference 2.2.24, page 7), "the *100-40-40 percent combination* alternative produces higher estimates of maximum response than the *SRSS combination* method by as much as 16 percent, while the maximum under-prediction is 1 percent". Therefore, the *100-40-40 percent combination method* used in this calculation is conservative. For a more detailed description see Section 6.3.

4.3.2 Structural Analysis

There are three components for the analysis of each mat:

Structural analysis and design of the reinforced concrete mat
Stability of the mat against overturning and sliding
Bearing pressure check

4.3.2.1 Structural Analysis and Design of the Reinforced Concrete Mat

Structural analyses of these mats have two sources of loading:

1) External Loads

The external loading from the above supporting structures is composed of both static and seismic loads. The seismic loads are generated from *Response Spectrum Analysis* (RSA). The acceleration response generated from the RSA will be converted to an equivalent static loading by multiplying the earthquake acceleration and the mass at each joint. The external reaction forces from the *Equivalent Static Analysis* will be placed on the foundation mat (at junctions of the mat foundation and the columns or shear walls) as static loads.

The steel structure's contributory dead loads, live loads, and seismic loads are from *Initial Handling Facility (IHF) Steel Structure Seismic Analysis and Steel Member Design* (Reference 2.2.3). The concrete structure's dead load, live load, and seismic loads are from *Initial Handling Facility (IHF) Concrete Structure Design* (Reference 2.2.5). These loads are applied to the finite element model as individual joint loads. These loads will be further combined by following Section 4.2.11.4.5 of Reference 2.2.1.

2) Superimposed Loads on Mat

The superimposed loads on the mat foundation are:

Uniform Superimposed Dead Load: 140 psf (Reference 2.2.21)

Uniform Superimposed Live Load: 100 psf (Assumption 3.1.2)

Equivalent Static Structural Analysis

A SAP2000 static analysis will be used in analyzing the small and large mats in accordance with SADA section 7.2.7.2 (Reference 2.2.12), by *100-40-40 component factor method*. Non-linear (compression-only) links are placed under the mat to simulate the soil support.

Enveloping Bending Moment and Shear Forces

All load combinations are enveloped in order to obtain maximum bending moments and shear forces in all elements.

Structural Design

Enveloped bending moment and shear force output from the above *Equivalent Static Structural Analysis*, will be used to design the reinforced concrete by following ACI 349 (Reference 2.2.6) as specified by section 8.2 of Reference 2.2.12. The resulting flexural and shear Demand/Capacity ratios are calculated.

SAP2000 (Reference 2.2.19) is utilized to generate moment and shear contour plots, and these contour plots are used in designing the shear and flexural reinforcing in the foundation mat. In designing the flexural reinforcing, a standard pattern is selected and the corresponding moment capacity resulting from the placement of the reinforcement is computed. The contour plots will then be utilized to identify areas that may require additional reinforcing above the standard pattern. The shear capacity of the concrete, without any reinforcing considerations, is first computed. Shear contours are utilized to determine areas of the foundation mat requiring shear reinforcing. Shear reinforcing is then designed to provide additional shear capacity. Both shear and flexural capacities are determined in accordance with ACI-349 (Reference 2.2.6).

FEM Equivalent Static Model

Two SAP2000 (Reference 2.2.19) finite element models (FEM) of the IHF foundation are developed as follows:

Small Mat FEM – Small (west) foundation mat supporting the external concrete shear wall structure (as shown in Attachment H, Figures H-1 to H-5) based on the General Arrangement drawing (Reference 2.2.14).

Large Mat FEM –Large (east) foundation mat supporting the steel structure and its internal concrete structure (References 2.2.14 and 2.2.18) as shown in Figures H-1 to H-5 of Attachment H. The large steel portion extends from gridlines 4 to 10 as shown in IHF General Arrangement drawings (References 2.2.14 to 2.2.18).

Since a non-linear (compression-only) spring element is utilized to model soil stiffness, a non-linear analysis is required for each loading combination. Using SAP2000 (Reference 2.2.19) a solution for each load combination is obtained that verifies the spring elements are in compression. SAP2000 automatically removes those springs in tension and re-iterates the analysis.

Additional details of the finite element analysis for the foundation mat and the stability calculations are discussed in Section 6.

4.3.2.2 Sliding and Overturning

Sliding: The overall stability of the IHF structures against sliding is evaluated. Due to high seismic accelerations associated with 2000-year return period Design Basis Ground Motion-2 (DBGM-2), the energy balance method discussed in ASCE 43-05 (Reference 2.2.8) is used to compute the maximum anticipated sliding displacement for both small and large mat foundations. Any umbilical utility piping, electrical raceway, or other system components connecting from or to the IHF from outside the structure will need to be designed to accommodate the sliding displacement with a suitable safety factor. Twenty-five percent of the live load is included in the foundation mat stability evaluation, which is consistent with the live load earthquake contributing masses used in the seismic analysis.

Overturning: The overturning calculations are performed using two different approaches, A and B:

Approach A: Consist of the following steps shown below:

- Calculate the story shear
- Calculate overturning moment (OTM) by multiplying the story shear with the moment arm with respect to the base.
- Calculate resisting moment (RM) by multiplying the weight of the structure with the offset from the center of mat to the edge of overturning.
- Factor of Safety against Overturning = $\frac{\text{RM}}{\text{OTM}}$

Approach B: Calculation is performed by SAP2000 FEM using 100-40-40 combination. The computed overturning moment (OTM) and the resisting moment (RM) are described in section 6.6.5. The Factor of Safety is the ratio of RM to OTM.

Approach A is used for both small mat and large mat. Approach B is used only for the large mat.

4.3.2.3 Combined model for SASSI input

A combined model (including the small mat, exterior concrete superstructure, the large mat, the steel superstructure and interior concrete superstructure as shown in Attachment H) is created for SASSI (Reference 2.2.23) analysis.

5. LIST OF ATTACHMENTS

6. BODY OF CALCULATION

6.1 FOUNDATION MAT FINITE ELEMENT MODEL

Small and large IHF foundation mats were modeled using SAP2000 (Reference 2.2.19). The two mats consist of shell elements with a nominal mesh size of 5 ft. by 5 ft. Actual mesh sizes vary from this nominal size as required to maintain the correct location of the shear walls. The origin and orientation of the global axes are shown in Attachment A.

When considering the stiffness properties of the soil beneath the mat foundation, a series of non-linear (compression-only) springs were used. The global soil spring stiffness is computed using the 100 ft lower-bound soil springs from the *Initial Handling Facility (IHF) Soil Springs and Damping* (Reference 2.2.4). A series of local soil springs with three translational directions were derived from the global soil spring stiffness and modeled under each joint of the foundation mat mesh. The method to derive these local springs is discussed in the *Seismic Analysis and Design Approach Document* (Reference 2.2.12, Section C1). Details of the soil spring calculation are given in Section 6.2.

In order to adequately simulate the local uplifting of the base mat, the vertical soil support is modeled as a compression-only spring at each joint of the base mat. These compression-only springs are modeled in SAP2000 (Reference 2.2.19) as "two-joint" type non-linear link elements. These non-linear links have full compression soil spring stiffness and zero tension stiffness when the gap of the link element is in the "open" stage. In order to minimize axial orientation change of the link elements in tension, a negative 10 ft vertical offset (i.e., -Z direction) has been used for anchoring the joint of the link element. The anchoring joint identification designations (ID) of the link elements are labeled with corresponding mat joint IDs with a prefix letter 'J' (e.g., J2000 for mat joint 2000).

The joint labeling scheme, where the structural steel columns and the concrete walls are attached to the mat is different. The joint IDs where the steel column attaches to the mat are labeled s1, s2, s3…sn, for example. Where the concrete walls are anchored to the mat, the joints are labeled c1,c2 c3…cn, for example. The joint ID of the mat will be manually matched to its corresponding joint ID of the super-structures through common coordinates.

Linear horizontal springs were used to model the lateral soil resistance. The primary load path to resist lateral loads is friction under the foundation mat. Therefore, each joint has an equal share of the total mat friction resistance. Hence, the lateral horizontal soil springs, which are placed at each joint of the basemat, will have identical spring stiffness.

Concrete material properties used in this finite element model are from the *Project Design Criteria Document* (Reference 2.2.1, Section 4.2.11.6.6).

SAP2000 (Reference 2.2.19) model files are included in Attachments B and C.

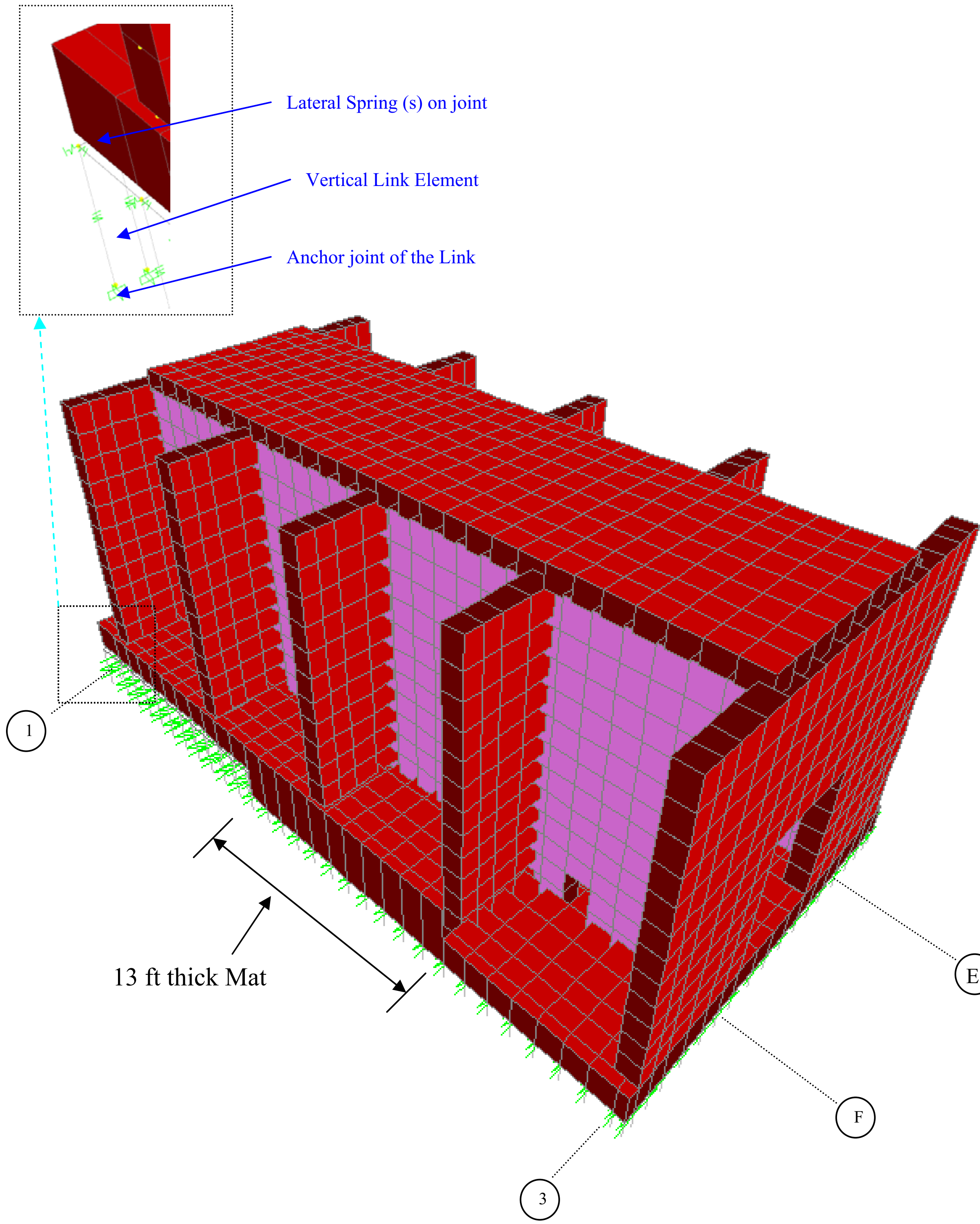


Figure 6.1.1 Isometric View of the Model of Small Mat Foundation

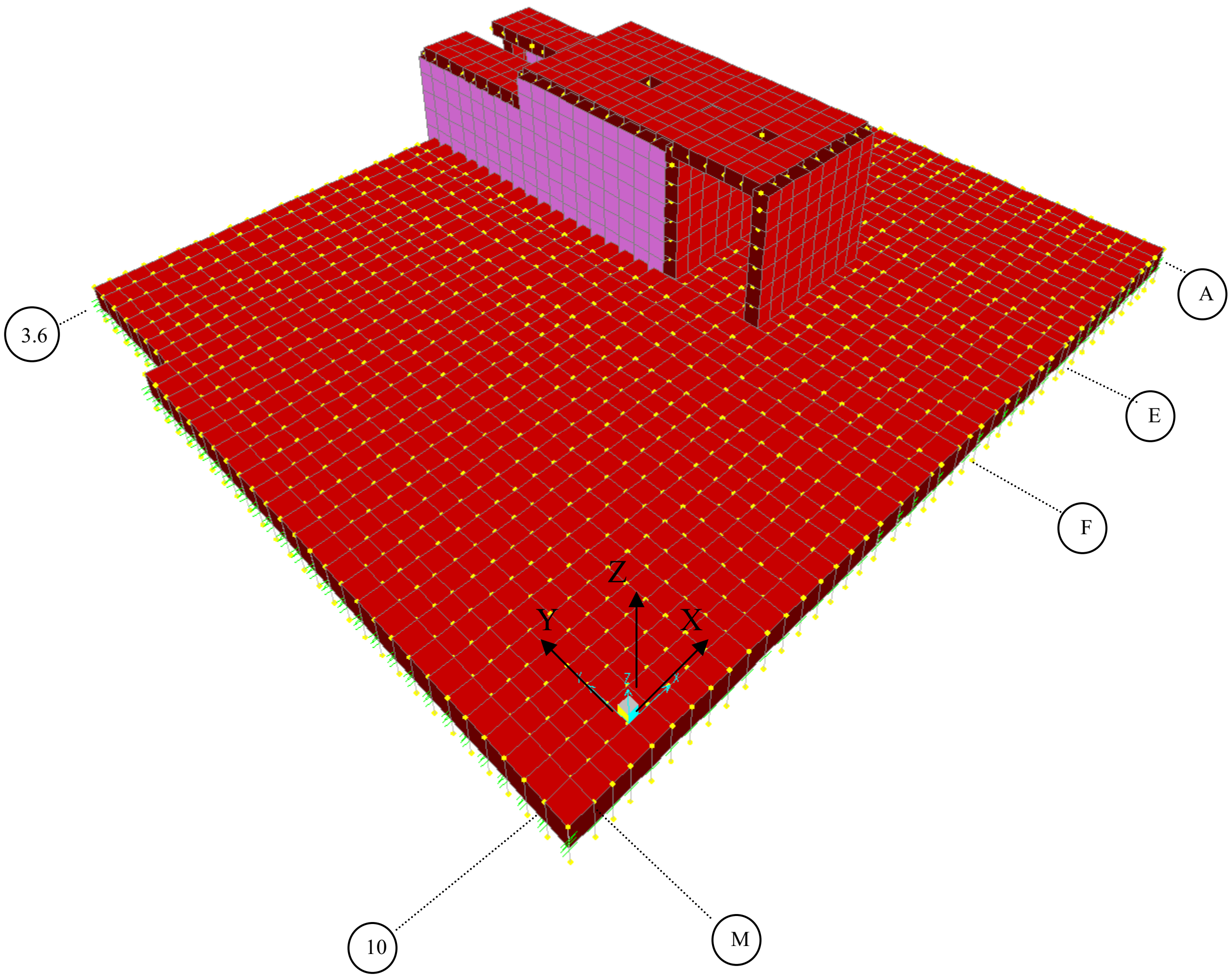


Figure 6.1.2 Isometric View of the Model of Large Mat Foundation

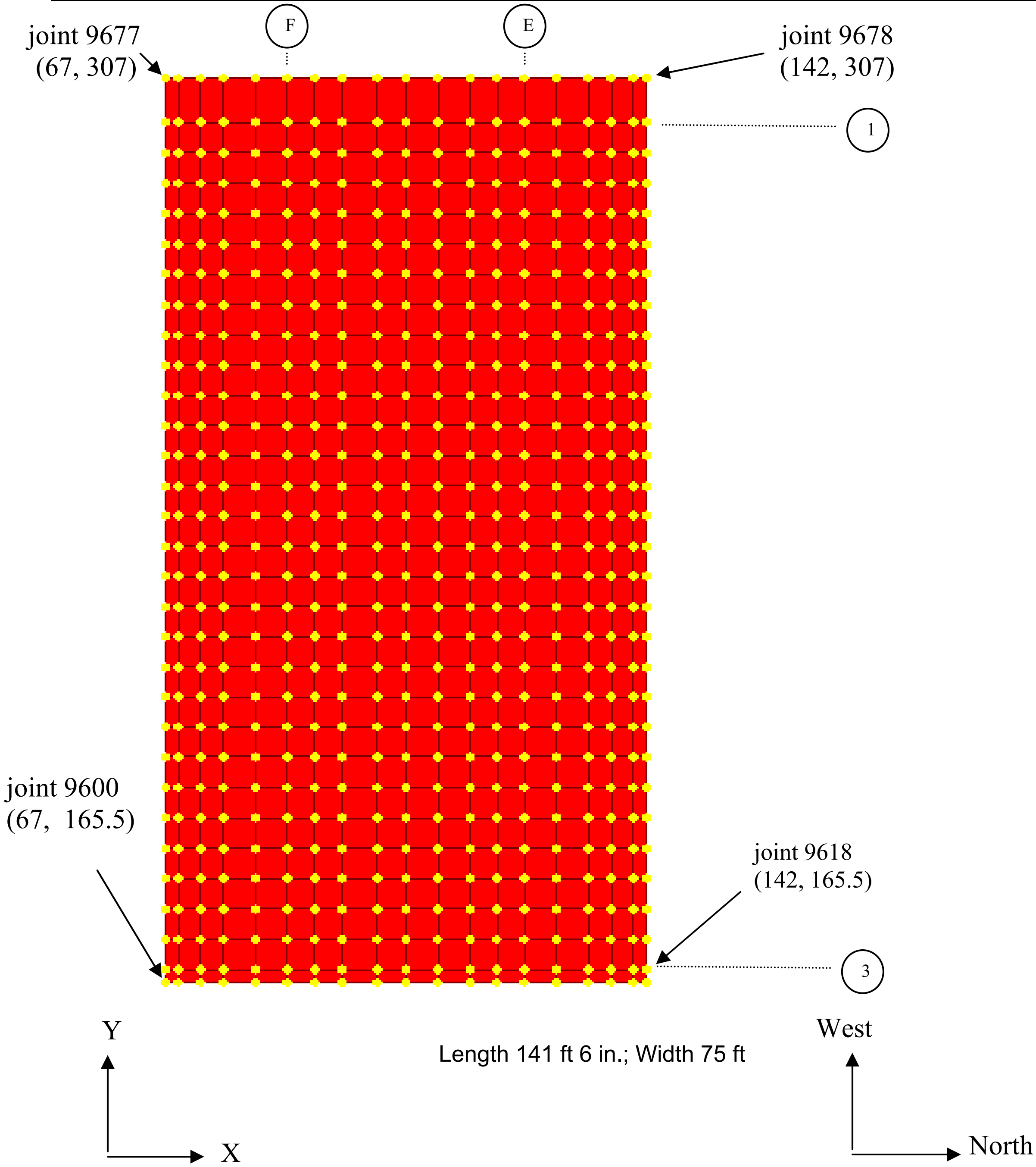


Figure 6.1.3 Foundation Mat Finite Element Mesh for the Small Mat Foundation

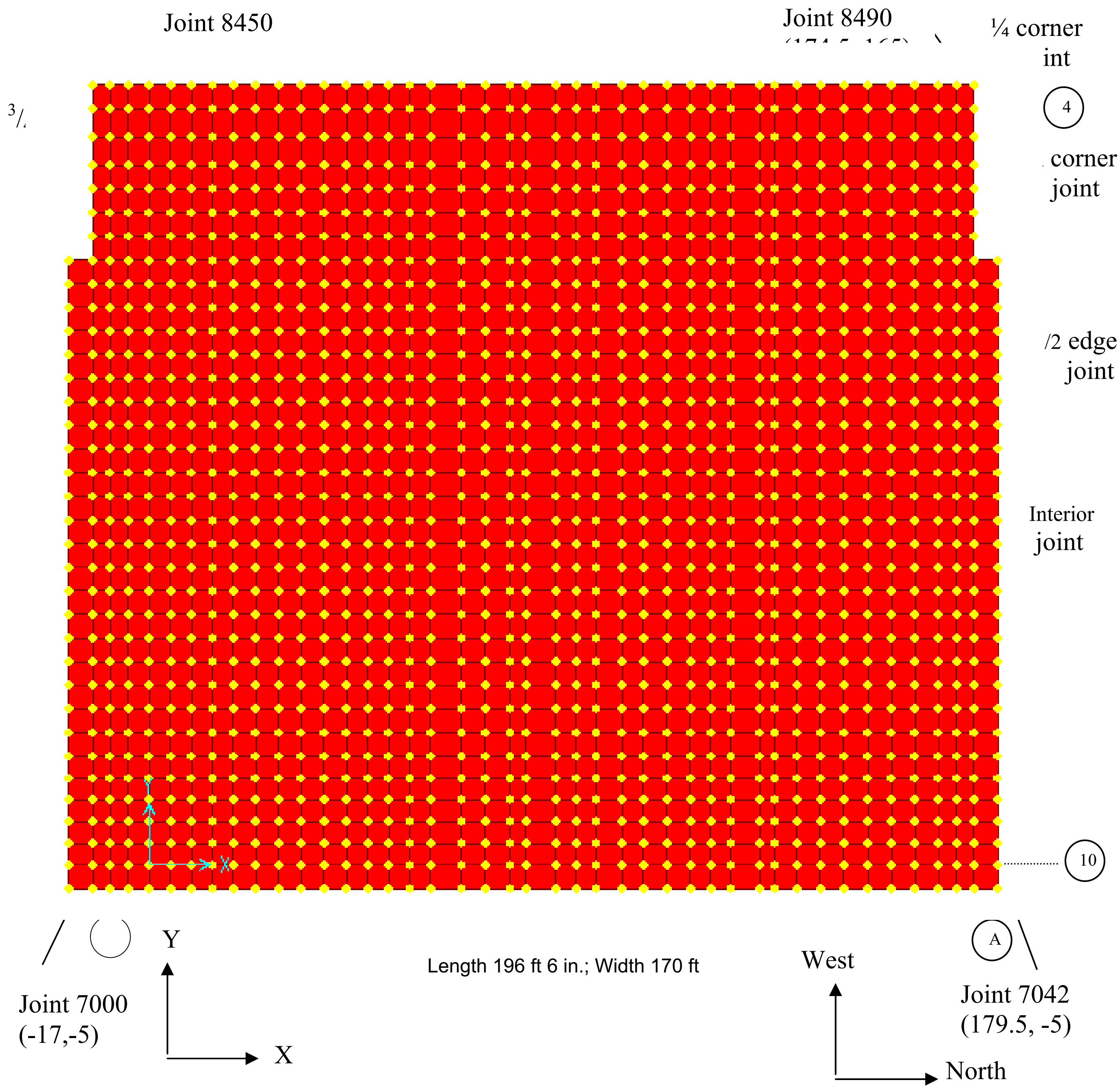


Figure 6.1.4 Foundation Mat Finite Element Mesh for the Large Mat Foundation

6.2 SOIL SPRINGS

As stated in Section 6.1, the boundary conditions for the mat foundation are modeled using non-linear (compression-only) springs based on the DBGM-2 (5E-4) lower bound 100 ft alluvium case (Reference 2.2.4) for the vertical springs, as well as for the linear springs in the horizontal direction. The soil springs calculated in *Initial Handling Facility (IHF) Soil Springs and Damping* (Reference 2.2.4) are global springs. The mat foundation finite element model (FEM) will have a support point located at each joint of the foundation mat mesh. Therefore, the global springs must be converted into individual springs applied to each joint of the foundation finite element mesh. This section contains the joint-spring calculation.

The full mat (global) lower bound 100 ft alluvium spring constants (Reference 2.2.4) are listed in Tables 6.2.1 and 6.2.2. As shown in the table, the labeling of the horizontal axis in the soil spring calculation coordinate system is different from the labeling of the axis used in this calculation (i.e. the *Y* axis of the soil spring calculation becomes the *X* axis of this mat calculation while the *X* axis of the soil spring calculation becomes the *Y* axis of this mat calculation).

Table 6.2.1 Full Mat (Global) Translational Spring Constants for Small Mat Foundation

Soil Spring Coordinate System	Value in kips/ft	Finite Element Model Coordinate System
KY	1.267E+06	KFX
KZ	1.583E+06	KFZ
KX	1.217E+06	KFY

Table 6.2.2 Full Mat (Global) Translational Spring Constants for Large Mat Foundation

Soil Spring Coordinate System	Value in kips/ft	Finite Element Model Coordinate System
KY	3.140E+06	KFX
KZ	3.822E+06	KFZ
KX	2.983E+06	KFY

Vertical Soil Spring Stiffness Distribution

As part of the finite element methodology, vertical full mat soil spring constants need to be distributed to individual joints. Ideally, this distribution is in proportion to the mat's tributary foundation area to properly represent the soil support. Due to the geometry from locations of walls and columns, the element areas for the small (west) mat are not equal. Therefore, a more precise distribution approach based on the exact tributary area of the joint is used to distribute vertical stiffness of soil springs (see Figure 6.1.3 and distribution calculation in Attachment I). The large (east) mat has a larger footprint (see Figure 6.1.4) and the vertical stiffnesses are uniformly distributed.

Lateral Soil Spring Stiffness Distribution

In order to capture the "collective resistance" of the soil stiffness against lateral sliding, full mat lateral soil spring constants are evenly distributed among all joints (i.e. the total spring constant divided by total number of joints), regardless of their tributary area size. This approach will be applied on the lateral soil spring stiffness distribution of both large (east) and small (west) foundation mats.

The spring stiffness distribution calculation for each mat follows. The calculated vertical stiffness of the nodal soil spring will be used in non-linear link elements, while the lateral stiffness will be used as the joint's spring stiffness in SAP2000 (Reference 2.2.19).

6.2.1 SMALL (WEST) MAT FOUNDATION

Vertical Spring Distribution

Vertical spring constant (stiffness) distribution is calculated based on the tributary area of each joint in Attachment I as described below. The finite element model for the small (west) mat foundation is shown in Figure 6.1.3 and the spring constant (stiffness) distribution at the joints of the finite elements mesh is described in Attachment I.

Table I-1 of Attachment I, calculates the tributary area of each joint. From the joint X (top row) coordinates and the joint Y (left vertical column) coordinates, edge lengths are calculated (the coordinates for each corner of the small mat are shown in parentheses below the link property ID in Figure 6.1.3). These tributary lengths are multiplied in order to determine the tributary area of each joint. The tributary area is arranged in a cell order corresponding to SAP2000 mesh. The SAP200 model for the small mat can be found in Attachment C.

Table I-2 of Attachment I shows the corresponding SAP2000 model link property identification (ID) in identical cell order to the SAP2000 mesh. The link property identification for each corner is shown in Figure 6.1.3.

Table I-3 of Attachment I distributes the vertical full mat soil spring stiffness ("KZ" from Table 6.2.1 as shown in row 2 of Table I-3) to each individual link. The unit area spring stiffness ("area spring kz." in row 2 of Table I-3) is calculated by dividing the vertical (global) soil spring stiffness (KZ) by the total area of the mat ("Total Area" in row 2 of Table I-3). Individual link stiffnesses are determined by multiplying the tributary area by the unit area spring stiffness. This distribution is arranged in the identical cell order, as Table I-1 with the link element properties ID of Table I-2.

Table I-4 of Attachment I assigns the individual link element properties of Table I-3 to the corresponding link element of Table I-2.

Table I-5 of Attachment I defines individual link element properties for input into SAP2000.

Table I-6 includes link IDs from Table I-2 and individual link soil spring stiffness' from Table I-3 for input into SAP2000.

These values are input in SAP2000 (Reference 2.2.19) as link property assignments and link property definitions.

Lateral Spring Distribution

Translational springs are derived from global values as follows:

Total mat foundation joints = 589

Horizontal springs in model X direction:

1.267E + 06 / 589 = 2,151 k/ft

Horizontal Springs in model Y direction:

1.217E + 06 / 589 = 2,066 k/ft

The above values are input as joint spring assignments in the global X and Y directions at foundation mesh joints on the small mat foundation model.

6.2.2 LARGE (EAST) MAT FOUNDATION

The finite element mesh for the large (east) mat is shown in Figure 6.1.4. The coordinates for each corner of the large mat are shown in parentheses below the link property ID in Figure 6.1.4. The spring constant (stiffness) distribution for the large mat is described below.

Vertical Spring Distribution

- The interior joints will have 100% of the nominal tributary area.
- The transition (3/4) corner joints (as indicated on Figure 6.1.4) will have a nominal tributary area of 75% of the interior joint's assigned spring value or 3/4 of the interior joint spring.
- The perimeter (1/2) joints will have a nominal tributary area of 50% of the interior joint's assigned spring value or 1/2 of the interior joint spring.
- The four corner (1/4) joints will have a nominal tributary area of 25% of the interior joint's assigned spring value or 1/4 of the interior joint spring.

Total number of joints = 1,491

Number of interior joints = 1,339

Number of 3/4 corner joints = 2

Number of 1/2 edge joints = 144

Number of 1/4 corner joints = 6

Letting k = spring value of the interior joint, the total spring value 'K' can be computed:

K = 1,339k + 2* 0.75k + 144*0.5 k + 6 * 0.25k = 1,414k

Where 'K' is KZ (3.822E+06 from Table 6.2.2). Therefore, k = 3.822E + 06 / 1414 = 2,703 kips/ft

Therefore, the vertical spring constant distribution is:

At interior joints = 2,703 kips/ft

At 3/4 corner joints = 3/4* k = 3/4* (2,703) = 2,027 kips/ft

At 1/2 edge joints = 1/2* k = 1/2* (2,703) = 1,351.5 kips/ft

At 1/4 corner joints = 1/4* k = 1/4* (2,703) = 675.7 kips/ft

Lateral Spring Distribution

The translational springs are derived from global values as follows:

Horizontal springs in model X direction:

3.140E + 06 / 1,491 = 2,106 k/ft

Horizontal springs in model Y direction:

2.983E + 06 / 1,491 = 2,001 k/ft

These values are input as joint spring assignments in the global X and Y directions at foundation mesh nodes on the large mat model.

6.3 LOADS AND LOADING COMBINATIONS

Based on Reference 2.2.1, Section 4.2.11.4.5, there are 17 reinforced concrete design load combinations. Loads such as H (Lateral earth pressure load), T_a (Thermal load during accident condition), T_0 (Thermal load during normal operating conditions), F (Fluid load), F' (Buoyant force of design basis flood), R_0 (Operating pipe reaction load), W (Wind load), W_t (Tornado load) and S (Snow load) are not included in this calculation.

The load combinations for the IHF mats are listed below:

a): DL + LL + E

b): 1.4DL + 1.7LL

Loads for the two mats come from the two sources: (1) the self weight of the mat and its superimposed dead and live loads, and (2) the external loads transmitted from the supporting structures (the steel and concrete structures). These external loads include both self-weight of the supporting structures, superimposed dead, live loads and equivalent seismic loads. Limited heavy equipment loads that exceed mat superimposed dead and live load will be taken into account when designing the local reinforcement in the final design (Assumption 3.1.1).

The seismic loads from the supporting super structures are derived from the seismic analysis in the east-west (EX), north-south (EY) and vertical (EZ) directions of References 2.2.3 and 2.2.5. Static loads are also derived from the structural analysis in Reference 2.2.3 and 2.2.5. In order to account for non-orthogonal seismic effects, the "100-40-40" component factor method from ASCE 4-98 (Reference 2.2.7, Section 3.2.7.1.2) is used. The "100-40-40" method uses 100%

seismic loading in one direction combined with 40% seismic in the remaining two directions. The "100-40-40" component factor method yields three basic seismic loading combinations:

±1.0 EX ± 0.4 EY ± 0.4 EZ
±0.4 EX ± 1.0 EY ± 0.4 EZ
±0.4 EX ± 0.4 EY ± 1.0 EZ

Expanding the above three seismic load combinations yields 24 loading permutations:

1.	+ 1.0	EX	+ 0.4 EY	+ 0.4	EZ
2.	+ 1.0	EX	− 0.4 EY	+ 0.4	EZ
3.	+ 1.0	EX	+ 0.4 EY	− 0.4	EZ
4.	+ 1.0	EX	− 0.4 EY	− 0.4	EZ
5.	− 1.0	EX	+ 0.4 EY	+ 0.4	EZ
6.	− 1.0	EX	− 0.4 EY	+ 0.4	EZ
7.	− 1.0	EX	+ 0.4 EY	− 0.4	EZ
8.	− 1.0	EX	− 0.4 EY	− 0.4	EZ
9.	+ 0.4	EX	+ 1.0 EY	+ 0.4	EZ
10.	− 0.4	EX	+ 1.0 EY	+ 0.4	EZ
11.	+ 0.4	EX	+ 1.0 EY	− 0.4	EZ
12.	− 0.4	EX	+ 1.0 EY	− 0.4	EZ
13.	+ 0.4	EX	− 1.0 EY	+ 0.4	EZ
14.	− 0.4	EX	− 1.0 EY	+ 0.4	EZ
15.	+ 0.4	EX	− 1.0 EY	− 0.4	EZ
16.	− 0.4	EX	− 1.0 EY	− 0.4	EZ
17.	+ 0.4	EX	+ 0.4 EY	+ 1.0	EZ
18.	− 0.4	EX	+ 0.4 EY	+ 1.0	EZ
19.	+ 0.4	EX	− 0.4 EY	+ 1.0	EZ
20.	− 0.4	EX	− 0.4 EY	+ 1.0	EZ
21.	+ 0.4	EX	+ 0.4 EY	− 1.0	EZ
22.	− 0.4	EX	+ 0.4 EY	− 1.0	EZ
23.	+ 0.4	EX	− 0.4 EY	− 1.0	EZ
24.	− 0.4	EX	− 0.4 EY	− 1.0	EZ

Combining the seismic components with dead and live loads yields the 24 load combinations used in the calculation:

1. DL + LL + 1.0 EX + 0.4 EY + 0.4 EZ
2. DL + LL + 1.0 EX − 0.4 EY + 0.4 EZ
3. DL + LL + 1.0 EX + 0.4 EY − 0.4 EZ
4. DL + LL + 1.0 EX − 0.4 EY − 0.4 EZ
5. DL + LL − 1.0 EX + 0.4 EY + 0.4 EZ
6. DL + LL − 1.0 EX − 0.4 EY + 0.4 EZ
7. DL + LL − 1.0 EX + 0.4 EY − 0.4 EZ
8. DL + LL − 1.0 EX − 0.4 EY − 0.4 EZ
9. DL + LL + 0.4 EX + 1.0 EY + 0.4 EZ
10. DL + LL − 0.4 EX + 1.0 EY + 0.4 EZ
11. DL + LL + 0.4 EX + 1.0 EY − 0.4 EZ

12. DL + LL − 0.4 EX + 1.0 EY − 0.4 EZ
13. DL + LL + 0.4 EX − 1.0 EY + 0.4 EZ
14. DL + LL − 0.4 EX − 1.0 EY + 0.4 EZ
15. DL + LL + 0.4 EX − 1.0 EY − 0.4 EZ
16. DL + LL − 0.4 EX − 1.0 EY − 0.4 EZ
17. DL + LL + 0.4 EX + 0.4 EY + 1.0 EZ
18. DL + LL − 0.4 EX + 0.4 EY + 1.0 EZ
19. DL + LL + 0.4 EX − 0.4 EY + 1.0 EZ
20. DL + LL − 0.4 EX − 0.4 EY + 1.0 EZ
21. DL + LL + 0.4 EX + 0.4 EY − 1.0 EZ
22. DL + LL − 0.4 EX + 0.4 EY − 1.0 EZ
23. DL + LL + 0.4 EX − 0.4 EY − 1.0 EZ
24. DL + LL − 0.4 EX − 0.4 EY − 1.0 EZ

Where

DL = 100% self weight of the mat + 100% superimposed dead load on mat + 100% superimposed dead load from supporting superstructure (Sections 6.4.1 and 6.4.4 of Reference 2.2.21).

LL = 25% of superimposed live load from supporting superstructures (Reference 2.2.12, Section 8.3.1) + 100% of 100 psf superimposed live load on mat per Assumption 3.1.2.

In addition to the 24 “100-40-40” load case combinations, Bullet Point B, Section 4.2.11.4.4 of PDC (Reference 2.2.1) requires that where any load reduces the effect of other loads, the corresponding coefficient for that load shall be taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with other loads. Otherwise, the coefficient shall be taken as zero. This requirement further expands the standard “100-40-40” 24 load case combinations to include another 24 load cases. These expanded load cases are listed as follows:

1a. 0.9DL + LL + 1.0 EX + 0.4 EY + 0.4 EZ
2a. 0.9DL + LL + 1.0 EX − 0.4 EY + 0.4 EZ
3a. 0.9DL + LL + 1.0 EX + 0.4 EY − 0.4 EZ
4a. 0.9DL + LL + 1.0 EX − 0.4 EY − 0.4 EZ
5a. 0.9DL + LL − 1.0 EX + 0.4 EY + 0.4 EZ
6a. 0.9DL + LL − 1.0 EX − 0.4 EY + 0.4 EZ
7a. 0.9DL + LL − 1.0 EX + 0.4 EY − 0.4 EZ
8a. 0.9DL + LL − 1.0 EX − 0.4 EY − 0.4 EZ
9a. 0.9DL + LL + 0.4 EX + 1.0 EY + 0.4 EZ
10a. 0.9DL + LL − 0.4 EX + 1.0 EY + 0.4 EZ
11a. 0.9DL + LL + 0.4 EX + 1.0 EY − 0.4 EZ
12a. 0.9DL + LL − 0.4 EX + 1.0 EY − 0.4 EZ
13a. 0.9DL + LL + 0.4 EX − 1.0 EY + 0.4 EZ
14a. 0.9DL + LL − 0.4 EX − 1.0 EY + 0.4 EZ
15a. 0.9DL + LL + 0.4 EX − 1.0 EY − 0.4 EZ
16a. 0.9DL + LL − 0.4 EX − 1.0 EY − 0.4 EZ

17a. 0.9DL + LL + 0.4 EX + 0.4 EY + 1.0 EZ
18a. 0.9DL + LL − 0.4 EX + 0.4 EY + 1.0 EZ
19a. 0.9DL + LL + 0.4 EX − 0.4 EY + 1.0 EZ
20a. 0.9DL + LL − 0.4 EX − 0.4 EY + 1.0 EZ
21a. 0.9DL + LL + 0.4 EX + 0.4 EY − 1.0 EZ
22a. 0.9DL + LL − 0.4 EX + 0.4 EY − 1.0 EZ
23a. 0.9DL + LL + 0.4 EX − 0.4 EY − 1.0 EZ
24a. 0.9DL + LL − 0.4 EX − 0.4 EY − 1.0 EZ

where

DL = 100% of self-weight and + 10% of steel platform superimposed loads (Assumption 3.2.3) + 0% of superimposed DL from supporting superstructure + 0% of superimposed DL on mat;

LL = 0% of superimposed live load from supporting superstructure and 0% of the superimposed live load on mat (Bullet Point B, Section 4.2.11.4.4 of PDC, Reference 2.2.1).

The following static load case (section 4.2.11.4.5 of PDC, Reference 2.2.1) also needs to be included:

1.4DL + 1.7LL

Where

DL = 100% of mat self-weight + 100% of superimposed dead load on mat + 100% superimposed dead load from supporting superstructure (Sections 6.4.1and 6.4.4 of Reference 2.2.21).

LL = 25% of live load on supporting superstructure + 100% superimposed live load (100 psf) on mat (Assumption 3.1.2).

6.3.1 SESIMIC AND NON-SEISMIC DEAD LOADS

The self-weight of the foundation mats are automatically calculated by SAP2000 based on element properties. Steel structure loads and wall loads are from calculations *Initial Handling Facility (IHF) Steel Structure Seismic Analysis and Steel Member Design* (Reference 2.2.3) and calculations *Initial Handling Facility (IHF) Concrete Structure Design* (Reference 2.2.5).

6.4 SAP2000 ANALYSIS RESULTS

6.4.1 BENDING MOMENTS AND SHEAR FORCES IN THE MAT FOUNDATION

Stress contour plots for the foundation mat are included in Attachments D and E. The contour plots represent bending moments M11 (along X-axis) and M22 (along Y-axis), twisting moment M12, and shear forces V13 (along X-axis) and V23 (along Y-axis). The axes of these coordinates are in the global coordinates of the model.

The design forces (maximum moment and shear values) used in Tables 6.5.1, 6.5.2, 6.5.3 and 6.5.5 of this section are derived graphically by visual inspection for the extreme value on the field contours in Attachments D and E. Figures 6.4.1 and 6.4.2 illustrate the force field of the element (where axis 1 represents the model's X-axis and axis 2 represents the model's Y-axis).

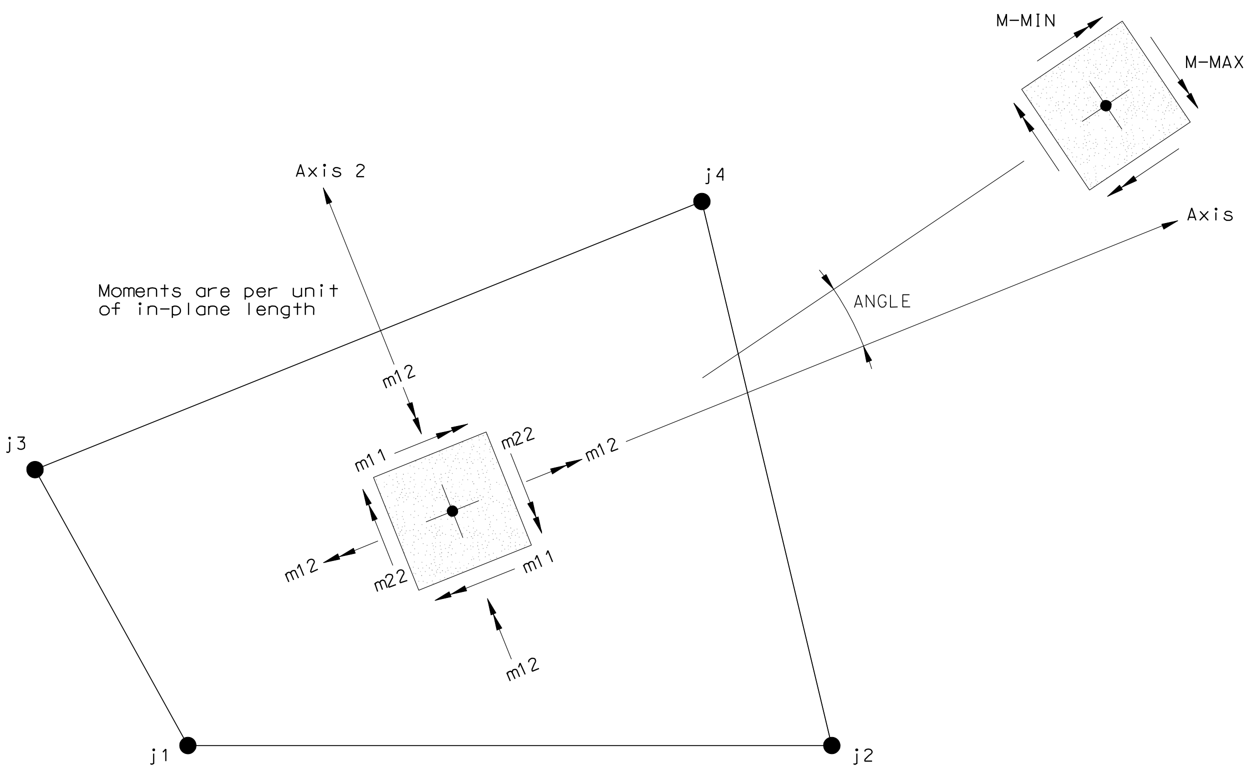
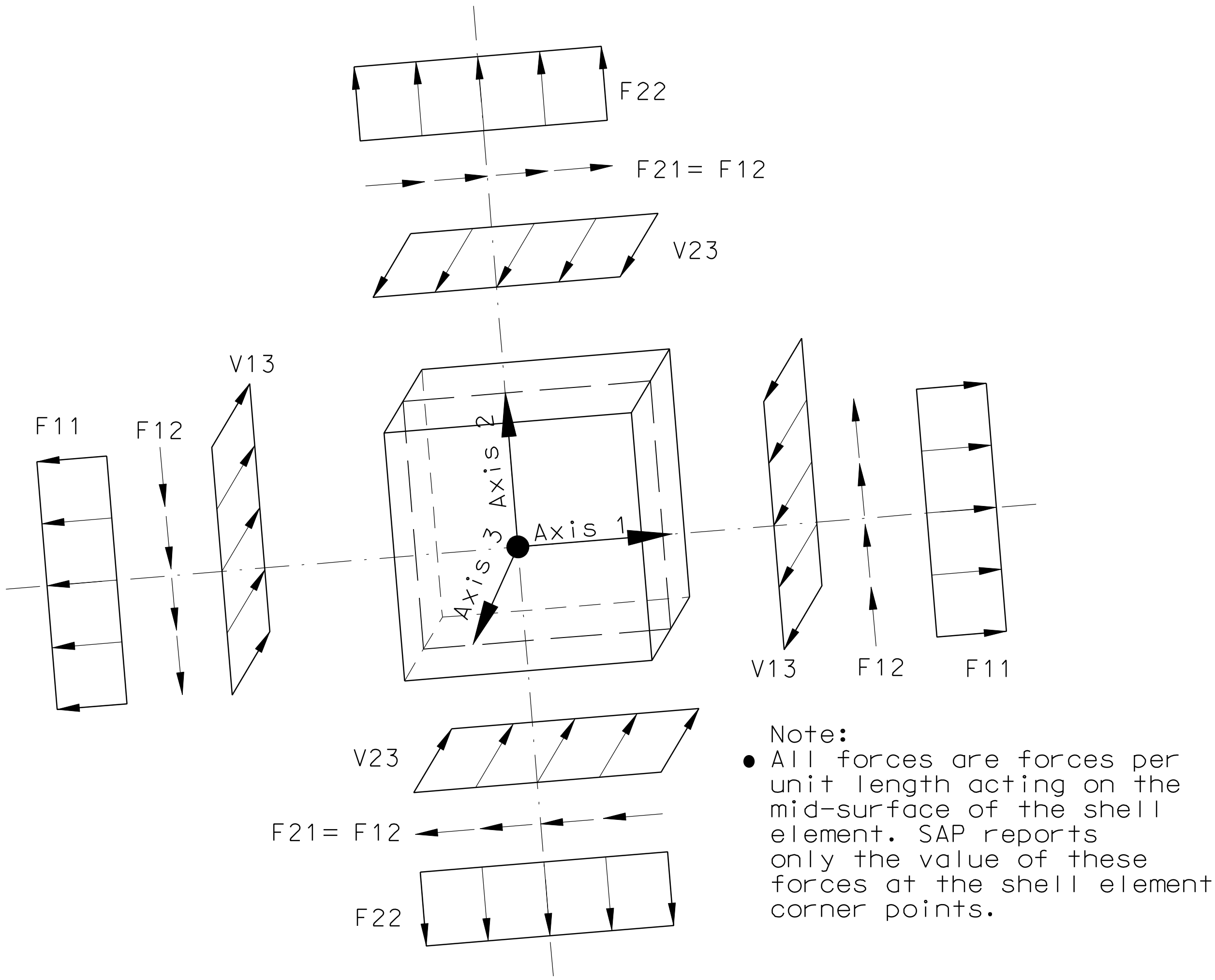


Figure 6.4.1 Shell Element Bending and Twisting Moments



- In-Plane axial forces (F11, F22, etc.) are very small compared to the section size. Therefore, they are neglected in reinforcing design.

Figure 6.4.2 Shell Element Membrane and Shear Forces

6.5 REINFORCING DESIGN

Project Design Criteria Document (Section 4.2.11.6.2 of Reference 2.2.1) specifies the concrete and reinforcing materials recommended for ITS surface facilities as follows:

Concrete compressive strength, f'_c, of 4,000 or 5,000 psi, and
- Grade 60 steel bar reinforcing steel.

Reinforcement Concrete Design Parameters (section 4.2.11.6.6 of PDC, Reference 2.2.1):

$f'_c = 5000\,psi$

$E'c = 4.29x10^6\ psi$

$Possion's\ ratio\ \nu = 0.17$

- Minimum concrete cover is 3 inches (per Reference 2.2.6, ACI 349-01 Section 7.7.1(a), for concrete cast against and permanently exposed to earth).
- Grade 60 bar reinforcing steel:

  $f_y = 60ksi$

  $Es = 29x10^6\ psi$

  $Possion's\ ratio\ \nu = 0.3$

6.5.1 SMALL (WEST) MAT MOMENT AND SHEAR CAPACITY

Moment and Shear Capacity for 6 ft thick mid-span foundation mat:

Determine the effective structural depth, *d*, (Reference 2.2.6, Section 10.0) by using a double layer of #18 bars, 12 inches on center each way, top and bottom of the foundation mat:

For a mat foundation with a depth of 72 inches, minimum concrete cover of the rebar = 3 inches, #18 bar diameter d_b=2.257 in., then the effective depth *d* of rebar = (72"- 3"- 2.5*2.257) = 63.4 inches.

Calculate the mat moment capacity using two layers of #18 bars at 12 inches on center, top and bottom:

As = 8.0 in^2

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) \geq M_u$$

ϕ = Strength reduction factor for flexure = 0.9 (Reference 2.2.6, Section 9.3.2.2(a))

Where

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8\ in.^2 (60\ ksi)}{0.85\ (5\ ksi)(12\ in.)} = 9.41\ in.$$

$$\phi M_n = 0.9\ (8\ in.^2)(60\ ksi)(63.40\ in. - \frac{9.41\ in.}{2})x(12\ in./12in.)/(12in/1ft) = 2113.0\ kips - ft/ft$$

Moment capacity for the 6 ft. thick mat = 2113 *k-ft/ft*

Determine the beam shear capacity of 5,000-psi concrete for a 1 ft wide strip with an effective depth of 63.4 in. (Reference 2.2.6, Equation 11-3):

ϕ = strength reduction factor for out-of-plane shear = 0.85 (Reference 2.2.6, Section 9.3.2.3)

$$\phi V_c = \phi 2 \sqrt{f_c^{'}}\ bd = \frac{0.85\ (2)\ \sqrt{5{,}000\,psi}\ (12\ in./\ ft)(63.40\ in.)}{1{,}000\,lb/kip} = 91.45\ kips/ft$$

Determine the shear capacity of #6 stirrup ties at 12 in. on center in each direction (Reference 2.2.6, Equation 11-15):

$$\varphi V_S = \varphi\ A_v{*}f_y{*}d\ /\ s$$

with, φ=0.85, s = 12 inches and A_v = 0.44 sq-inch / ft

then

$$\phi V_s = 0.85 * 0.44 * 60 * 63.4/12 = 118.56 kips/ft$$

The total shear capacity for 12 inches strip of concrete plus stirrup ties is:

= $\varphi\ V_c + \varphi\ V_S$ = (91.45 + 118.56) = 210.01 *kips/ft*

Moment and Shear Capacity for 13 ft thick mid-span foundation:

Determine the effective structural depth, d, (Reference 2.2.6, Section 10.0) by using a double layer of #18 bars, 12 inches on center each way, top and bottom of the foundation mat:

For a mat foundation with a depth of 156 inches, minimum concrete cover of the rebar = 3 inches, #18 bar diameter d_b=2.257 in., then the effective depth d of rebar is:

$$d = 156\ in. - 3\ in.(\text{cover}) - 2.5d_b = 156in. - 3\ in. - 2.5(2.257\ in.) = 147.36\ in.$$

Calculate the mat moment capacity using two layers of #18 bars at 12 inches on center, top and bottom:

A_s = 8.0 in^2

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) \geq M_u$$

ϕ = Strength reduction factor for flexure = 0.9 (Reference 2.2.6, Section 9.3.2.2(a))

where,

$$a = \frac{A_s f_y}{0.85 f_c^{'} b} = \frac{8\ in.^2\,(60\ ksi)}{0.85\ (5\ ksi)(12\ in.)} = 9.41\ in.$$

$$\phi M_n = 0.9\ (8\ in.^2)(60\ ksi)(147.36\ in. - \frac{9.41\ in.}{2})x(12\ in./12in.)/(12in/1ft) = 5135.6\ kips - ft/ft$$

Moment capacity for the 13 ft. thick mat = 5135.6 *kips-ft/ft*

Determine the beam shear capacity of 5,000-psi concrete for a 1 ft wide strip with an effective rebar depth of 147.36 in. (Reference 2.2.6, Equation 11-3):

ϕ = strength reduction factor for out-of-plane shear = 0.85 (Reference 2.2.6, Section 9.3.2.3)

$$\phi V_c = \phi 2\sqrt{f_c'}\ bd = \frac{0.85\ (2)\ \sqrt{5{,}000\,psi}\ (12\ in./ft)(147.36\ in.)}{1{,}000\,lb/kip} = 212.57\ kips/ft$$

Determine the shear capacity of #6 stirrup ties at 12 in. on center in each direction (Reference 2.2.6, Equation 11-15):

$\varphi V_S = \varphi\ A_v{*}f_y{*}d\ /\ s$

with, φ=0.85, s = 12 inches and A_v = 0.44 sq-inch / ft

then

$\phi V_s = 0.85 * 0.44 * 60 * 147.36 / 12 = 275.56 kips / ft$

The total shear capacity for 12 inches strip of concrete plus stirrup ties is:

= $\varphi\ V_c + \varphi\ V_S$ = 212.57 + 275.56 = 488.13 *kips/ft*

6.5.2 SMALL MAT MAXIMUM MOMENT AND SHEAR DEMAND/CAPACITY COMPARISON

The maximum moment and shear capacities for the small mat were compared to the maximum moment and shear demand values obtained from the contour plots (M11, M22, V13, and V23 shown in Attachment E). The torsional moment M12 was added to the demand values for M11 and M22 to determine the total moment demand (Section 15-2 of Reference 2.2.9).

Table 6.5.1 and Table 6.5.2 summarize the maximum Demand/Capacity (D/C) ratios for moments and shears.

Table 6.5.1 IHF Small Mat Foundation Maximum Moment D/C Ratios

Mat Type	Maximum Moment M (kips-ft/ft)	M12 at Maximum Moment (kips-ft/ft)	Total Demand (D) \M\+\M12\ (kips-ft/ft)	Capacity (C) (kips-ft/ft)	D/C Ratio	Crane Load Case	Appendix E Page No.
13 feet thickness mat	−M11 −1210	−550	1760	5,136	0.34	1	E-3 E-6
	+M11 1700	550	2,250	5,136	0.44	2	E-12 E-6
	−M22 −750	−550	1,300	5,136	0.25	2	E-15 E-6
	+M22 1320	550	1,870	5,136	0.36	2	E-14 E-6
6 feet thickness mat	−M11 −660	−350	1010	2,113	0.48	2	E-13 E-16
	+M11 1020	350	1,370	2,113	0.65	2	E-12 E-16
	−M22 −825	−350	1,175	2,113	0.56	2	E-15 E-16
	+M22 960	350	1,310	2,113	0.62	2	E-14 E-16

As shown in Table 6.5.1, all moment D/C ratios are less than one. Therefore, the two layers #18 reinforcing bars spaced at 12” on center, each way, top and bottom, is adequate for the small mat.

Table 6.5.2 IHF Small Mat Foundation Maximum Shear D/C Ratios

Mat Type	Maximum Shear V (kips/ft)	Capacity (C) (kips/ft)	D/C	Crane Load Case	Appendix E Page No.
13 feet thickness mat	−V13 −135	488	0.28	2	E-19
	+V13 153	488	0.31	2	E-8 & E-18
	−V23 −176	488	0.36	2	E-21
	+V23 198	488	0.41	2	E-20
6 feet thickness mat	−V13 −120	210	0.57	2	E-19
	+V13 136	210	0.65	2	E-18
	−V23 −176	210	0.84	2	E-21
	+V23 176	210	0.84	2	E-20

As shown in Table 6.5.2, all shear D/C ratios are less than one. Therefore, the #6 stirrup ties spaced at 12" on center, each way, is adequate for the small mat.

6.5.3 LARGE (EAST) MAT MOMENT AND SHEAR CAPACITY

For 6 ft thick large foundation mat:

The large mat will mainly be reinforced with two layers of #18 bars at 12" on center.
Based on the SAP2000 Moment and Shear Contours for Large Mat (Attachment D), the maximum M11value for the double layer reinforcing area (see Figure 6.5.3) is 1260 *kips-ft/ft* at CLC7 (see page D-64 of Attachment D), which is the maximum for all load cases. The corresponding M12 Value is 420 *kips-ft/ft* (see page D-66 of Attachment D).

M11+ M12 = 1260+420 = 1680 *kips-ft /ft* (Section 15-2 of Reference 2.2.9)

The maximum M22 value is 1350 *kips-ft/ft* for CLC6 (see page D-58 of Attachment D), which is the maximum for all load cases. The corresponding M12 value is 390 *kips-ft/ft* (see page D-56 of Attachment D).

M22 + M12 = 1350 + 390 =1740 *kips-ft/ft*. (Section 15-2 of Reference 2.2.9)

For calculation of the concrete moment capacity, including 2 layers of #18 bars at 12" on center, see the small (west) mat calculation below. Portions of the large mat with high stresses require 3 layers of #18 bars at 12" on center (see Figure 6.5.3). In such areas, the moment capacity is calculated using triple layers of #18 bars at 12 in. on-center as follows:

The effective depth *d* of rebar is 72"-3"-3.5*2.257=61.10 inches.

Area of steel for triple layers of #18 bars (As = 4 in^2):

As = 3 * 4in^2 =12.0 in^2

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) \geq M_u \quad \text{(Reference 2.2.6, Chapter 10)}$$

ϕ = Strength reduction factor for flexure = 0.9 (Reference 2.2.6, Section 9.3.2.2(a))

Where

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{12\ in.^2 (60\ ksi)}{0.85\ (5\ ksi)(12\ in.)} = 14.12\ in.$$

$$\phi M_n = 0.9\ (12\ in.^2)(60\ ksi)(61.1\ in. - \frac{14.12\ in.}{2}) x (12\ in./12 in.)/(12 in/1 ft) = 2918.2\ kips - ft/ft$$

Note: Triple layers of #18 bars will only be used at certain areas on large mat, refer to Figure 6.5.3.

Determine the beam shear capacity of 5,000-psi concrete requirement for a 1 ft strip with an effective depth of 61.10 in. (Reference 2.2.6, Equation 11-3):

$$\phi V_c = \phi 2 \sqrt{f_c'}\ bd = \frac{0.85\ (2)\ \sqrt{5{,}000 psi}\ (12\ in./ft)(61.10\ in.)}{1{,}000 lb/kip} = 88.14\ kips/ft$$

Determine the shear capacity of #6 stirrup ties at 12 in. on center in each direction (Reference 2.2.6, Equation 11-15):

$\varphi V_S = \varphi\ A_v{*}f_y{*}d\ /\ s$

with, φ=0.85, s = 12 in.

$A_v = .44\ in.^2\ /\ ft$

then

$\phi V_s = 0.85 * 0.44 * 60 * 61.1/12 = 114.26 kips/ft$

For the total 12 inches strip shear capacity of concrete plus stirrup ties

= $\varphi\ V_c + \varphi\ V_S$ = 88.14 + 114.26 = 202.40 *kips/ft*

Large Mat Foundation

Table 6.5.3 IHF Large Mat Foundation Maximum Moment Summary Table

Load Case	M11 + M12				M22 + M12			
	Max. M11 Kips-ft/ft	M12 at Max. M11 Kips-ft/ft	M11+M12 Kips-ft/ft	Page #	Max. M22 Kips-ft/ft	M12 at Max. M22 Kips-ft/ft	M22+M12 Kips-ft/ft	Page #
CLC1	1950	385	2335	D-4 D-6	1680	385	2065	D-8 D-6
CLC2	2340	420	2760	D-14 D-16	1920	420	2340	D-18 D-16
CLC3	2340	455	2795	D-24 D-26	1690	455	2145	D-28 D-26
CLC4	2080	420	2500	D-34 D-36	1820	420	2240	D-38 D-36
CLC5	2080	440	2520	D-44 D-46	1820	440	2260	D-48 D-46
CLC6	2340	390	2730	D-54 D-56	1800	390	2190	D-58 D-56
CLC7	2340	490	2830	D-64 D-66	1800	420	2220	D-68 D-66
Max.			2830				2340	

Note: Based on Attachment D - SAP 2000 Moment and Shear Contours for large mat, the absolute values of Min. M11 and M22 are consistently smaller than Max. M11 and M22, and are not controlling the design, thus only Max M11 and M22 values were shown in this table.

Table 6.5.4 IHF Large Mat Foundation Maximum Moment D/C Ratios

Total Demand (kip-ft/ft)	Capacity © (kip-ft/ft)	D/C	Crane Load Case	REINFORCING
2830	2918	0.97	CLC7	3 layers #18 each way
1740	2113	0.82	CLC6	2 layers #18 each way as shown on enclosed area on pages D-56 & D-58

As shown in Table 6.5.4, all moment D/C ratios are less than one. Therefore, the specified reinforcing is adequate for the large mat.

Table 6.5.5 IHF Large Mat Foundation Maximum Shear Summary Table

Load Case	Max. V13			Max. V23		
	+V13 Kips/ft	-V13 Kips/ft	Page #	+V23 Kips/ft	-V23 Kips/ft	Page #
CLC1	175	-168	D-10 D-11	168	-176	D-12 D-13
CLC2	196	-180	D-20 D-21	196	-196	D-22 D-23
CLC3	196	-196	D-30 D-31	196	-192	D-32 D-33
CLC4	196	-168	D-40 D-41	196	-192	D-42 D-43
CLC5	180	-196	D-50 D-51	196	-187	D-52 D-53
CLC6	196	-196	D-60 D-61	196	-196	D-62 D-63
CLC7	196	-196	D-70 D-71	196	-196	D-72 D-73
Max.	196	-196		196	-196	

Table 6.5.6 IHF Large Mat Foundation Maximum Shear D/C Ratios

Maximum Shear V (kips/ft)	Capacity (C) 6 ft Mat (kips/ft)	D/C
−V13 −196	202.4	0.97
+V13 196	202.4	0.97
−V23 −196	202.4	0.97
+V23 196	202.4	0.97

As shown in Table 6.5.6, all shear D/C ratios are less than one. Therefore, the #6 stirrup ties spaced at 12” on center, each way, is adequate for the large mat.

From the contour plot, a 10 ft distance from the centerline of steel column or concrete wall was judged to be adequate for shear reinforcing. Beyond that distance, the shear demand is less than the shear capacity for plain concrete.

Figures 6.5.1, 6.5.2, 6.5.3 and 6.5.4 provide a foundation plan view and sections, showing flexural and shear reinforcement.

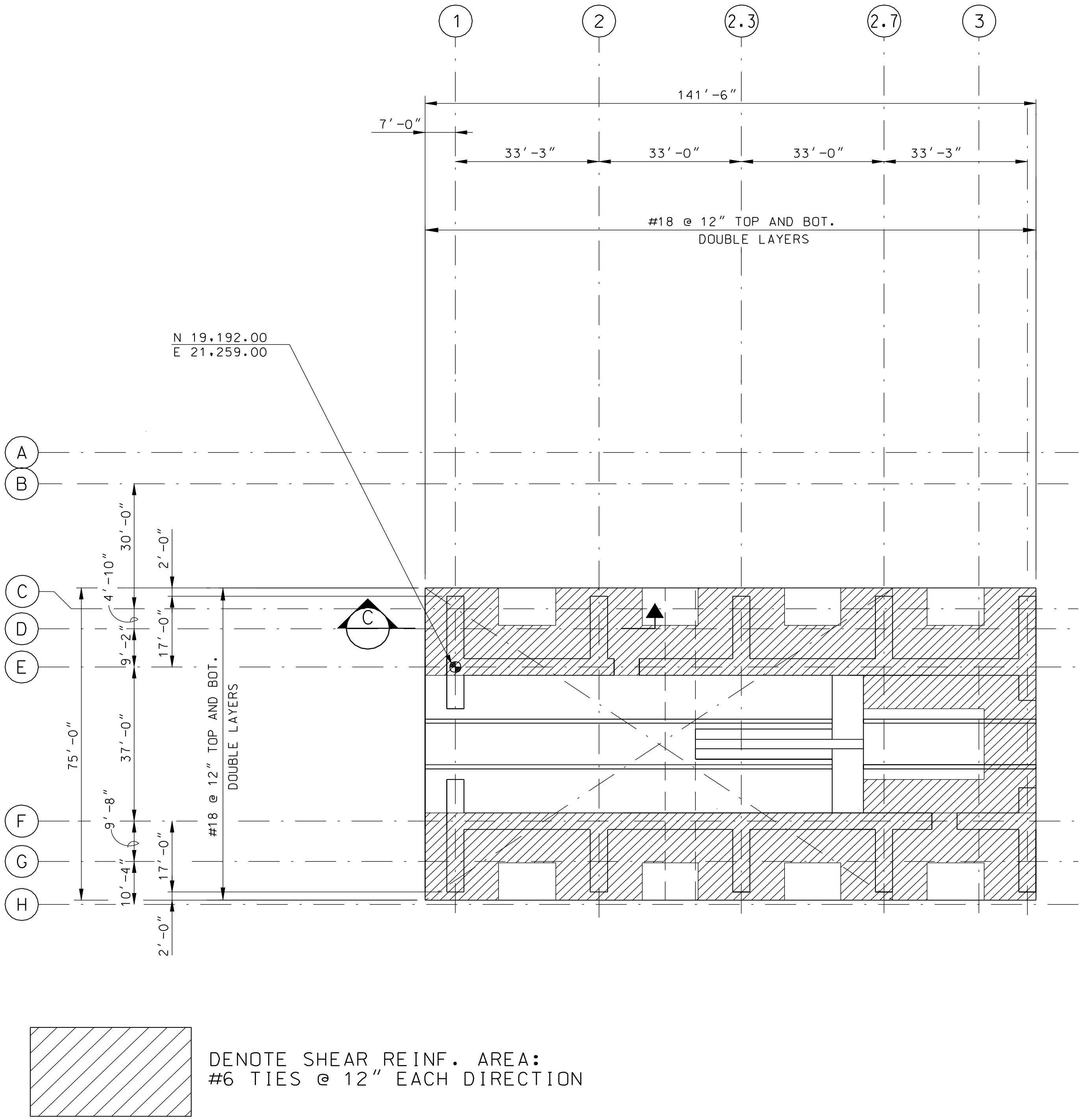


Figure 6.5.1 Small Mat Foundation Reinforcement Plan

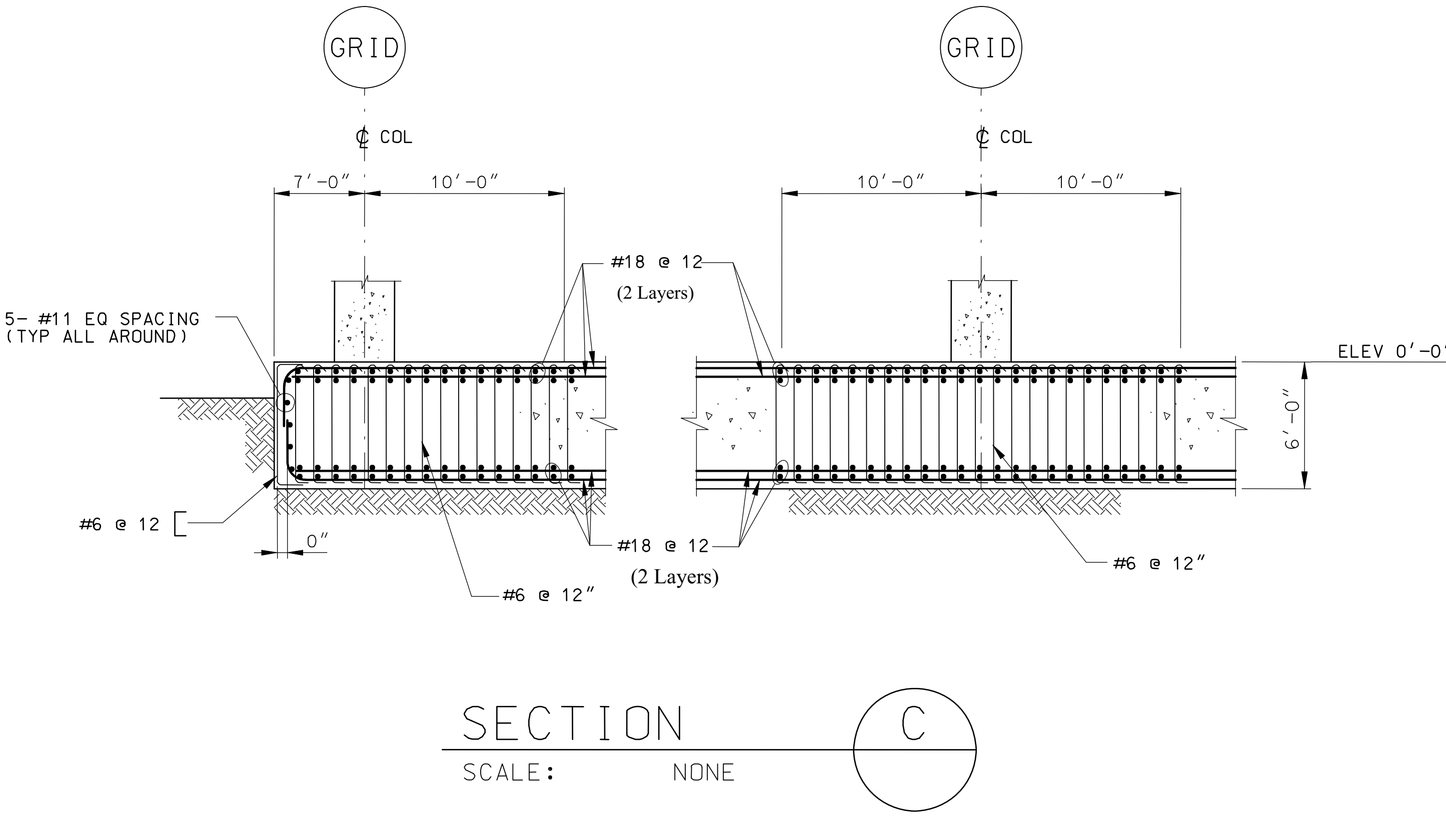


Figure 6.5.2 Small Mat Foundation Cross Section

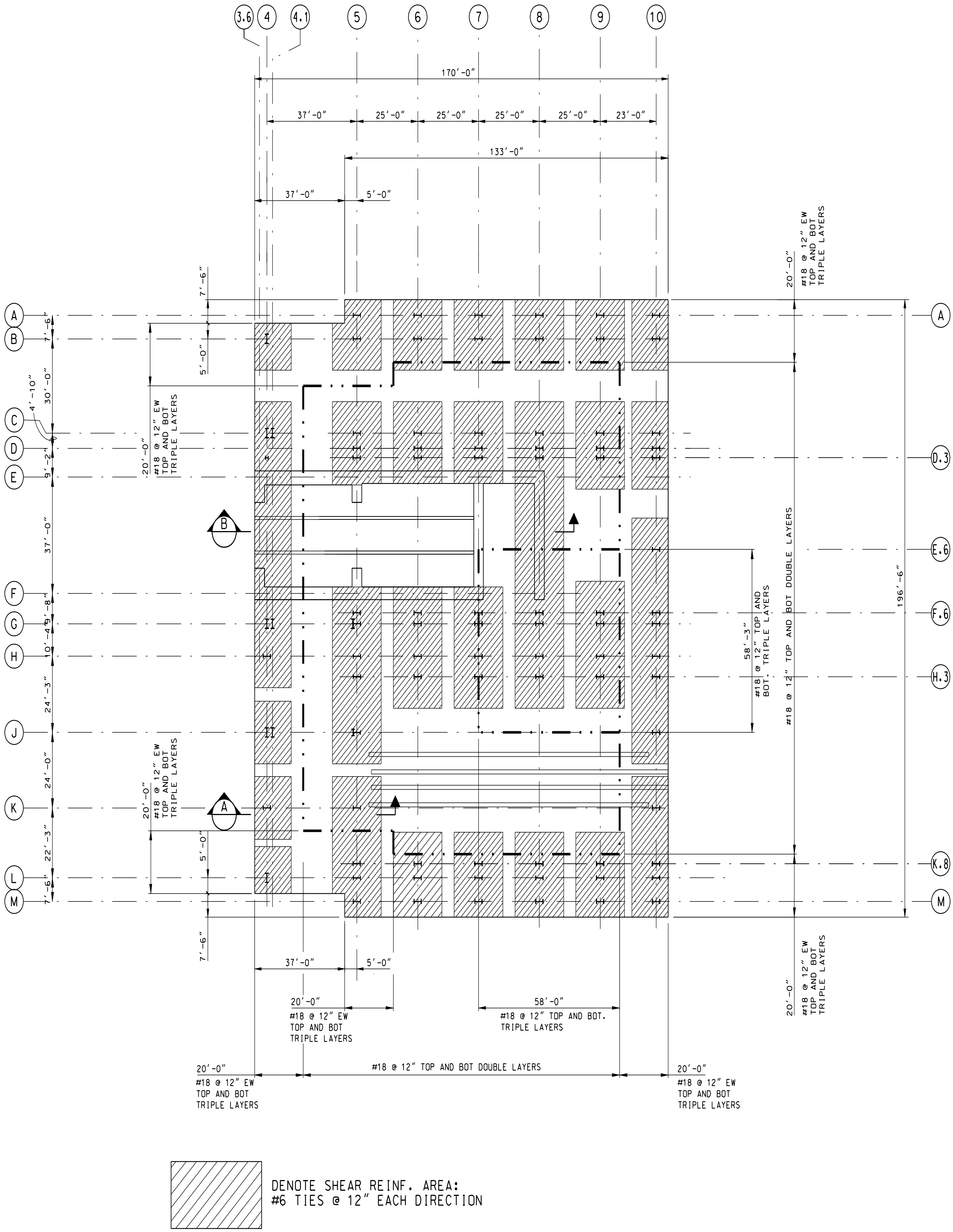
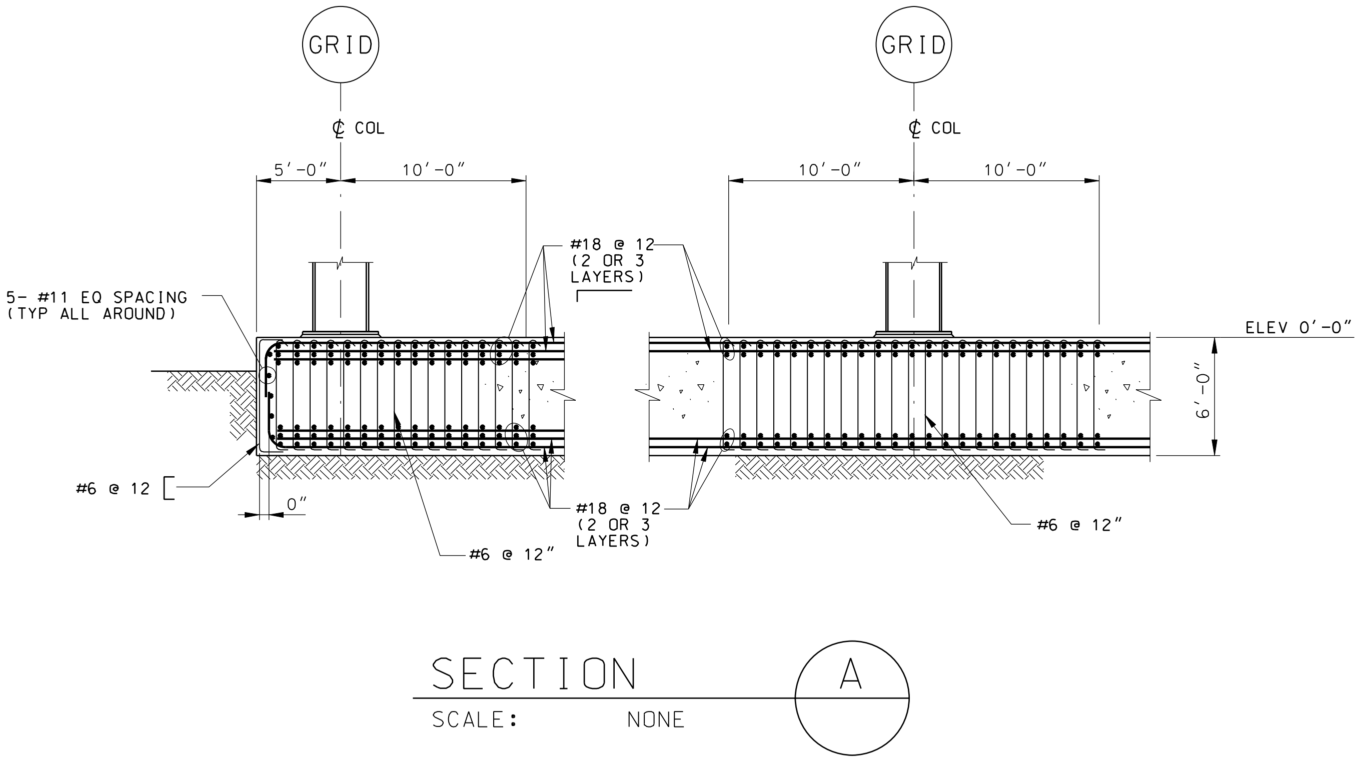


Figure 6.5.3 Large Mat Foundation Reinforcement Plan



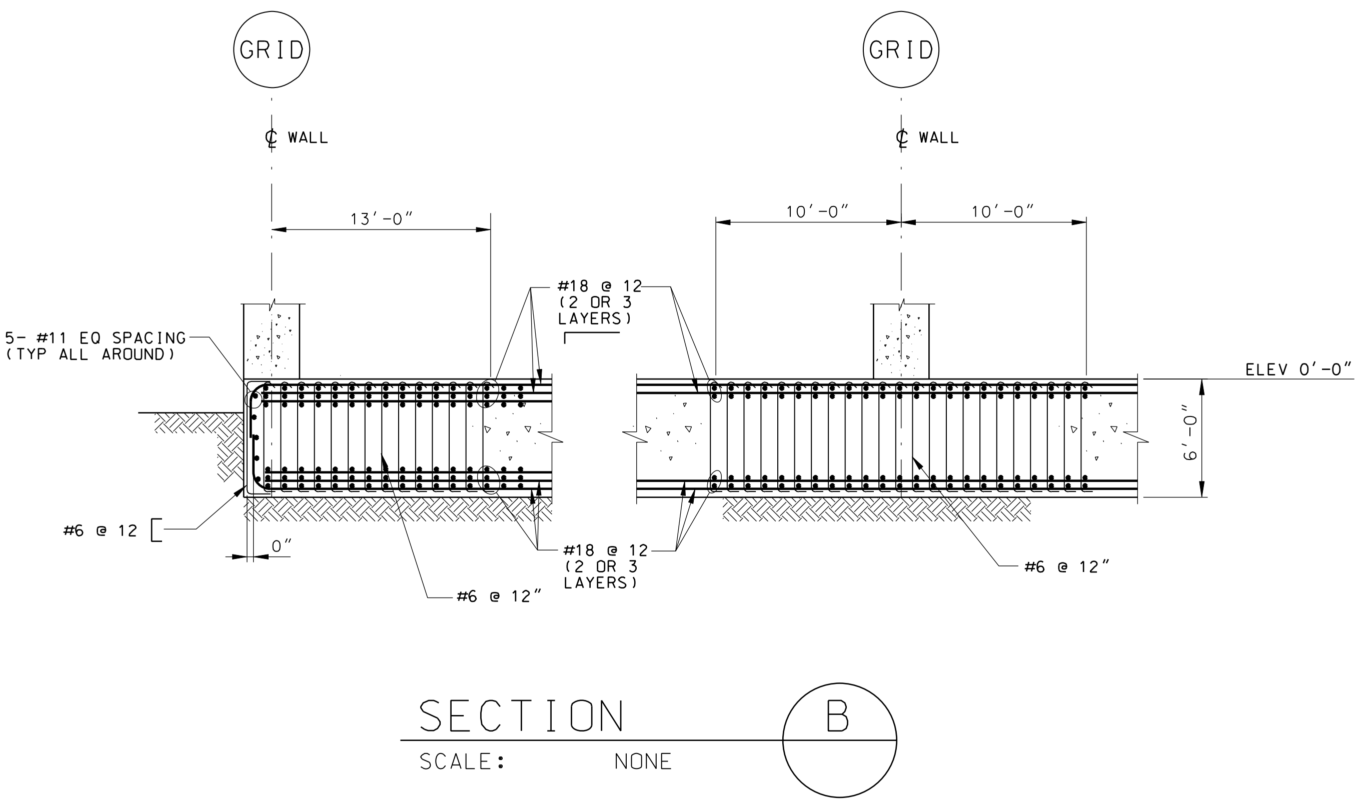


Figure 6.5.4 Large Mat Foundation Cross Section

6.6 STRUCTURAL STABILITY EVALUATION

This section evaluates the stability of the IHF small and large mat foundations in sliding and overturning. Seismic Analysis and Design Approach Document (Reference 2.2.12, Section 11.1) is used for the evaluation of sliding stability. The approximate sliding displacement is calculated by utilizing the energy method suggested in ASCE/SEI 43-05 (Reference 2.2.8).

6.6.1 SMALL MAT STABILITY AGAINST SLIDING

$PSA_H := 1.0267$ peak spectral acceleration (horizontal 7% damping)
Ref. 2.2.12, Table 6-5

$PSA_V := .7169$ peak spectral acceleration (vertical 7% damping)
Ref. 2.2.12, Table 6-6

$ZPA_H := .4537$ zero period ground acceleration Ref. 2.2.12, Table 6-5

$ZPA_V := .3194$ zero period ground acceleration Ref. 2.2.12, Table 6-6

Total applied vertical load =

From Attachment G - BaseReactions1-3 Load Case 2 (Load Combinations) *Base Reactions Worksheet* (cell D/39). This load includes structure self weight, roof & crane dead loads, and roof & crane live loads in the vertical direction

$Load_{vert} := 18658.8 \cdot k$

Determine weight of foundation slab (see Ref. 2.2.14 to 2.2.18) $i := 0 .. 2$

$t_{slab_i} :=$ $L_{slab_i} :=$ $width_{slab} := 75 \cdot ft$ $t_{found} := 6 \cdot ft$ thickness of foundation

t_{slab_i}	L_{slab_i}
$6 \cdot ft$	$55.5 \cdot ft$
$13 \cdot ft$	$43.5 \cdot ft$
$6 \cdot ft$	$42.5 \cdot ft$

concrete weight = $w_{con} := 150 \cdot \frac{lbf}{ft^3}$ see Ref. 2.2.1

$weight_{sect_i} := t_{slab_i} \cdot L_{slab_i} \cdot width_{slab} \cdot w_{con}$ $weight_{sect} = \begin{pmatrix} 3746.2 \\ 6361.9 \\ 2868.8 \end{pmatrix} k$

Total weight of slab= $weight_{slab} := weight_{sect_0} + weight_{sect_1} + weight_{sect_2}$

$weight_{slab} = 12977\, k$ total weight of foundation slab

Applied seismic force

$PH1 := PSA_H \cdot Load_{vert}$ $PH1 = 19157\, k$

$PH2 := ZPA_H \cdot weight_{slab}$ $PH2 = 5887.6\, k$

Total applied seismic force= $f_{seisappl} := PH1 + PH2$ $f_{seisappl} = 25044.6\, k$

Total resisting force

coefficient of friction (for alluvium) = $\mu := .81$ Ref. 2.2.12, Table 6-2

Total Weight= $Wt_{tot} := weight_{slab} + Load_{vert}$ $Wt_{tot} = 31635.7\, k$

$f_{resist} := \mu \cdot (1 - ZPA_V) \cdot Wt_{tot}$ $f_{resist} = 17440.3\ k$

Factor of Safety Against Sliding= $FS_{sl} := \dfrac{f_{resist}}{f_{seisappl}}$ $FS_{sl} = 0.7$

This calculation indicates the structure will slide, therefore the energy method must be used to determine the maximum sliding displacement. (Ref. 2.2.8)

<u>Determine Sliding Displacement</u> (from Appendix A, Ref. 2.2.8)

effective coefficient of friction= $\mu_e := \mu \cdot (1 - .4 \cdot ZPA_V)$ $\mu_e = 0.71$ Ref. 2.2.8 Equation A-1

sliding coefficient= $c_s := 2 \cdot \mu_e \cdot g$ $c_s = 1.41\ g$ Ref. 2.2.8 Equation A-2

f_{es} = The lowest natural frequency at which the horizontal 10% damped vertical spectral acceleration equals c_s

SA_{H1} = The 10% damped spectral acceration for horizontal direction 1

SA_{H2} = The 10% damped spectral acceration for horizontal direction 2

$SA_{H1}=SA_{H2}$

$SA_{VH} := c_s$ $SA_{VH} = 1.41\ g$

$SA_{VH} = (1+.16)^{1/2} * SA_{H1}$ (Ref. 2.2.8, Eq. A-4) $var1 := (1 + .16)^{.5}$ $var1 = 1.08$

$SA_{VH} = var1 * SA_{H1}$

$SA_{H1} := \dfrac{SA_{VH}}{var1}$ $SA_{H1} = 1.31\ g$

Peak Spectral Acceleration for 10% damping in the horiz. direction =.91g< $SA_{H1} = 1.31\ g$

(at T=0.1 second, Ref. 2.2.12)

$T_{psa} := .1 \cdot sec$

The lowest natural frequency (f_{es}) is taken as the frequency at the Peak Spectral Acceleration for 10% damping for the displacement calculation.

$f_{es} := \dfrac{1}{T_{psa}}$ $f_{es} = 10\ \dfrac{1}{s}$

Sliding distance= $\Delta_s := \dfrac{c_s}{(2 \cdot \pi \cdot f_{es})^2}$ Ref. 2.2.8 Equation A-3

$\Delta_s = 0.14\ in$

6.6.2 SMALL MAT OVERTURNING STABILITY

The total Overturning Moment (OTM) is the summation of OTM_1 and OTM_2, which are described below:

Concrete Superstructure Story Shear and Overturning Moment (OTM_1):
The story shears are the summation of individual joint shear forces, which are derived from the product of joint accelerations and the joint assembled masses of a particular elevation from the calculation of *Initial Handling Facility (IHF): Concrete Structure Design* (Reference 2.2.5). The analysis is based on the finite element concrete superstructure under a component seismic (Ex, Ey, and Ez) response spectrum analysis. Due to the fact that the concrete superstructure response spectrum analysis has "Fixed Based" boundary conditions, the near base joint accelerations are unrealistically small. Therefore, modification of joint acceleration has been made to any acceleration, which is less than the ZPA of the DBGM-2 ground motion, to ZPA ground acceleration. Also, due to the elevated mat foundation, the west end of the superstructure lower joint masses has been deducted (set to zero) to accurately reflect the true story mass of each level of joints.

Mat foundation Story Shear and Overturning Moment (OTM_2):
There are three segments of the mat: the west segment which is 6 ft. thick (with top of concrete @ 7ft.), the middle segment which is 13ft. thick (with top of concrete @ 7ft.), and the east segment that is 6ft. thick (with top of concrete @ 0ft.). The segment shear for each of the three segments of the mat is calculated from the ZPA accelerations times the weight of each segment. The Overturning Moment (OTM_2) is equal to the segment shear times its vertical offset to the toe of the lowest mat foundation (at EL -6ft.). The geometry of the mat foundation is based on *Initial Handling Facility General Arrangement Ground Floor Plan* (Reference 2.2.14).

Restoring Moment: The total Restoring Moment (RM) is the summation of RM_1, RM_2 and RM_3, which is described below:

Exterior Concrete Superstructure Weight Restoring Moment RM_1:
The RM_1 is calculated by taking the sum of the restoring moments, which are the weights of each elevation of the joints, multiplied by its horizontal offset to the turning edge of the mat. The weight of the concrete superstructure is the weight summation of each elevation joint masses from the calculation of *Initial Handling Facility (IHF): Concrete Structure Design* (Reference 2.2.5).

Exterior Concrete Mat Weight Restoring Moment RM_2:
The RM_2 is calculated by taking the sum of the restoring moments, which are the weights of each segment of the mat, multiplied by its horizontal offset to the turning edge of the mat. The weight for each segment of the concrete is calculated based on the geometry of the mat foundation, which is based on *Initial Handling Facility General Arrangement Ground Floor Plan* (Reference 2.2.14) times its thickness and unit weight of the concrete.

Concrete Mat Superimposed Load (100% dead load + 25% live load) Restoring Moment RM_3:

The RM_3 is calculated by taking the sum of the restoring moments, which are the superimposed loads on each segment of the mat times its horizontal offset to the turning edge of the mat.

Factor of Safety:

$OTM_{Total} = OTM_1 + OTM_2$

$RM_{Total} = RM_1 + RM_2 + RM_3$

The Factor of Safety is calculated by the ratio of $\dfrac{RM_{Total}}{OTM_{Total}}$.

The following shows the individual overturning moments and restoring moment summary calculation of two load cases LC1 and LC2 based on two critical crane positions (see Reference 2.2.5 for position description). Detail calculations are shown in Excel overturning calculations (*Small Mat Overturning Position 1.xls* and *Small Mat Overturning Position 2.xls*) of Attachment G, (DVD 2 of 2).

IHF SMALL MAT OVERTURNING MOMENTS - CRANE POSITION 1

Exterior Concrete Superstructure						
Exterior Concrete: North-South (X)			Tipping depth=	-6	ft	OTM
Elevation	Story Shear	Story Force	Story Mass	Effect Accel	Level Arms	Kips-ft
ft	Kips	Kips	Kips	G's	ft	Kips-ft
58.00	5309.39	5309.4	4151.8	1.279	64.00	339801
52.50	7100.52	1791.1	1604.4	1.116	58.50	104781
47.50	8685.55	1585.0	1528.0	1.037	53.50	84799
42.50	10136.69	1451.1	1528.0	0.950	48.50	70381
37.50	11769.24	1632.5	1779.5	0.917	43.50	71016
32.50	12902.57	1133.3	1473.4	0.769	38.50	43633
27.86	13859.35	956.8	1391.0	0.688	33.86	32394
23.21	14680.08	820.7	1363.1	0.602	29.21	23977
18.57	15378.81	698.7	1335.2	0.523	24.57	17169
13.93	16004.43	625.6	1307.4	0.479	19.93	12468
9.29	16601.91	597.5	1307.4	0.457	15.29	9133
4.64	16745.54	143.6	314.3	0.457	10.64	1529
0.00	16817.36	71.8	157.1	0.457	6.00	431
			19240.6		**Sum OTM1 =**	**811511**

Exterior Concrete Mat and Superimposed Loads					(Density=	0.15	kip/cubic ft)
Exterior Concrete: North-South (X)			Tipping depth=		-6	ft	
Length	Width	Thickness	Mass+Load	ZPA Acc	Shear	Level Arms	OTM
ft	ft	ft	Kips	G's	Kips	ft	Kips-ft
59.0	75.0	6.0	4712.625	0.457	2153.67	10	21537
47.2	75.0	13.0	7487.1	0.457	3421.605	6.5	22240
35.3	75.0	6.0	2819.5875	0.457	1288.551	3	3866
			15019.313			**Sum OTM2 =**	**47643**

Sum Total Overturning = (Sum OTM1 + Sum OTM2) = 859153 foot-kips

IHF SMALL MAT RESTORING MOMENTS - Crane Position 1

Exterior Concrete Superstructure Weight - Restoring Moment				
Exterior Concrete: Vertical (Z)	Tipping arm=	37.5	ft	OTM
Elevation	Story Mass	Effect Accel	Level Arms	Kips-ft
ft	Kips	G's	ft	Kips-ft
58.00	4151.8	1.00	37.50	155693
52.50	1604.4	1.00	37.50	60165
47.50	1528.0	1.00	37.50	57300
42.50	1528.0	1.00	37.50	57300
37.50	1779.5	1.00	37.50	66730
32.50	1473.4	1.00	37.50	55253
27.86	1391.0	1.00	37.50	52162
23.21	1363.1	1.00	37.50	51117
18.57	1335.2	1.00	37.50	50071
13.93	1307.4	1.00	37.50	49026
9.29	1307.4	1.00	37.50	49026
4.64	314.3	1.00	37.50	11786
0.00	157.1	1.00	37.50	5893
	19240.6		**Sum RM1=**	**721522**

Exterior Concrete Mat weight - Restoring Moment							
					(Density=	0.15	kip/cubic ft)
Exterior Concrete: North-South (X)							
Length	Width	Thickness	Mass	Gravity Acc	Weight	Level Arms	OTM
ft	ft	ft	Kips	G's	Kips	ft	Kips-ft
59.0	75.0	6.0	3982.5	1.000	3982.5	37.5	149344
47.2	75.0	13.0	6903.0	1.000	6903.0	37.5	258863
35.3	75.0	6.0	2382.8	1.000	2382.8	37.5	89353
			13268.3			**Sum RM2=**	**497559**

Exterior Concrete Mat superimposed Load - Restoring Moment						
				(DL=140 psf, LL=100 psf)	0.165	psf
Exterior Concrete: North-South (X)					ft	
Length	width	Dead Load	0.25LL Load	Total Load	Level Arms	OTM
ft	ft	Kips	Kips	Kips	ft	Kips-ft
59.0	75.0	619.5	110.6	730.1	37.5	27380
47.2	75.0	495.6	88.5	584.1	37.5	21904
35.3	75.0	370.7	66.2	436.8	37.5	16381
				1751.1	**Sum RM3=**	**65665**

IHF SMALL MAT OVERTURNING -Crane Position 1 (continued)

Exterior Concrete Superstructure Uplift	
Exterior Concrete Superstructure Weight	19240.6
Exterior Concrete Mat Weight	13268.3
Exterior Concrete Mat Superimposed Load	1751.1
Total Structure Weight	**34259.9**

Total Restoring Moment with uplift
= (Sum of RM's) - (0.4*Vertical Acceleration*Total Structure weight)*(Level Arms)
Total Restoring Moment with uplift = (RM1 + RM2 + RM3) - (0.4*0.3194*Wtotal) * (75/2)
Total Restoring Moment with uplift = (721522 + 497559 + 65665 - 164139) = 1,120,607 ft-kips

= 1120607 / 859153

Total Restoring Moment without uplift = (RM1 + RM2 + RM3)
Total Restoring Moment without uplift = (721522 + 497559 + 65665) = 1,284,747 ft-kips

=1284746 / 859153

IHF SMALL MAT OVERTURNING MOMENTS - CRANE POSITION 2

Exterior Concrete Superstructure						
Exterior Concrete: North-South (X)			Tipping depth=	-6	ft	OTM
Elevation	Story Shear	Story Force	Story Mass	Effect Accel	Level Arms	Kips-ft
ft	Kips	Kips	Kips	G's	ft	Kips-ft
58.00	5331.87	5331.9	4151.5	1.284	64.00	341240
52.50	7124.80	1792.9	1604.3	1.118	58.50	104886
47.50	8701.79	1577.0	1527.9	1.032	53.50	84369
42.50	10136.42	1434.6	1527.9	0.939	48.50	69580
37.50	11647.76	1511.3	1779.3	0.849	43.50	65743
32.50	12762.75	1115.0	1473.3	0.757	38.50	42927
27.86	13703.33	940.6	1390.8	0.676	33.86	31845
23.21	14514.71	811.4	1363.0	0.595	29.21	23704
18.57	15207.74	693.0	1335.1	0.519	24.57	17029
13.93	15827.16	619.4	1307.2	0.474	19.93	12344
9.29	16424.57	597.4	1307.2	0.457	15.29	9132
4.64	16568.18	143.6	314.3	0.457	10.64	1528
0.00	16639.99	71.8	157.1	0.457	6.00	431
			19238.8		**Sum OTM1 =**	**804758**

Exterior Concrete Mat and Superimposed Loads					(Density=	0.15	kip/cubic ft)
Exterior Concrete: North-South (X)			Tipping depth=		-6	ft	
Length	Width	Thickness	Mass+Load	ZPA Acc	Shear	Level Arms	OTM
ft	ft	ft	Kips	G's	Kips	ft	Kips-ft
59.0	75.0	6.0	4712.625	0.457	2153.67	10	21537
47.2	75.0	13.0	7487.1	0.457	3421.605	6.5	22240
35.3	75.0	6.0	2819.588	0.457	1288.551	3	3866
			15019.31			**Sum OTM2 =**	**47643**

Sum Total Overturning = (Sum OTM1 + Sum OTM2) = 852401 foot-kips

IHF SMALL MAT RESTORING MOMENTS - Crane Position 2

Exterior Concrete Superstructure weight - Restoring Moment				
Exterior Concrete: Vertical (Z)	Tipping arm=	37.5	ft	OTM
Elevation	Story Mass	Effect Accel	Level Arms	Kips-ft
ft	Kips	G's	ft	Kips-ft
58.00	4151.5	1.00	37.50	155680
52.50	1604.3	1.00	37.50	60159
47.50	1527.9	1.00	37.50	57295
42.50	1527.9	1.00	37.50	57295
37.50	1779.3	1.00	37.50	66724
32.50	1473.3	1.00	37.50	55248
27.86	1390.8	1.00	37.50	52156
23.21	1363.0	1.00	37.50	51112
18.57	1335.1	1.00	37.50	50066
13.93	1307.2	1.00	37.50	49021
9.29	1307.2	1.00	37.50	49021
4.64	314.3	1.00	37.50	11785
0.00	157.1	1.00	37.50	5892
	19238.8		**Sum RM1=**	**721454**

Exterior Concrete Mat Weight - Restoring Moment							
					(Density=	0.15	kip/cubic ft)
Exterior Concrete: North-South (X)							
Length	Width	Thickness	Mass	Gravity Acc	Weight	Level Arms	OTM
ft	ft	ft	Kips	G's	Kips	ft	Kips-ft
59.0	75.0	6.0	3982.5	1.000	3982.5	37.5	149344
47.2	75.0	13.0	6903.0	1.000	6903.0	37.5	258863
35.3	75.0	6.0	2382.8	1.000	2382.8	37.5	89353
			13268.3			**Sum RM2=**	**497559**

Exterior Concrete Mat superimposed Load - Restoring Moment							
				(DL=140 psf, LL=100 psf)		0.165	psf
Exterior Concrete: North-South (X)						ft	
Length	width	Dead Load	0.25LL Load	Total Load	Level Arms		OTM
ft	ft	Kips	Kips	Kips	ft		Kips-ft
59.0	75.0	619.5	110.6	730.1	37.5		27380
47.2	75.0	495.6	88.5	584.1	37.5		21904
35.3	75.0	370.7	66.2	436.8	37.5		16381
				1751.1		**Sum RM3=**	**65665**

IHF SMALL MAT OVERTURNING - Crane Position 2 (continued)

Exterior Concrete Superstructure Uplift	
Exterior Concrete Superstructure Weight	19238.8
Exterior Concrete Mat Weight	13268.3
Exterior Concrete Mat Superimposed Load	1751.1
Total Structure Weight	**34258.1**

Total Restoring Moment with uplift
= (Sum of RM's) - (0.4*Vertical Acceleration*Total Structure weight)*(Level Arms)
Total Restoring Moment with uplift = (RM1 + RM2 + RM3) - (0.4*0.3194*Wtotal)* (75/2)
Total Restoring Moment with uplift = (721454 + 497559 + 65665 - 164130) = 1,120,548 ft-kips

=1120548 / 852401

Total Restoring Moment without uplift = (RM1 + RM2 + RM3)
Total Restoring Moment without uplift = (721454 + 497559 + 65665) = 1,284,679 ft-kips

=1284679 / 852401

6.6.3 LARGE MAT STABILITY AGAINST SLIDING

The large mat dimensions are:

$L_{found} := 196.5 \cdot ft$ length of foundation

$t_{foun} := 6 \cdot ft$ Thickness of foundation mat

$W_{found} := 170 \cdot ft$ width of foundation

$w_{con} := 0.15 \frac{k}{ft^3}$ weight of concrete

Total weight of foundation= $wt_{found} := L_{found} \cdot W_{found} \cdot t_{foun} \cdot w_{con}$

$wt_{found} = 30064.5\ k$

Applied forces=

Weight of above grade concrete structure= $wt_{concstr} := 8456.0 \cdot k$ Ref. Attach. F, Folder: Interior Concrete Base Reaction, Excel File: BaseReactions 4-8 from *Renumbered Model* (cell 4/E)

Weight of steel structure= $wt_{stl} := 9281.7 \cdot k$ Ref. Attach. F, Folder: Steel Base Reaction, Excel File: CLC1 SAP2000 Output Data from *reaction Loads summary* (cell 4/E)

Dead weight of crane= $wt_{dcr} := 1539.9 \cdot k$ Ref. Attach. F, Folder: Steel Base Reaction, Excel File: CLC1 SAP2000 Output Data from *reaction Loads summary* (cell 15/E)

Applied seismic force

Seismic (steel struct.) force x direction= $P_{ssx} := 11787.2 \cdot k$ Ref. Attach. F, Folder: Steel Base Reaction, Excel File: CLC1 SAP2000 Output Data from *reaction Loads summary* (cell 20/C)

Seismic (steel struct.) force y direction= $P_{ssy} := 13093.4 \cdot k$ Ref. Attach. F, Folder: Steel Base Reaction, Excel File: CLC1 SAP2000 Output Data from *reaction Loads summary* (cell 21/D)

Seismic (concrete struct.) force x direction $P_{csx} := 5446.3 \cdot k$ Ref. Attach. F, Folder: Interior Concrete Base Reaction, Excel File: BaseReactions 4-8 from *Renumbered Model* (cell 7/C)

Seismic (concrete struct.) force y direction $P_{csy} := 4076.6 \cdot k$ Ref. Attach. F, Folder: Interior Concrete Base Reaction, Excel File: BaseReactions 4-8 from *Renumbered Model* (cell 8/D)

$ZPA_H := 0.45$ zero period ground acceleration, Ref. 2.2.12

Seismic (base mat) force x direction $PH1_{lmx} := ZPA_H \cdot wt_{found}$ $PH1_{lmx} = 13529\,k$

Seismic (base mat) force y direction $PH1_{lmy} := ZPA_H \cdot wt_{found}$ $PH1_{lmy} = 13529\,k$

Total applied seismic force=

$$f_{seiappl} := \left[\left(P_{ssx} + P_{csx} + PH1_{lmx}\right)^2 + \left(P_{ssy} + P_{csy} + PH1_{lmy}\right)^2\right]^{.5}$$

$$f_{seiappl} = 43459.9\,k$$

Total resisting force

Vertical resisting seismic uplift $up_{seismicup} := 10083.5k$ Ref. Attach. F, Folder: Steel Base Reaction, Excel File: CLC1 SAP2000 Output Data from *reaction Loads summary* (cell 22/E)

coefficient of friction= $\mu := 0.81$ Ref. 2.2.12, Table 6-2

$$P_{resis} := \mu \cdot \left(wt_{found} + wt_{concstr} + wt_{stl} + wt_{dcr} - up_{seismicup}\right) \qquad P_{resis} = 31799.5\,k$$

Factor of Safety Against Sliding= $FS_{lm} := \dfrac{P_{resis}}{f_{seiappl}}$ $FS_{lm} = 0.73$

This calculation indicates the structure will slide, therefore the energy method must be used to determine the maximum sliding displacement. (Ref. 2.2.8)

Determine Sliding Displacement (from Appendix A, Ref. 2.2.8) $ZPA_V := 0.3194$

effective coefficient of friction= $\mu_{e.large} := \mu \cdot \left(1 - .4 \cdot ZPA_V\right)$ $\mu_{e.large} = 0.71$ Ref. 2.2.8 Equation A-1

sliding coefficient= $c_{s.large} := 2 \cdot \mu_{e.large} \cdot g$ $c_{s.large} = 1.41\,g$ Ref. 2.2.8 Equation A-2

f_{es} = The lowest natural frequency at which the horizontal 10% damped vertical spectral acceleration equals c_s

SA_{H1} = The 10% damped spectral acceration for horizontal direction 1

SA_{H2} = The 10% damped spectral acceration for horizontal direction 2

$SA_{H1}=SA_{H2}$

$SA_{VH.large} := c_{s.large}$ $SA_{VH.large} = 1.41\,g$

$SA_{VHlarge} = (1+.16)^{1/2} * SA_{H1}$ (Ref. 2.2.8, Eq. A-4) $var1_{large} := (1 + .16)^{.5}$ $var1_{large} = 1.08$

$SA_{VHlarge} = var1_{large} * SA_{H1large}$

$$SA_{H1.large} := \frac{SA_{VH.large}}{var1_{large}} \qquad SA_{H1.large} = 1.31\,g$$

Peak Spectral Acceleration for 10% damping in the horiz. direction =.91g < $SA_{H1.large} = 1.31\,g$

(at T=0.1 second, Ref. 2.2.12)

$$T_{psa.large} := .1 \cdot sec$$

The lowest natural frequency (f_{es}) is taken as the frequency at Peak Spectral Acceleration for 10% damping for the displacement calculation.

$$f_{es.large} := \frac{1}{T_{psa.large}} \qquad f_{es.large} = 10\,\frac{1}{s}$$

Sliding distance=

$$\Delta_{s.large} := \frac{c_{s.large}}{\left(2 \cdot \pi \cdot f_{es.large}\right)^2}$$

Ref. 2.2.8
Equation A-3

$$\Delta_{s.large} = 0.14\,in$$

6.6.4 LARGE MAT OVERTURNING STABILITY USING APPROACH A

Static Overturning of the Combined Structures (Steel + Concrete + Mat):

The overturning moment is calculated based on the summation of individual joint forces generated from seismic ground motion multiplied by each force's corresponding moment arm. The IHF building is comprised of an Interior Concrete structure and a braced frame steel structure. The steel structure's primary function is to handle waste containers and/or canisters by means of large industrial capacity overhead cranes. Due to the various crane positions; the structure was analyzed for 7 differing Crane Load Cases.

In order to determine the worst-case scenario, in overturning, the following conditions were considered:

- A review of all 7 Crane Load Cases found that Crane Load Case 1 loads the steel structure towards the Northwest corner of the building (overturning is taken about the Northwest corner).
- The Interior Concrete Structure center of gravity is located towards the Northwest corner.
- The Steel Structure center of gravity is approximately at the geometric center of the mat.
- The Mat foundation center of gravity is approximately at the geometric center of the mat.

After reviewing the impact of the Crane Load Cases, the Interior Concrete structure, the Steel structure and the Mat foundation centers of gravity, and based on sound engineering judgment, Crane Load Case 1 will yield the worst-case overturning scenario (largest overturning moments with the lowest resisting moment = lowest factor of safety against overturning).

Total Overturning Moment: (The total Overturning Moment (OTM) is the summation of OTM_1, OTM_2, & OTM_3, which are described below)

Steel Structure Overturning Moment (OTM_1):
The joint forces are derived from the product of joint accelerations and joint assembled masses for all locations, and the data was extracted from the SAP2000 finite element model for Crane Load Case 1 of the calculation of *Initial Handling Facility (IHF) Steel Structure Seismic Analysis and Steel Member Design* (Reference 2.2.3) as Seismic components Ex, Ey, and Ez.

Determine the location of the center of gravity for the steel structure:
Step 1) Multiply each Joint Force by the Joint Coordinate dimension of that particular joint (in each of the 3-directions (x, y and z).
Step 2) Sum up all values of Joint Force times Joint coordinate dimensions
Step 3) Sum up all values of Joint Forces in each of the 3-directions (x, y, and z)
Step 4) Divide the Summed Total value obtained in Step 2) above, by the Summed Total value obtained in Step 3) above

According to the above process, one can determine the center of gravity for the entire steel building. One can then determine the Overturning Moments due to Seismic Components Ex, Ey and Ez, in either direction, Mxx and Myy.

(Mxx = overturning moment about the X-axis and Myy = overturning moment about the Y-axes).

The overturning moment for the steel structure was taken about the base of the mat foundation to accurately reflect the true overturning moment of the building.

Interior Concrete structure Joint Forces and Overturning Moment (OTM_2):
The overturning moment for the Interior Concrete Superstructure was derived following the same process as described for the Steel Structure but from the calculation of *Initial Handling Facility (IHF): Concrete Structure Design* (Reference 2.2.5). Refer to OTM_2.

Mat foundation Forces and Overturning Moment (OTM_3):
The entire mat is 6 ft. thick. The shear force for the mat foundation is calculated from the ZPA (vertical and horizontal accelerations) multiplied by its weight. The Overturning Moment (OTM_3) is then equal to this shear force multiplied by the mat's horizontal/vertical offsets (moment arms). The geometry of the mat foundation is based on *Initial Handling Facility General Arrangement Ground Floor Plan* (Reference 2.2.14).

Total Resisting Moment: (The total Resisting Moment (RM) is the summation of RM_1, RM_2, and RM_3 which follow)

Steel Structure Weight Resisting Moment RM_1:
RM_1 is calculated by taking the sum of the resisting moments, which are the steel member's total dead weight multiplied by its horizontal offset taken about the X and Y-axes.

Interior Concrete Structure Resisting Moment RM_2:
RM_2 is calculated by multiplying the weight of the entire structure by its horizontal offset taken about the X and Y-axes. The weight of the concrete superstructure is the weight summation of each elevation joint masses from the calculation of *Initial Handling Facility (IHF): Concrete Structure Design* (Reference 2.2.5).

Concrete Mat Weight In Addition to the Superimposed Loads (100% dead load + 25% live load) Resisting Moment RM_3:
RM_3 is calculated by taking the sum of the resisting moments, which is the self-weight of the concrete mat along with the superimposed loads multiplied by their horizontal offset taken about the X- and Y-axes.
Factor of Safety:
The Factor of Safety is calculated by the $\frac{\sum RM_i}{\sum OTM_i}$.

Values shown in the following subsections were taken from SAP2000 analysis output of the interior concrete structure, steel structure, and mat foundation. The excel spreadsheets are located at Attachment F/Large mat OTM - Approach A.

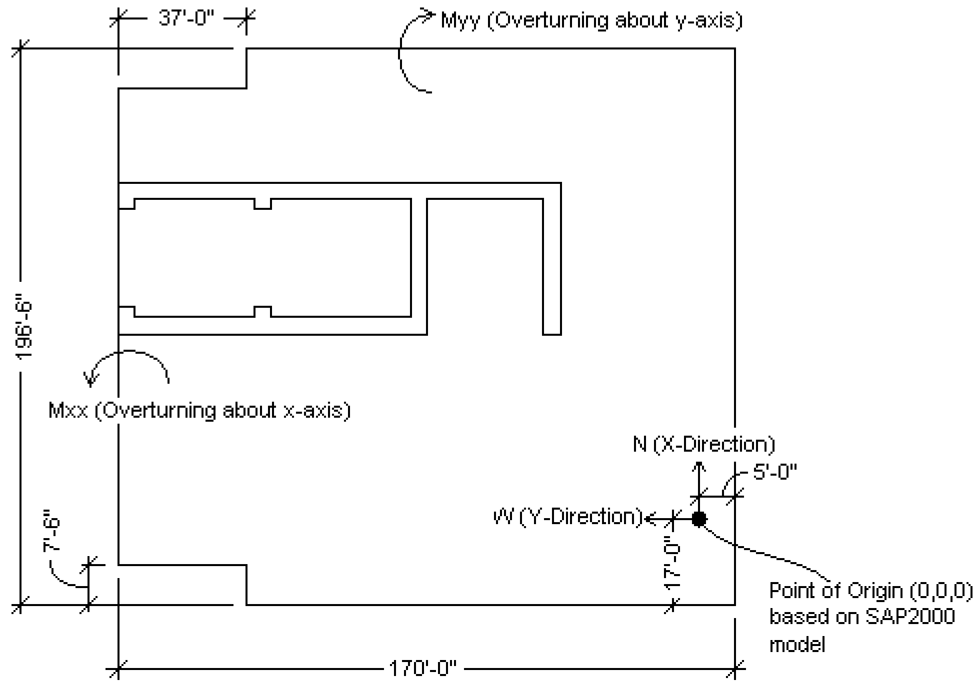


Figure 6.6.1: Large Mat Plan View

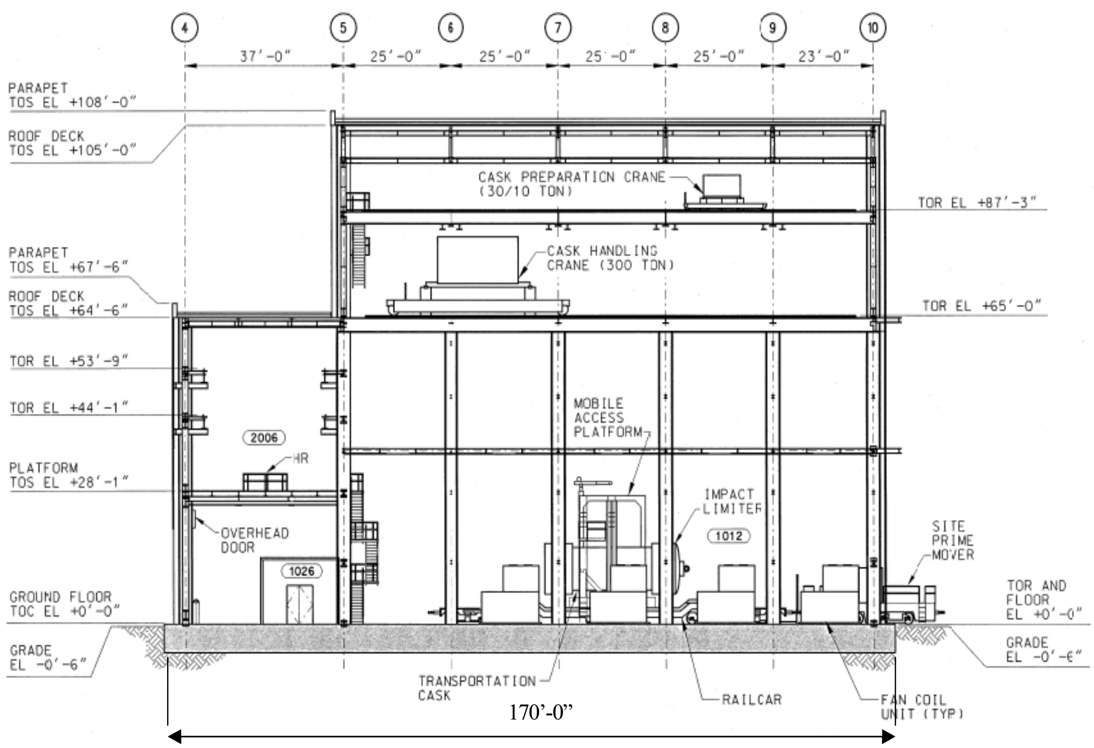


Figure 6.6.2: Large Mat Elevation View (from Section Cut D of Ref. 2.2.17)

6.6.4.1 INTERIOR CONCRETE STRUCTURE:

Note: The centers of gravity are determined according to the 'Point of Origin' (see Figure 6.6.1) and later shall be resolved into each corresponding moment arm in order to calculate the overturning and restoring moments. Values were taken from the Excel spreadsheet entitled 'Concrete Structure - Seismic EX, EY, & EZ.xls'. To the side of each value is its Excel cell location. For instance, (A10) would be located in 'Concrete Structure - Seismic EX, EY, & EZ.xls' column A, row 10.

Forces and centers of gravity for seismic ground motion on the interior concrete structure in the X-direction:

$EX_{x\_comp.conc} := 5446.3kip$ (E640) $CG_{y.x\_comp.EX.conc} := 110.74ft$ (T641) $CG_{z.x\_comp.EX.conc} := 25.82ft$ (U641)

$EX_{y\_comp.conc} := 1727.4kip$ (F640) $CG_{x.y\_comp.EX.conc} := 104.28ft$ (W641) $CG_{z.y\_comp.EX.conc} := 24.93ft$ (V641)

$EX_{z\_comp.conc} := 1552.9kip$ (G640) $CG_{x.z\_comp.EX.conc} := 104.46ft$ (X641) $CG_{y.z\_comp.EX.conc} := 105.36ft$ (Y641)

Forces and centers of gravity for seismic ground motion on the interior concrete structure in the Y-direction:

$EY_{x\_comp.conc} := 2639.6kip$ (E1286) $CG_{y.x\_comp.EY.conc} := 103.5ft$ (T1287) $CG_{z.x\_comp.EY.conc} := 23.76ft$ (U1287)

$EY_{y\_comp.conc} := 4076.6kip$ (F1286) $CG_{x.y\_comp.EY.conc} := 104.23ft$ (W1287) $CG_{z.y\_comp.EY.conc} := 25.77ft$ (V1287)

$EY_{z\_comp.conc} := 2114.8kip$ (G1286) $CG_{x.z\_comp.EY.conc} := 104.39ft$ (X1287) $CG_{y.z\_comp.EY.conc} := 103.38ft$ (Y1287)

Forces and centers of gravity for seismic ground motion on the interior concrete structure in the Z-direction:

$EZ_{x\_comp.conc} := 1861.6kip$ (E1932) $CG_{y.x\_comp.EZ.conc} := 105.26ft$ (T1933) $CG_{z.x\_comp.EZ.conc} := 21.29ft$ (U1933)

$EZ_{y\_comp.conc} := 1626.3kip$ (F1932) $CG_{x.y\_comp.EZ.conc} := 104.03ft$ (W1933) $CG_{z.y\_comp.EZ.conc} := 23.17ft$ (V1933)

$EZ_{z\_comp.conc} := 2836.3kip$ (G1932) $CG_{x.z\_comp.EZ.conc} := 104.84ft$ (X1933) $CG_{y.z\_comp.EZ.conc} := 99.68ft$ (Y1933)

Dead & live loads, including center of gravity in both the X- and Y-directions for the interior concrete structure. These values are taken from the Excel spreadsheet entitled 'Dead Load & Live Load Interior Concrete Structure.xls':

$DL_{conc} := 8730.3kip$ (V71) $CG_{x.conc.DL} := 105.6ft$ (Y72) $CG_{y.conc.DL} := 100.05ft$ (Z72)

$LL_{25\%.conc} := 95.2kip$ (V148) $CG_{x.conc.25\%LL} := 104.54ft$ (Y149) $CG_{y.conc.25\%LL} := 102.57ft$ (Z149)

6.6.4.2 STEEL STRUCTURE:

Forces and centers of gravity for seismic ground motion on the steel structure in the X-direction, taken from the Excel spreadsheet entitled 'Steel Structure - Seismic EX, EY, & EZ.xls':

$EX_{x\_comp.steel} := 11787.2kip$ (Q1610) $CG_{y.x\_comp.EX.steel} := 77.93ft$ (Y1611) $CG_{z.x\_comp.EX.steel} := 66.52ft$ (Z1611)

$EX_{y\_comp.steel} := 3589.4kip$ (R1610) $CG_{x.y\_comp.EX.steel} := 80.44ft$ (AB1611) $CG_{z.y\_comp.EX.steel} := 60.91ft$ (AA1611)

$EX_{z\_comp.steel} := 2793.6kip$ (S1610) $CG_{x.z\_comp.EX.steel} := 82.47ft$ (AC1611) $CG_{y.z\_comp.EX.steel} := 83.91ft$ (AD1611)

Forces and centers of gravity for seismic ground motion on the steel structure in the Y-direction:

$EY_{x\_comp.steel} := 4531.7kip$ (Q3227) $CG_{y.x\_comp.EY.steel} := 81.53ft$ (Y3228) $CG_{z.x\_comp.EY.steel} := 59.01ft$ (Z3228)

$EY_{y\_comp.steel} := 13093.4kip$ (R3227) $CG_{x.y\_comp.EY.steel} := 78.62ft$ (AB3228) $CG_{z.y\_comp.EY.steel} := 66.79ft$ (AA3228)

$EY_{z\_comp.steel} := 3901.9kip$ (S3227) $CG_{x.z\_comp.EY.steel} := 89.56ft$ (AC3228) $CG_{y.z\_comp.EY.steel} := 87.61ft$ (AD3228)

Forces and centers of gravity for seismic ground motion on the steel structure in the Z-direction:

$EZ_{x\_comp.steel} := 5611.2kip$ (Q4843) $CG_{y.x\_comp.EZ.steel} := 72.17ft$ (Y4844) $CG_{z.x\_comp.EZ.steel} := 58.16ft$ (Z4844)

$EZ_{y\_comp.steel} := 5414.2kip$ (R4843) $CG_{x.y\_comp.EZ.steel} := 81.78ft$ (AB4844) $CG_{z.y\_comp.EZ.steel} := 60.67ft$ (AA4844)

$EZ_{z\_comp.steel} := 10083.5kip$ (S4843) $CG_{x.z\_comp.EZ.steel} := 87.6ft$ (AC4844) $CG_{y.z\_comp.EZ.steel} := 83.99ft$ (AD4844)

Dead & live loads, and the center of gravity in both the X- and Y-directions for the steel structure, which were taken from the Excel file entitled 'Dead Load & Live Load Steel Structure.xls':

$DL_{steel} := 14582.6kip$ (G90) $CG_{x.steel.DL} := 86.63ft$ (O91) $CG_{y.steel.DL} := 80.11ft$ (P91)

$LL_{25\%.steel} := 770.5kip$ (K90) $CG_{x.steel.25\%LL} := 84ft$ (P91) $CG_{y.steel.25\%LL} := 95.99ft$ (Q91)

6.6.4.3 MAT FOUNDATION:

Seismic loads due to ground motions on the foundation mat:

Mat area self weight,

$\sigma_{mat} := .15 \frac{kip}{ft^3} \cdot 6ft$ $\sigma_{mat} = 0.9 \frac{kip}{ft^2}$

The uniform superimposed dead & live loads applied on the Mat (Section 4.3.2.1 of this calculation) are,

$\sigma_{uniform.DL.mat} := 0.14 \frac{kip}{ft^2}$ $\sigma_{uniform.LL.mat} := 0.1 \frac{kip}{ft^2}$ (Assumption 3.1.2)

The horizontal ground motion acceleration for DBGM-2 is 0.4537g (Reference 2.2.12, Table 6-5), while the vertical ground motion acceleration for DBGM-2 is 0.3194g (Reference 2.2.12, Table 6-6)

$acc_{horiz} := 0.4537\ g$ $acc_{vert} := 0.3194\ g$

The area of the foundation mat:

$area_{mat} := (170ft \cdot 196.5ft) - 2(37ft \cdot 7.5ft)$ $area_{mat} = 32850\ ft^2$

The Mat self weight and superimposed dead load plus 25% live load is,

$Wt_{mat} := area_{mat} \cdot \left[\left(\sigma_{mat} + \sigma_{uniform.DL.mat} \right) + \left(0.25 \cdot \sigma_{uniform.LL.mat} \right) \right]$ $Wt_{mat} = 34985.3\ kip$

The horizontal loads applied on the large mat in both the X- & Y-directions due to seismic ground motion are:

$F_{xx.mat} := Wt_{mat} \cdot acc_{horiz}$ $F_{xx.mat} = 15872.8\ kip$ $CG_{z.x\_comp.EX.mat} := 3ft$ (Half the mat thickness)

$F_{yy.mat} := F_{xx.mat}$ $F_{yy.mat} = 15872.8\ kip$ $CG_{z.y\_comp.EY.mat} := 3ft$ (Half the mat thickness)

The vertical load applied on the Large Mat in the Z-direction due to seismic is,

$F_{zz.mat} := Wt_{mat} \cdot acc_{vert}$ $F_{zz.mat} = 11174.3\ kip$

The center of gravity of the mat foundation in the X-direction is located in the center of the mat (the mat in this direction is symmetrical); therefore:

$CG_{xx.mat} := \frac{196.5ft}{2}$ $CG_{xx.mat} = 98.3\ ft$ (Distance from the north face of the mat foundation)

The center of gravity of the mat in the Y-direction is calculated as follows:

$A_1 := 37ft\cdot(196.5ft - 2\cdot 7.5ft)$ $A_1 = 6715.5\ ft^2$

$A_2 := (170ft - 37ft)\cdot 196.5ft$ $A_2 = 26134.5\ ft^2$

$CG_{yy.mat} := \dfrac{\left(A_1\cdot\dfrac{37ft}{2}\right) + A_2\cdot\left(37ft + \dfrac{170ft - 37ft}{2}\right)}{A_1 + A_2}$ $CG_{yy.mat} = 86.1\ ft$ (Distance from the west face of the mat foundation)

6.6.4.4 OVERTURNING MOMENT ABOUT THE X-AXIS: **(Overturning is taken about the point of origin (x, y) and at the base of the mat (z). The seismic components will be combined using the '100-40-40' method.)**

A. Interior concrete structure: (0.4EX+EY+0.4EZ)

$OTM_{xx.conc.major.y} := EY_{y\_comp.conc}\cdot\left(CG_{z.y\_comp.EY.conc} + 6ft\right)$

$OTM_{xx.conc.major.y} = 1.295 \times 10^5\ ft\cdot kip$

$OTM_{xx.z\_comp.conc.major.y} := EY_{z\_comp.conc}\cdot\left(170ft - 5ft - CG_{y.z\_comp.EY.conc}\right)$

$OTM_{xx.z\_comp.conc.major.y} = 1.303 \times 10^5\ ft\cdot kip$

$OTM_{xx.conc.minor.x} := 0.4\cdot\left[EX_{y\_comp.conc}\cdot\left(CG_{z.y\_comp.EX.conc} + 6ft\right)\right]$

$OTM_{xx.conc.minor.x} = 2.137 \times 10^4\ ft\cdot kip$

$OTM_{xx.z\_comp.conc.minor.x} := 0.4\cdot EX_{z\_comp.conc}\cdot\left(170\cdot ft - 5\cdot ft - CG_{y.z\_comp.EX.conc}\right)$

$OTM_{xx.z\_comp.conc.minor.x} = 3.705 \times 10^4\ ft\cdot kip$

$OTM_{xx.conc.minor.z} := 0.4\cdot\left[EZ_{y\_comp.conc}\cdot\left(CG_{z.y\_comp.EZ.conc} + 6ft\right)\right]$

$OTM_{xx.conc.minor.z} = 1.898 \times 10^4\ ft\cdot kip$

$OTM_{xx.z\_comp.conc.minor.z} := 0.4\cdot EZ_{z\_comp.conc}\cdot\left(170\cdot ft - 5\cdot ft - CG_{y.z\_comp.EZ.conc}\right)$

$OTM_{xx.z\_comp.conc.minor.z} = 7.411 \times 10^4\ ft\cdot kip$

$OTM_{xx.conc.total} := OTM_{xx.conc.major.y} + OTM_{xx.conc.minor.x} + OTM_{xx.conc.minor.z}$

$OTM_{xx.conc.total} = 1.699 \times 10^{5}\ ft \cdot kip$

$OTM_{x\_axis.z\_comp.conc.total} := OTM_{xx.z\_comp.conc.major.y} + OTM_{xx.z\_comp.conc.minor.x} + OTM_{xx.z\_comp.conc.minor.z}$

$OTM_{x\_axis.z\_comp.conc.total} = 2.415 \times 10^{5}\ ft \cdot kip$

B. Steel structure:

$OTM_{xx.steel.major.y} := EY_{y\_comp.steel} \cdot \left(CG_{z.y\_comp.EY.steel} + 6ft\right)$

$OTM_{xx.steel.major.y} = 9.531 \times 10^{5}\ ft \cdot kip$

$OTM_{xx.z\_comp.steel.major.y} := EY_{z\_comp.steel} \cdot \left(170ft - 5ft - CG_{y.z\_comp.EY.steel}\right)$

$OTM_{xx.z\_comp.steel.major.y} = 3.02 \times 10^{5}\ ft \cdot kip$

$OTM_{xx.steel.minor.x} := 0.4 \cdot \left[EX_{y\_comp.steel} \cdot \left(CG_{z.y\_comp.EX.steel} + 6ft\right)\right]$

$OTM_{xx.steel.minor.x} = 9.607 \times 10^{4}\ ft \cdot kip$

$OTM_{xx.z\_comp.steel.minor.x} := 0.4 \cdot EX_{z\_comp.steel} \cdot \left(170 \cdot ft - 5 \cdot ft - CG_{y.z\_comp.EX.steel}\right)$

$OTM_{xx.z\_comp.steel.minor.x} = 9.061 \times 10^{4}\ ft \cdot kip$

$OTM_{xx.steel.minor.z} := 0.4 \cdot \left[EZ_{y\_comp.steel} \cdot \left(CG_{z.y\_comp.EZ.steel} + 6ft\right)\right]$

$OTM_{xx.steel.minor.z} = 1.444 \times 10^{5}\ ft \cdot kip$

$OTM_{xx.z\_comp.steel.minor.z} := 0.4 \cdot EZ_{z\_comp.steel} \cdot \left(170 \cdot ft - 5 \cdot ft - CG_{y.z\_comp.EZ.steel}\right)$

$OTM_{xx.z\_comp.steel.minor.z} = 3.267 \times 10^{5}\ ft \cdot kip$

$OTM_{xx.steel.total} := OTM_{xx.steel.major.y} + OTM_{xx.steel.minor.x} + OTM_{xx.steel.minor.z}$

$OTM_{xx.steel.total} = 1.194 \times 10^{6}\ ft \cdot kip$

$OTM_{x\_axis.z\_comp.steel.total} := OTM_{xx.z\_comp.steel.major.y} + OTM_{xx.z\_comp.steel.minor.x} + OTM_{xx.z\_comp.steel.minor.z}$

$OTM_{x\_axis.z\_comp.steel.total} = 7.193 \times 10^{5}\ ft \cdot kip$

C. Mat foundation:

$OTM_{xx.mat.total} := \left(F_{yy.mat} \cdot CG_{z.y\_comp.EY.mat}\right)$

$OTM_{xx.mat.total} = 4.762 \times 10^{4}\ ft \cdot kip$

$OTM_{x\_axis.z\_comp.mat.total} := 0.4\left(F_{zz.mat} \cdot CG_{yy.mat}\right)$

$OTM_{x\_axis.z\_comp.mat.total} = 3.849 \times 10^{5}\ ft \cdot kip$

$OTM_{xx.total} := OTM_{xx.conc.total} + OTM_{xx.steel.total} + OTM_{xx.mat.total}$ $OTM_{xx.total} = 1.411 \times 10^{6}\ ft \cdot kip$

$OTM_{xx.z\_comp.total} := OTM_{x\_axis.z\_comp.conc.total} + OTM_{x\_axis.z\_comp.steel.total} + OTM_{x\_axis.z\_comp.mat.total}$

$OTM_{xx.z\_comp.total} = 1.346 \times 10^{6}\ ft \cdot kip$

$OTM_{XX.grand.total} := OTM_{xx.total} + OTM_{xx.z\_comp.total}$ $OTM_{XX.grand.total} = 2.757 \times 10^{6}\ ft \cdot kip$

6.6.4.5 RESISTING MOMENT ABOUT THE X-AXIS:

A. Interior concrete structure:

$RM_{xx.conc.total} := DL_{conc} \cdot \left(170ft - 5ft - CG_{y.conc.DL}\right) + LL_{25\%.conc} \cdot \left(170ft - 5ft - CG_{y.conc.25\%LL}\right)$

$RM_{xx.conc.total} = 5.73 \times 10^{5}\ ft \cdot kip$

B. Steel structure:

$RM_{xx.steel.total} := DL_{steel} \cdot \left(170ft - 5ft - CG_{y.steel.DL}\right) + LL_{25\%.steel} \cdot \left(170ft - 5ft - CG_{y.steel.25\%LL}\right)$

$RM_{xx.steel.total} = 1.291 \times 10^{6}\ ft \cdot kip$

C. Mat foundation:

$RM_{xx.mat.total} := Wt_{mat} \cdot CG_{yy.mat}$ $RM_{xx.mat.total} = 3.013 \times 10^{6}\ ft \cdot kip$

$RM_{xx.grand.total} := RM_{xx.conc.total} + RM_{xx.steel.total} + RM_{xx.mat.total}$ $RM_{xx.grand.total} = 4.877 \times 10^{6}\ ft \cdot kip$

6.6.4.6 FACTOR OF SAFETY FOR LARGE MAT OVERTURNING ABOUT THE X-AXIS:

$$FS_{xx} := \frac{RM_{xx.grand.total} - OTM_{xx.z\_comp.total}}{OTM_{XX.grand.total} - OTM_{xx.z\_comp.total}}$$

6.6.4.7 OVERTURNING MOMENT ABOUT THE Y-AXIS:

A. Interior concrete structure: (EX+0.4EY+0.4EZ)

$$OTM_{yy.conc.major.x} := EX_{x\_comp.conc} \cdot \left(CG_{z.x\_comp.EX.conc} + 6ft\right)$$

$$OTM_{yy.conc.major.x} = 1.733 \times 10^{5}\, ft \cdot kip$$

$$OTM_{yy.z\_comp.conc.major.x} := EX_{z\_comp.conc} \cdot \left(196.5ft - 17ft - CG_{x.z\_comp.EX.conc}\right)$$

$$OTM_{yy.z\_comp.conc.major.x} = 1.165 \times 10^{5}\, ft \cdot kip$$

$$OTM_{yy.conc.minor.y} := 0.4 \cdot \left[EY_{x\_comp.conc} \cdot \left(CG_{z.x\_comp.EY.conc} + 6ft\right)\right]$$

$$OTM_{yy.conc.minor.y} = 3.142 \times 10^{4}\, ft \cdot kip$$

$$OTM_{yy.z\_comp.conc.minor.y} := 0.4 \cdot EY_{z\_comp.conc} \cdot \left(196.5 \cdot ft - 17 \cdot ft - CG_{x.z\_comp.EY.conc}\right)$$

$$OTM_{yy.z\_comp.conc.minor.y} = 6.354 \times 10^{4}\, ft \cdot kip$$

$$OTM_{yy.conc.minor.z} := 0.4 \cdot \left[EZ_{x\_comp.conc} \cdot \left(CG_{z.x\_comp.EZ.conc} + 6ft\right)\right]$$

$$OTM_{yy.conc.minor.z} = 2.032 \times 10^{4}\, ft \cdot kip$$

$$OTM_{yy.z\_comp.conc.minor.z} := 0.4 \cdot EZ_{z\_comp.conc} \cdot \left(196.5 \cdot ft - 17 \cdot ft - CG_{x.z\_comp.EZ.conc}\right)$$

$$OTM_{yy.z\_comp.conc.minor.z} = 8.47 \times 10^{4}\, ft \cdot kip$$

$$OTM_{yy.conc.total} := OTM_{yy.conc.major.x} + OTM_{yy.conc.minor.y} + OTM_{yy.conc.minor.z}$$

$$OTM_{yy.conc.total} = 2.25 \times 10^{5}\, ft \cdot kip$$

$$OTM_{y\_axis.z\_comp.conc.total} := OTM_{yy.z\_comp.conc.major.x} + OTM_{yy.z\_comp.conc.minor.y} + OTM_{yy.z\_comp.conc.minor.z}$$

$$OTM_{y\_axis.z\_comp.conc.total} = 2.648 \times 10^{5}\, ft \cdot kip$$

B. Steel structure:

$$OTM_{yy.steel.major.x} := EX_{x\_comp.steel} \cdot (CG_{z.x\_comp.EX.steel} + 6ft)$$

$$OTM_{yy.steel.major.x} = 8.548 \times 10^{5}\ ft \cdot kip$$

$$OTM_{yy.z\_comp.steel.major.x} := EX_{z\_comp.steel} \cdot (196.5ft - 17ft - CG_{x.z\_comp.EX.steel})$$

$$OTM_{yy.z\_comp.steel.major.x} = 2.711 \times 10^{5}\ ft \cdot kip$$

$$OTM_{yy.steel.minor.y} := 0.4 \cdot \left[EY_{x\_comp.steel} \cdot (CG_{z.x\_comp.EY.steel} + 6ft)\right]$$

$$OTM_{yy.steel.minor.y} = 1.178 \times 10^{5}\ ft \cdot kip$$

$$OTM_{yy.z\_comp.steel.minor.y} := 0.4 \cdot EY_{z\_comp.steel} \cdot (196.5ft - 17ft - CG_{x.z\_comp.EY.steel})$$

$$OTM_{yy.z\_comp.steel.minor.y} = 1.404 \times 10^{5}\ ft \cdot kip$$

$$OTM_{yy.steel.minor.z} := 0.4 \cdot \left[EZ_{y\_comp.steel} \cdot (CG_{z.y\_comp.EZ.steel} + 6ft)\right]$$

$$OTM_{yy.steel.minor.z} = 1.444 \times 10^{5}\ ft \cdot kip$$

$$OTM_{yy.z\_comp.steel.minor.z} := 0.4 \cdot EZ_{z\_comp.steel} \cdot (196.5ft - 17ft - CG_{x.z\_comp.EZ.steel})$$

$$OTM_{yy.z\_comp.steel.minor.z} = 3.707 \times 10^{5}\ ft \cdot kip$$

$$OTM_{yy.steel.total} := OTM_{yy.steel.major.x} + OTM_{yy.steel.minor.y} + OTM_{yy.steel.minor.z}$$

$$OTM_{yy.steel.total} = 1.117 \times 10^{6}\ ft \cdot kip$$

$$OTM_{y\_axis.z\_comp.steel.total} := OTM_{yy.z\_comp.steel.major.x} + OTM_{yy.z\_comp.steel.minor.y} + OTM_{yy.z\_comp.steel.minor.z}$$

$$OTM_{y\_axis.z\_comp.steel.total} = 7.821 \times 10^{5}\ ft \cdot kip$$

C. Mat foundation:

$OTM_{yy.mat.total} := (F_{xx.mat} \cdot CG_{z.x\_comp.EX.mat})$

$OTM_{yy.mat.total} = 4.762 \times 10^{4}\ ft \cdot kip$

$OTM_{y\_axis.z\_comp.mat.total} := 0.4(F_{zz.mat} \cdot CG_{xx.mat})$

$OTM_{y\_axis.z\_comp.mat.total} = 4.391 \times 10^{5}\ ft \cdot kip$

$OTM_{yy.total} := OTM_{yy.conc.total} + OTM_{yy.steel.total} + OTM_{yy.mat.total}$ $OTM_{yy.total} = 1.39 \times 10^{6}\ ft \cdot kip$

$OTM_{yy.z\_comp.total} := OTM_{y\_axis.z\_comp.conc.total} + OTM_{y\_axis.z\_comp.steel.total} + OTM_{y\_axis.z\_comp.mat.total}$

$OTM_{yy.z\_comp.total} = 1.486 \times 10^{6}\ ft \cdot kip$

$OTM_{YY.grand.total} := OTM_{yy.total} + OTM_{yy.z\_comp.total}$ $OTM_{YY.grand.total} = 2.876 \times 10^{6}\ ft \cdot kip$

6.6.4.8 RESISTING MOMENT ABOUT THE X-AXIS:

A. Interior concrete structure:

$RM_{yy.conc.total} := DL_{conc} \cdot (196.5ft - 17ft - CG_{x.conc.DL}) + LL_{25\%.conc} \cdot (196.5ft - 17ft - CG_{x.conc.25\%LL})$

$RM_{yy.conc.total} = 6.523 \times 10^{5}\ ft \cdot kip$

B. Steel structure:

$RM_{yy.steel.total} := DL_{steel} \cdot (196.5ft - 17ft - CG_{x.steel.DL}) + LL_{25\%.steel} \cdot (196.5ft - 17ft - CG_{x.steel.25\%LL})$

$RM_{yy.steel.total} = 1.428 \times 10^{6}\ ft \cdot kip$

C. Mat foundation:

$RM_{yy.mat.total} := Wt_{mat} \cdot CG_{xx.mat}$ $RM_{yy.mat.total} = 3.437 \times 10^{6}\ ft \cdot kip$

$RM_{yy.grand.total} := RM_{yy.conc.total} + RM_{yy.steel.total} + RM_{yy.mat.total}$ $RM_{yy.grand.total} = 5.517 \times 10^{6}\ ft \cdot kip$

6.6.4.9 FACTOR OF SAFETY FOR LARGE MAT OVERTURNING ABOUT THE X-AXIS:

$$FS_{yy} := \frac{RM_{yy.grand.total} - OTM_{yy.z\_comp.total}}{OTM_{YY.grand.total} - OTM_{yy.z\_comp.total}}$$

$FS_{yy} = 2.9$

Summary:

Table 6.6.1 IHF Large Mat Summary of Factors of Safety against Overturning for Load Case1 – Equivalent Static Approach

Table 6.6.1 Summary of Factors of Safety against Overturning for Crane Load Case 1 (CLC1)

Load Case	X-Axis	Y-Axis
	Factor of Safety	Factor of Safety
	RMxx/OTMxx	Rmyy/OTMyy
CLC1	2.5	2.9

6.6.5 LARGE MAT OVERTURNING STABILITY USING APPROACH B

The basis for checking maximum overturning stability on the mat is to establish a factor of safety by comparing the ratio of maximum resisting moments composed of the downward acting gravity loads by the maximum seismic overturning moment in each orthogonal direction Ex and Ey. The Factor of Safety for each seismic direction Ex and Ey is checked against the allowable value of 1.1 (Reference 2.2.1, section 4.2.11.4.7). Attachment F includes the SAP2000 models used to establish the stability calculations.

The SAP2000 models for checking the large mat overturning stability are very similar to the structural analysis model. All the loads from the structure above the foundation mat are kept in the model. Also, seismic loads due to ground motions on the foundation mat are applied as uniform area loads as follows:

The Mat self weight is 6 ft x 0.15 Kip/ft^3 = 0.9 Kip/ft^2

The uniform superimposed dead load applied on Mat is 0.14 Kip/ft^2 (Reference 2.2.21)

The horizontal ground motion acceleration for DBGM-2 is 0.4537g (Ref. 2.2.12, Table 6-5)

The vertical ground motion acceleration for DBGM-2 is 0.3194g (Ref. 2.2.12, Table 6-6)

The horizontal uniform area loads applied on Large Mat at both X and Y directions are:

(0.9 Kip/ft^2 + 0.14 Kip/ft^2) x 0.4537= 0.472 Kip/ft^2

The vertical uniform area load applied on Large Mat at Z direction is:

(0.9 Kip/ft^2 + 0.14 Kip/ft^2) x 0.3194 = 0.332 Kip/ft^2

Because the concrete structure is close to North-West corner, the North-West corner is critical for overturning. The point of origin for checking the overturning occurs at 179.5 feet from the Y-axis and 165 feet from the X-axis, (Model Origin see Figure 6.6.3).

The non-linear links in the large mat structural analysis model are deleted. Joint #2 is added at vertical negative 6 feet (the overturning point elevation is -6’-0” because the bottom of large mat is at elevation -6’-0”) below point (179.5, 165) at the North-West corner. SAP2000 body constraint function is used to constrain Joint #8490 and Joint #2. Fixed support is added to Joint #2. By obtaining the reactions at Joint #2, the value of overturning moments and resisting moments for each load combinations are found.

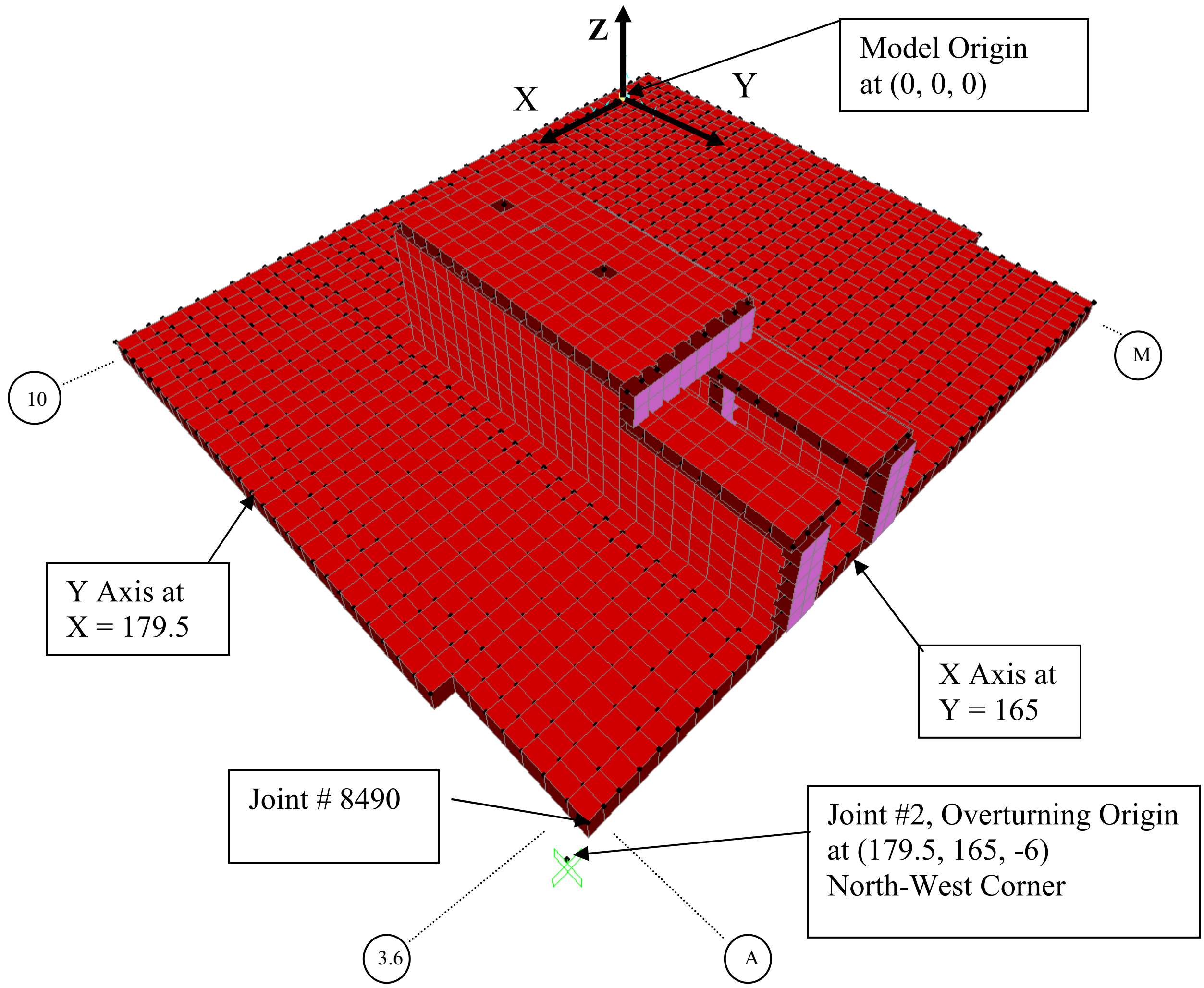


Figure 6.6.3 Isometric View of Overturning Model for Large Mat Foundation

The seismic loads from the supporting super-structures are derived from the seismic analysis in the east-west (EX), north-south (EY) and vertical (EZ) directions (References 2.2.3 and 2.2.5), and also include the foundation mat ground motion loads. Static loads are also derived from the structural analysis in Reference 2.2.3 and 2.2.5. In order to account for 3-D spatial seismic effects, the "100-40-40" component factor method from ASCE 4-98 (Reference 2.2.7, Section 3.2.7.1.2) is used.

There are 24 load combinations for overturning as follow. The envelope of the 24 load combinations determine the overturning moment M1 and M2 in Attachment F for each crane load case.

1.	+ 1.0 EX	+ 0.4 EY	+ 0.4	EZ
2.	+ 1.0 EX	− 0.4 EY	+ 0.4	EZ
3.	+ 1.0 EX	+ 0.4 EY	− 0.4	EZ
4.	+ 1.0 EX	− 0.4 EY	− 0.4	EZ
5.	− 1.0 EX	+ 0.4 EY	+ 0.4	EZ
6.	− 1.0 EX	− 0.4 EY	+ 0.4	EZ
7.	− 1.0 EX	+ 0.4 EY	− 0.4	EZ
8.	− 1.0 EX	− 0.4 EY	− 0.4	EZ
9.	+ 0.4 EX	+ 1.0 EY	+ 0.4	EZ
10.	− 0.4 EX	+ 1.0 EY	+ 0.4	EZ
11.	+ 0.4 EX	+ 1.0 EY	− 0.4	EZ
12.	− 0.4 EX	+ 1.0 EY	− 0.4	EZ
13.	+ 0.4 EX	− 1.0 EY	+ 0.4	EZ
14.	− 0.4 EX	− 1.0 EY	+ 0.4	EZ
15.	+ 0.4 EX	− 1.0 EY	− 0.4	EZ
16.	− 0.4 EX	− 1.0 EY	− 0.4	EZ
17.	+ 0.4 EX	+ 0.4 EY	+ 1.0	EZ
18.	− 0.4 EX	+ 0.4 EY	+ 1.0	EZ
19.	+ 0.4 EX	− 0.4 EY	+ 1.0	EZ
20.	− 0.4 EX	− 0.4 EY	+ 1.0	EZ
21.	+ 0.4 EX	+ 0.4 EY	− 1.0	EZ
22.	− 0.4 EX	+ 0.4 EY	− 1.0	EZ
23.	+ 0.4 EX	− 0.4 EY	− 1.0	EZ
24.	− 0.4 EX	− 0.4 EY	− 1.0	EZ

The envelope of the above 24 load cases, determine the overturning moment.

The following load combination to determine the restoring moment for each crane load case is shown in Attachment F:

25. +1.0DL + 0.25 LL

This load combination result is combined with Ez for computation of restoring moment.

Where

DL = 100% self weight of the mat + 100% superimposed dead load on mat + 100% superimposed dead load from supporting superstructure (Sections 6.4.1 and 6.4.4 of Reference 2.2.21);

LL = 100% of superimposed live load from supporting superstructures (Reference 2.2.12, Section 8.3.1).

Table 6.6.2 below is a sample of SAP2000 output file for CLC1 (see Attachment F\OTM Models for Large Mat\Large Mat CLC1 OTM Analysis\ Large Mat CLC1 Overturning Model - OUTPUT.xls, work sheet joint reactions).

Table 6.6.2 IHF Large Mat Joint Reactions for CLC1

TABLE 6.6.2: Joint Reactions CLC1							
Joint	**OutputCase**	**CaseType**	**F1**	**F2**	**F3**	**M1**	**M2**
						Moment about x-axis	**Moment about y-axis**
Text	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft
2	(1)+EX+0.4EY+0.4EZ	LinStatic	-38679	-21235	-16311	2178136	-2866588
2	(2)+EX-0.4EY+0.4EZ	LinStatic	-32942	4969	-11497	928447	-2160220
2	(3)+EX+0.4EY-0.4EZ	LinStatic	-32700	-15603	2804	297141	-764650
2	(4)+EX-0.4EY-0.4EZ	LinStatic	-26963	10602	7617	-952549	-58282
2	(5)-EX+0.4EY+0.4EZ	LinStatic	26963	-10602	-7617	952549	58282
2	(6)-EX-0.4EY+0.4EZ	LinStatic	32700	15603	-2804	-297141	764650
2	(7)-EX+0.4EY-0.4EZ	LinStatic	32942	-4969	11497	-928447	2160220
2	(8)-EX-0.4EY-0.4EZ	LinStatic	38679	21235	16311	-2178136	2866588
2	(9)+0.4EX+EY+0.4EZ	LinStatic	-23289	-37699	-17313	2747726	-2518903
2	(10)-0.4EX+EY+0.4EZ	LinStatic	2968	-33445	-13835	2257492	-1348955
2	(11)+0.4EX+EY-0.4EZ	LinStatic	-17311	-32066	1802	866731	-416965
2	(12)-0.4EX+EY-0.4EZ	LinStatic	8946	-27813	5279	376496	752983
2	(13)+0.4EX-EY+0.4EZ	LinStatic	-8946	27813	-5279	-376496	-752983
2	(14)-0.4EX-EY+0.4EZ	LinStatic	17311	32066	-1802	-866731	416965
2	(15)+0.4EX-EY-0.4EZ	LinStatic	-2968	33445	13835	-2257492	1348955
2	(16)-0.4EX-EY-0.4EZ	LinStatic	23289	37699	17313	-2747726	2518903
2	(17)+0.4EX+0.4EY+EZ	LinStatic	-23470	-22270	-28038	3221206	-3565581
2	(18)-0.4EX+0.4EY+EZ	LinStatic	2787	-18016	-24561	2730971	-2395633
2	(19)+0.4EX-0.4EY+EZ	LinStatic	-17733	3935	-23225	1971517	-2859213
2	(20)-0.4EX-0.4EY+EZ	LinStatic	8524	8189	-19748	1481282	-1689265
2	(21)+0.4EX+0.4EY-EZ	LinStatic	-8524	-8189	19748	-1481282	1689265
2	(22)-0.4EX+0.4EY-EZ	LinStatic	17733	-3935	23225	-1971517	2859213
2	(23)+0.4EX-0.4EY-EZ	LinStatic	-2787	18016	24561	-2730971	2395633
2	(24)-0.4EX-0.4EY-EZ	LinStatic	23470	22270	28038	-3221206	3565581
	Note 1: M1 and M2 results of Load case 1~24 are overturning moments against X and Y axis. The M1 and M2 include the values from the EZ component.						
2	(25)DL+0.25LL	LinStatic	0	0	58628	-4811292	5462658
	Note 2: M1 and M2 results of Load case 25 are resisting moments against X and Y axis. The EZ component should be adjusted from the M1 and M2 results for the computation on resisting moment.						
2	(26)EX	LinStatic	-32821	-5317	-4347	612794	-1462435
2	(27)EY	LinStatic	-7171	-32756	-6017	1562111	-882960
2	(28)EZ	LinStatic	-7473	-7041	-23893	2351244	-2627423

Note: Counteracting effects due to F_3 from Ex and Ey are not considered.

Table 6.6.3., Summary of Factors of Safety against overturning for all load cases is shown below. The Factors of Safety are extracted from Attachment F.

For detailed SAP2000 output of overturning and resisting moments of every crane load case:

Attachment F\OTM Models for Large Mat\Large Mat CLC1 OTM Analysis\Large Mat CLC1 Overturning Model - OUTPUT.xls
Attachment F\OTM Models for Large Mat\Large Mat CLC2 OTM Analysis\Large Mat CLC2 Overturning Model - OUTPUT.xls
Attachment F\OTM Models for Large Mat\Large Mat CLC3 OTM Analysis\Large Mat CLC3 Overturning Model - OUTPUT.xls
Attachment F\OTM Models for Large Mat\Large Mat CLC4 OTM Analysis\Large Mat CLC4 Overturning Model - OUTPUT.xls
Attachment F\OTM Models for Large Mat\Large Mat CLC5 OTM Analysis\Large Mat CLC5 Overturning Model - OUTPUT.xls
Attachment F\OTM Models for Large Mat\Large Mat CLC6 OTM Analysis\Large Mat CLC6 Overturning Model - OUTPUT.xls
Attachment F\OTM Models for Large Mat\Large Mat CLC7 OTM Analysis\Large Mat CLC7 Overturning Model - OUTPUT.xls

Table 6.6.3 IHF Large Mat Summary of Factors of Safety against Overturning for all Load Cases – SAP 2000 Approach

Load Case	X-Axis (Y = 165)	Y-Axis (X = 179.5)
	Factor of Safety	Factor of Safety
	RM/OTM	RM/OTM
CLC1	2.14	2.43
CLC2	2.15	2.43
CLC3	2.17	2.44
CLC4	2.11	2.42
CLC5	2.16	2.49
CLC6	2.16	2.35
CLC7	2.14	2.33
Min.	2.11	2.33

Based on the *Project Design Criteria* (Reference 2.2.1, section 4.2.11.4.7), for ITS structures, the factor of safety should be 1.1 for load combination DL + H + E. The summary tables above show that the minimum factor of safety is 2.11 for CLC4. It is noted that this calculation is based on a static overturning approach.

6.7 SOIL BEARING PRESSURE EVALUATION

The permissible soil bearing capacity is 50 ksf per Section 6.2.3 of *SADA* (Reference 2.2.12).

Maximum Bearing Pressure on Mat Foundation

The maximum soil bearing pressure is determined by dividing the summation of link forces by the summation of these link's tributary areas. Due to overturning effects from the earthquake, the corner area of each mat will have the highest soil bearing pressure.

6.7.1 SMALL MAT SOIL BEARING PRESSURE

The summation of link forces at the Northwest corner of the small mat is 4509 kips, as shown in Figure 6.7.1 below. These link forces are obtained from Table I-8 (Table I-8 is the maximum value link forces from sheet "Element Joint Forces - Links", of "IHF Small (west) Mat SAP2000 output envelope file.xls" in Attachment C). The total tributary area of these links is 102.4 sq. ft (102.4 sq. ft =7.25 ft *14.125 ft) as shown in Figure 6.7.1. The corner area of 7.25 ft by 14.125 ft for the distribution of the bearing pressure from the link forces, is reasonable considering the dispersion of the load through the 6' thick mat. The soil pressure at this critical corner = 44 ksf (44 ksf = 4509kips/102.4sq. ft). This soil pressure is less than the permissible soil bearing capacity of 50 ksf. Therefore, the soil bearing pressure on the small mat is acceptable.

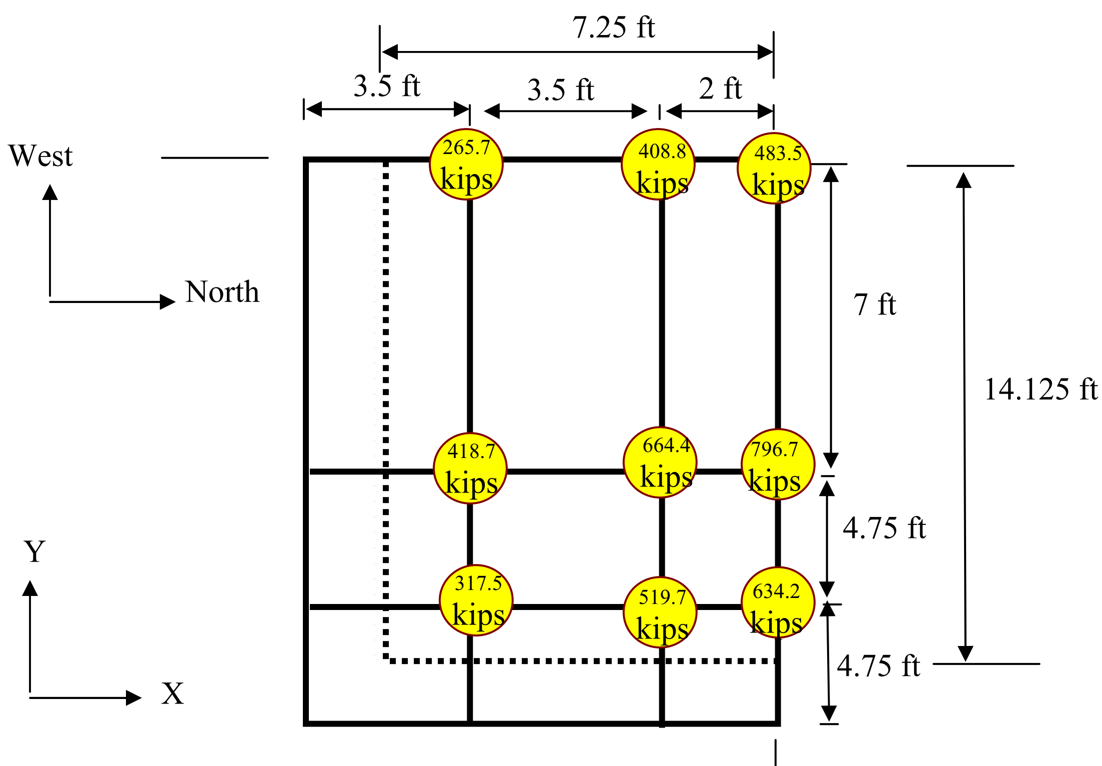


Figure 6.7.1 Small Mat Northwest Corner Soil Bearing Pressure

6.7.2 LARGE MAT SOIL BEARING PRESSURE

The maximum link reaction for the large mat foundation is 56.9 kips at link #7000 (south east corner) in load case 7 (Attachment B, *IHF Large Mat CLC7 SAP2000 Output Data.xls*). The tributary area for link #7000 at the large mat is 6.25 ft^2. Therefore, the maximum bearing pressure on the large mat = 56.9 / 6.25 = 9.1 kips per square foot, which is less than the permissible soil bearing capacity of 50 ksf (Reference 2.2.12 Section 6.2.3) for a large mat foundation. Therefore, the large mat soil bearing pressure is acceptable.

6.8 COMBINED IHF STRUCTURAL MODEL

To facilitate SASSI (Reference 2.2.23) analysis, a combined model of the IHF structure was created. The combined IHF structural finite element model consists of the IHF Steel Structure (Reference 2.2.3), Concrete Structures (Reference 2.2.5) and Mat Foundations (this calculation). Additional NOG-1 *Rules for Construction of Overhead and Gantry Cranes* (Reference 2.2.20) boundary conditions for the steel structure cranes were modeled at the cranes and the rest of the structural interface as noted in Section 6.9. The NOG-1 boundary conditions for the cranes follow node designations A through D in Table 4154.3-1 of Reference 2.2.20. The structural steel model (Reference 2.2.3) consists of load combinations associated with seven distinct crane locations. For the SASSI efforts, only two pertinent load combinations, "Load Case 1" and "Load Case 5" from the Steel Structure calculation are included (see Attachment J for all 7 load cases).

Table 6.8.1 shows SAP2000 grid-labeling scheme used in the combined model.

Table 6.8.1 Combined Model Joint Labeling Scheme

	Load Case 5 Joint Labeling	**Load Case 1 Joint Labeling**
Steel Structure	1001-2549	1001-2553
Interior Concrete Structure	4000-4569	4000-4569
Exterior Concrete Structure	5000-6508	5000-6508
East Base Mat	7000-8490	7000-8490
West Base Mat	9000-9509	9000-9509

DVD Attachment H contains the entire SAP2000 combined model files.

6.9 COUPLED MODEL FOR THE MAJOR CRANES

The major cranes located in the IHF are included in the combined IHF structural model for the SASSI. (Reference 2.2.23) The approach is to couple the IHF structural model and the crane model. The crane elements in the combined model are represented by beam elements. The advantage of the coupled approach is that it enables reduction of interface reactions between the cranes and the structure. Attachment K provides detailed discussions on the coupled model.

7. RESULTS AND CONCLUSIONS

7.1 RESULTS

The primary results of this calculation are:

- Design forces and moments:
  - Contour plots shown in Attachments D and E represent the shear forces and bending moments that will occur in the IHF foundation mats under the design loading combinations. The contours are used to obtain the design forces for designing the flexural and shear reinforcement for the IHF foundation mats.
- Foundation mat flexural reinforcement:
  - Tables 6.5.1 and 6.5.4 show the maximum bending moments, reinforced concrete capacity and D/C ratios of small and large mat foundations.
  - Tables 6.5.2 and 6.5.6 show the maximum shear, reinforced concrete capacity and D/C ratios of small and large mat foundations.
- Foundation overturning stability check:
  - The small and large mat foundations have a computed static factor of safety against overturning of 1.30 (Section 6.6.2) and 2.11 (Table 6.6.3), respectively due to DBGM-2 seismic motion. This meets the required factor of safety of 1.1 per Ref. 2.2.1, Section 4.2.11.4.7. Thus the structure is stable against overturning.
- Foundation sliding stability check:
  - The calculation indicates that the large and small mats will slide due to the seismic forces from DBGM-2. The energy approach permitted by PDC (Ref. 2.2.1), Section 4.2.11.4.7, is utilized for the evaluation of sliding. The energy method shown in Section A.1 of Appendix A in ASCE 43-05 (Ref. 2.2.8) has been used for calculation of sliding displacements. The small mat has a maximum sliding displacement of 0.14 inches. The large mat has a maximum sliding displacement of 0.14 inches. These sliding displacements are insignificantly small for the large size of the building footprints. However, any commodities (piping, electrical cable, etc.) attached to the building shall be designed so that the commodities have adequate factors of safety to withstand the building sliding displacements.

- Soil Bearing Pressure

  - The soil pressure of the large mat is estimated to be 9.1 ksf (Section 6.7) and small mat soil pressure is 44 ksf (Section 6.7). Both soil pressures are less than permissible soil bearing capacity of 50 ksf (Section 6.2.3 of Reference 2.2.12).

- Combined structural model for SASSI

  - Attachment H describes the combined IHF structural model that serves as an input for the SASSI (Reference 2.2.23) analysis.

7.2 CONCLUSIONS

Results from this calculation demonstrate that a reasonable mat design for the IHF is achieved for the imposed design loads. The maximum shear forces and moments occur at the corner areas of the structure, as expected, due to combined spatial effects. The maximum shear forces occur at the face of supports (walls and columns), as expected. The foundation mat thickness in flexural and shear reinforcement indicates that the design is reasonable, conforming to *ACI-349* (Reference 2.2.6).

The stability evaluation against sliding and overturning effects for both mats indicates that the structure satisfies the stability requirements of the *Project Design Criteria* (Reference 2.2.1).

Appendix H provides a road map for the combined structural model to be used for SASSI (Reference 2.2.23) analysis.

The soil pressures of both mats (Section 6.7) are less than the permissible soil bearing pressure.

Attachment A

Small and Large Mat Foundation Plans at Elevation 0 ft.

Page

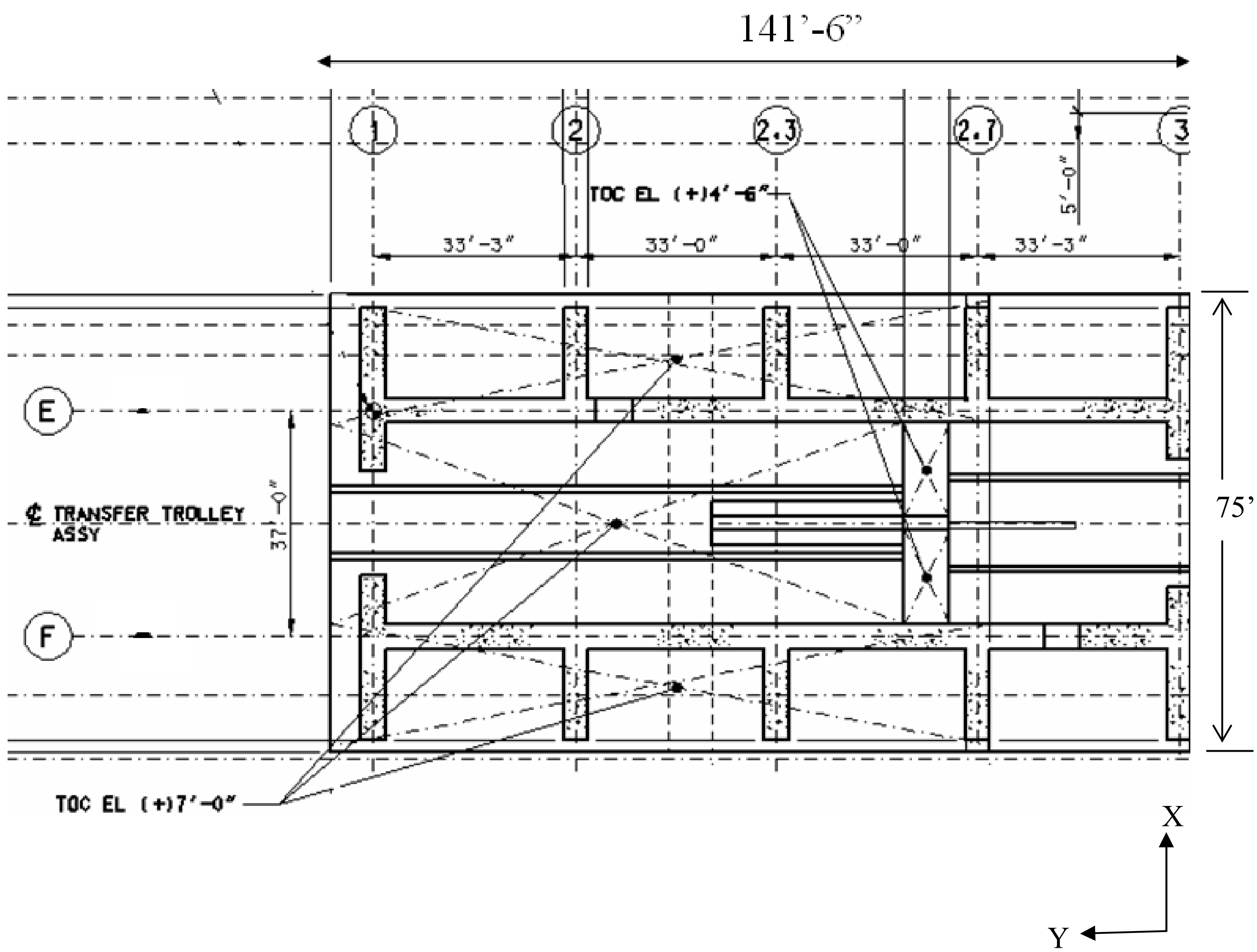


Figure A-1: IHF Small Mat Foundation Plan at elevation 0'-0"

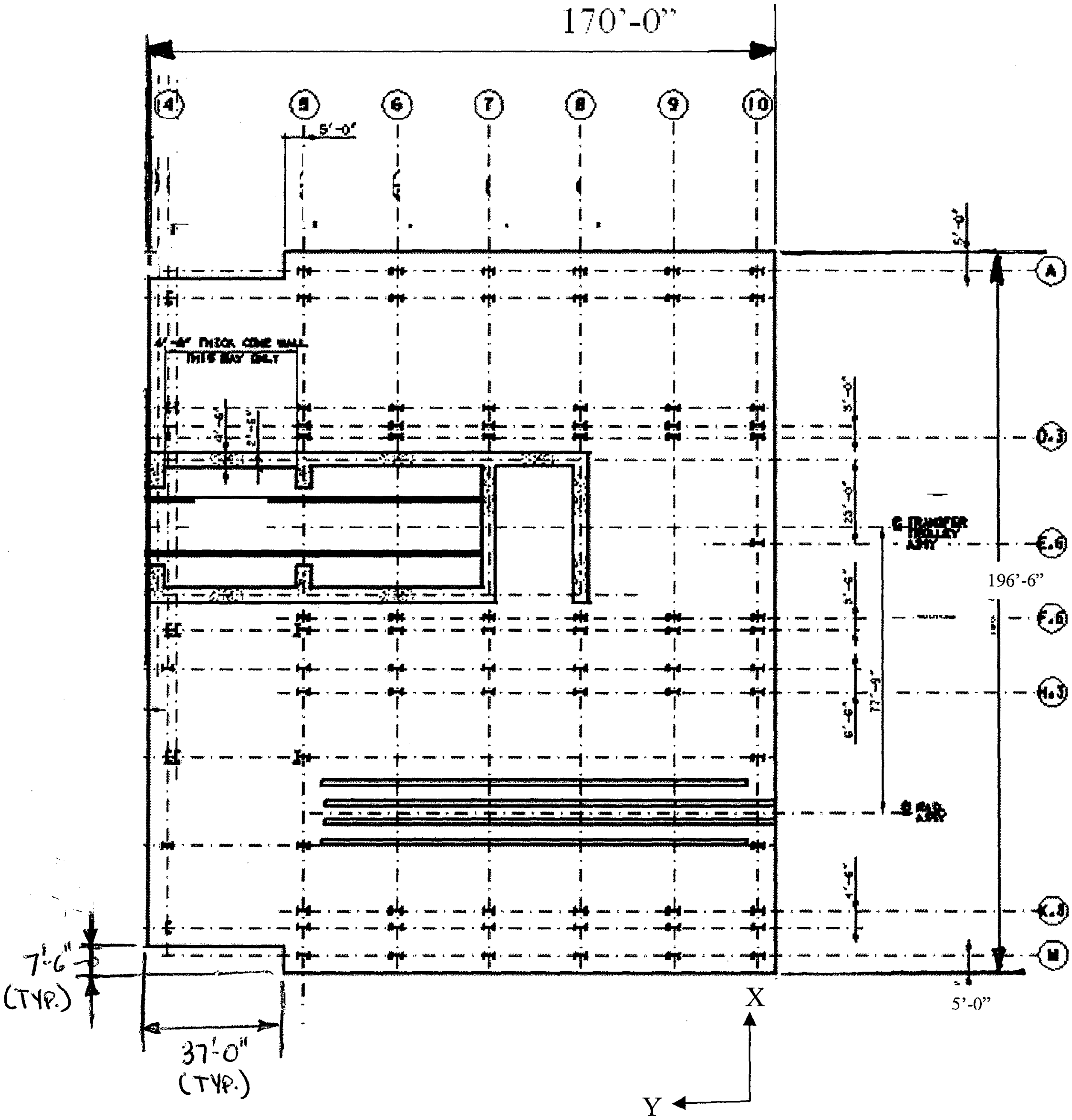


Figure A-2: IHF Large Mat Foundation Plan at elevation 0'-0"

Attachment B

SAP2000 Input & Output files for Large Mat - Crane Load Cases 1 thru 7

(Refer to the DVD 1 of 2)

Large Foundation Mat Model Explanation

Interior Concrete structure load cases:

Load Case Name	Load Description
Deadload-mat	Superimposed dead load on Mat foundation
Liveload-mat	Superimposed live load on Mat foundation
Dfloor2	Superimposed dead load on roof of interior concrete Superstructure
Lfloor2	Superimposed live load on roof of interior concrete Superstructure
SelfWt	Self weight of concrete mat foundation and Self weight of concrete superstructure
E_FX	X direction component earthquake equivalent static load of concrete superstructure
E_FY	Y direction component earthquake equivalent static load of concrete superstructure
E_FZ	Z direction component earthquake equivalent static load of concrete superstructure

Steel Structure Load Cases:

Load Case Name	Load Description
Platformdead	Superimposed dead load on platforms in steel model
Platformlive	Superimposed dead load on platforms in steel model
Roof + Cladding	Dead load due to superimposed dead load on steel roof and self-weight of cladding
Rooflive	Superimposed live load on steel roof
Cranedead	Dead load due to crane
Cranelive	Live load due to crane
Self + Rail	Self-weight of steel structure and crane rail
Snow	Load due to snow and snow drift
StaticEX	X direction component earthquake equivalent static load of Steel structure
StaticEY	Y direction component earthquake equivalent static load of Steel structure
StaticEZ	Z direction component earthquake equivalent static load of Steel structure

Attachment C

SAP2000 Input & Output files for Small Mat - Crane Load Cases 1 and 2

(Refer to the DVD 1 of 2)

Small Foundation Mat Model Load Case Explanation
Exterior Concrete load cases:

Load Case Name	Load Description
Deadload-mat	Superimposed dead load on Mat
Liveload-mat	Superimposed live load on Mat
SELF	Self weight of concrete mat foundation
W_Droof+Dcrane	Superimposed dead load on roof and dead load of the crane of Superstructure
W_Lcrane	Superimposed crane live load on Superstructure
W_Lroof	Superimposed live load on roof of Superstructure
W_SelfWt	Self weight of concrete superstructure
W_FX	X direction component earthquake equivalent static load of superstructure
W_FY	Y direction component earthquake equivalent static load of superstructure
W_FZ	Z direction component earthquake equivalent static load of superstructure

Attachment D

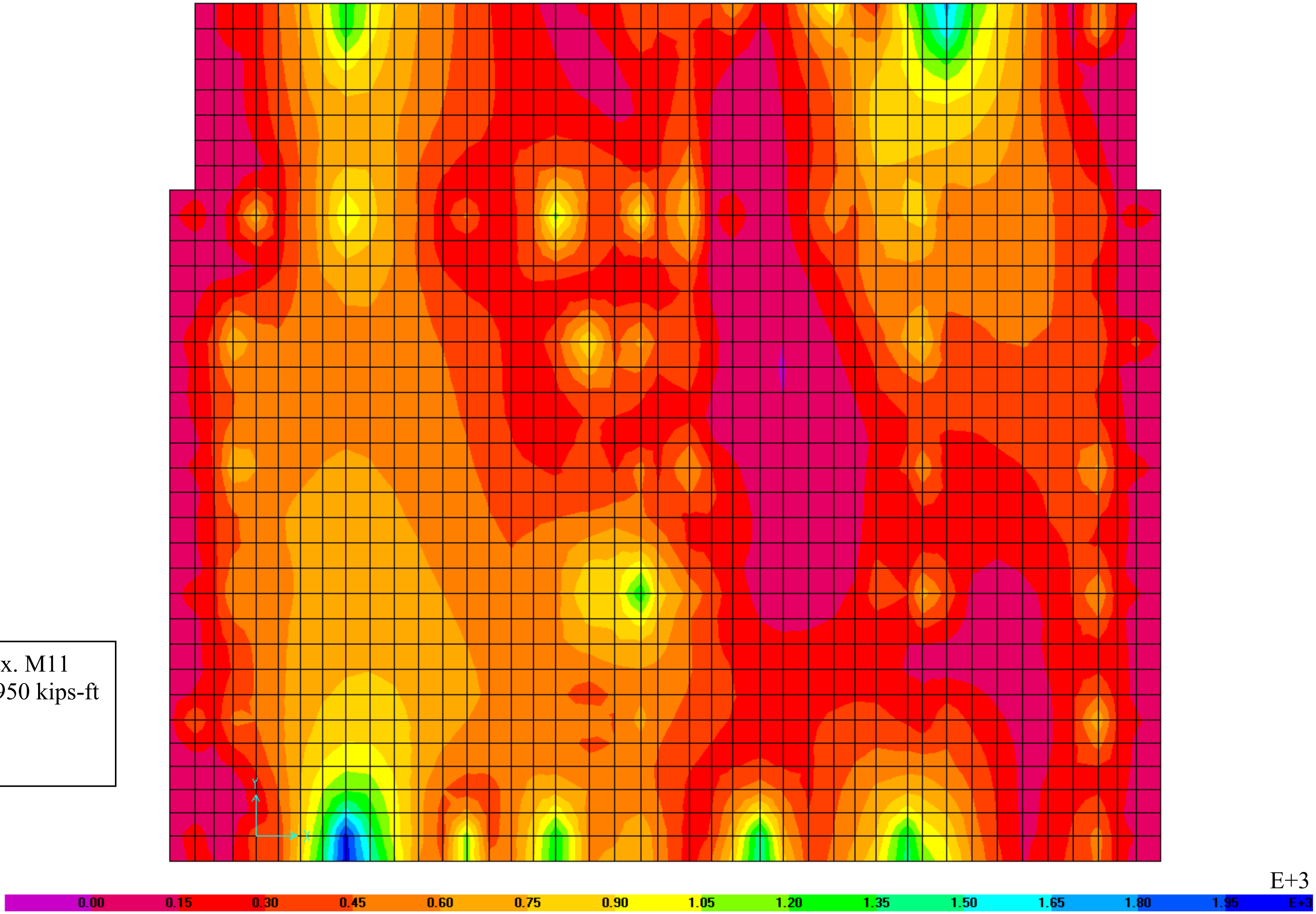
SAP2000 Output - Moment and Shear Contours For Large Mat

Page

Crane Load Case 3:

Crane Load Case 4:

Crane Load Case 5:



Max. M11
+1950 kips-ft
/ft

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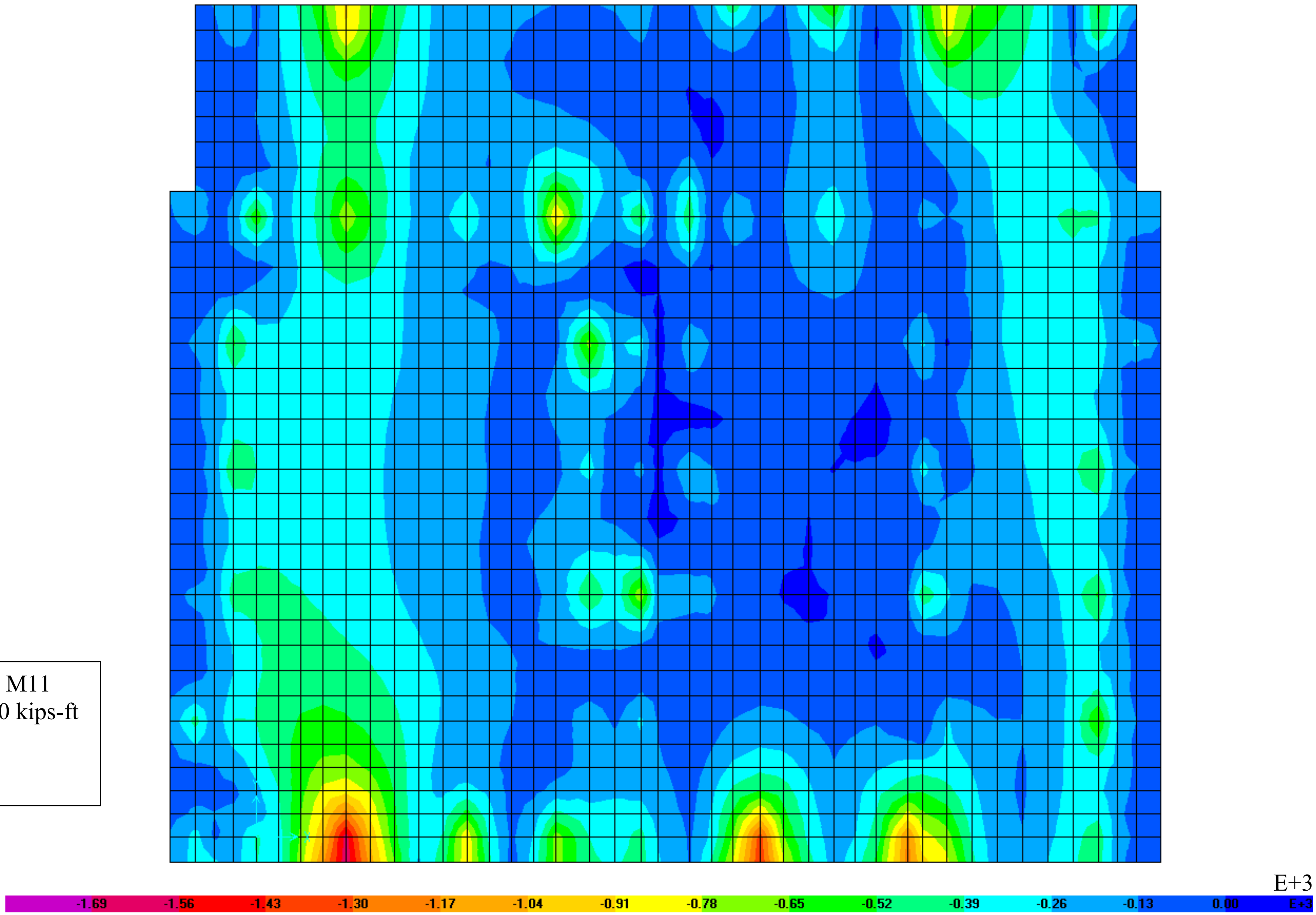

Min. M11
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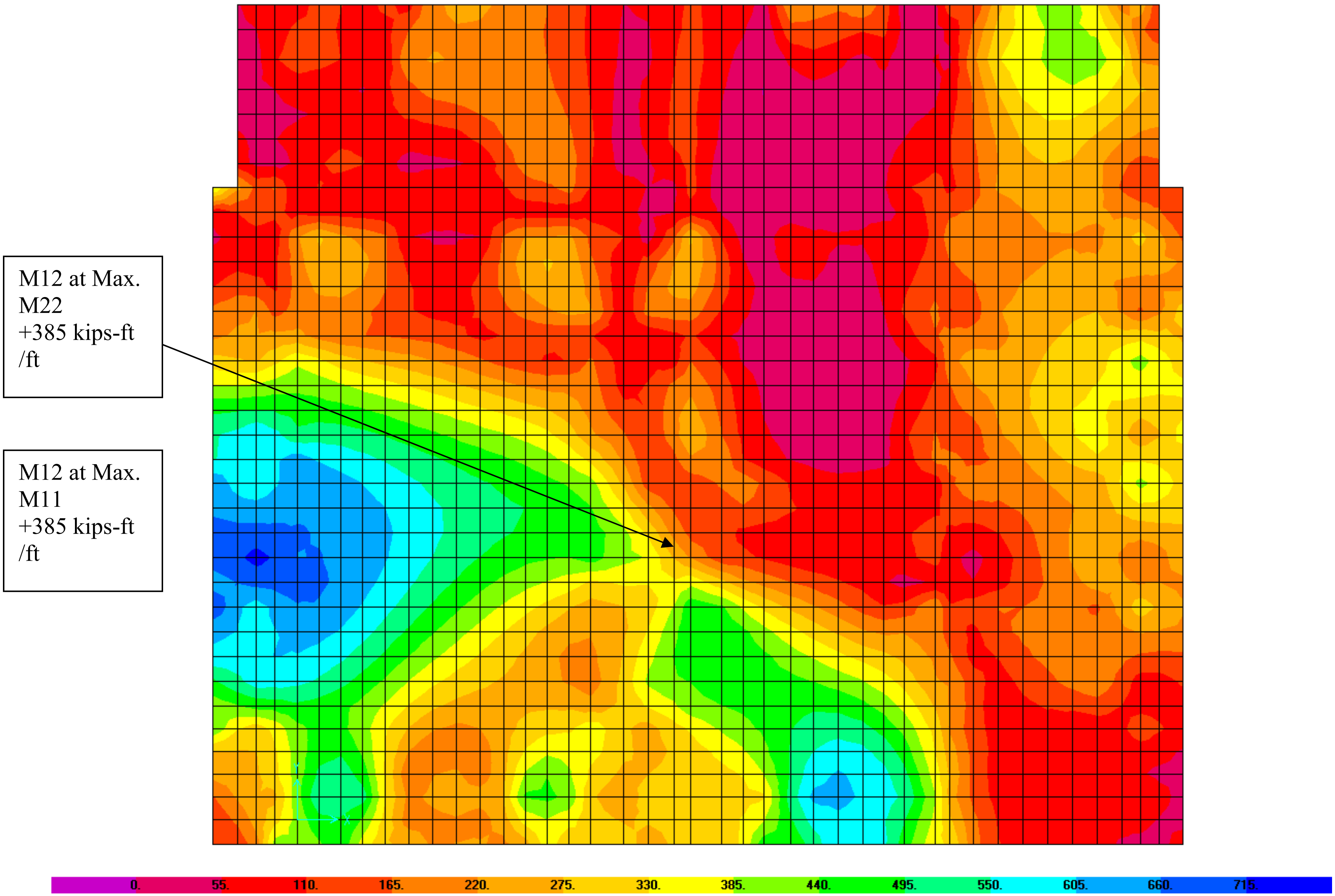

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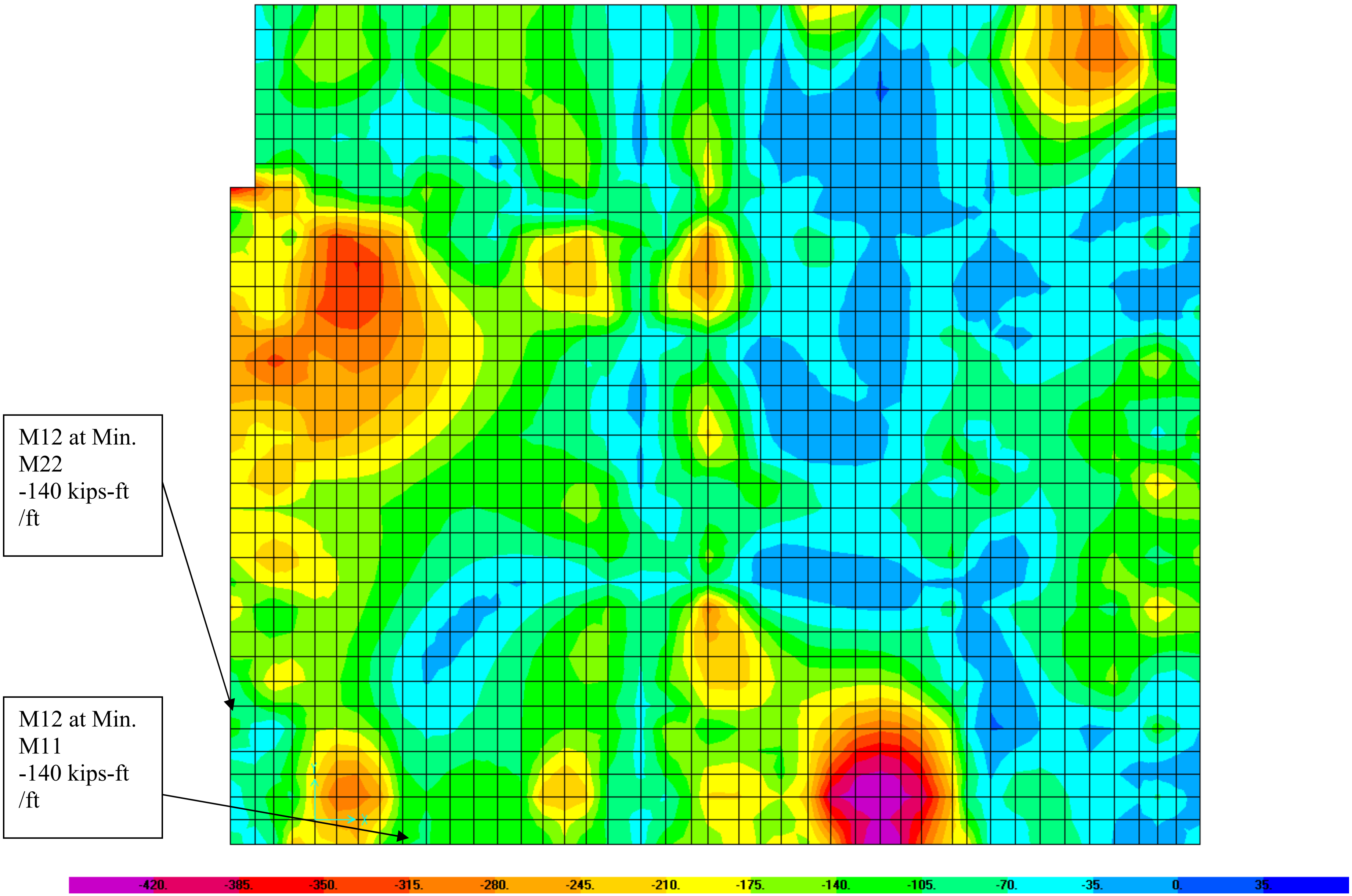

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Max. M22
+1680 kips-ft
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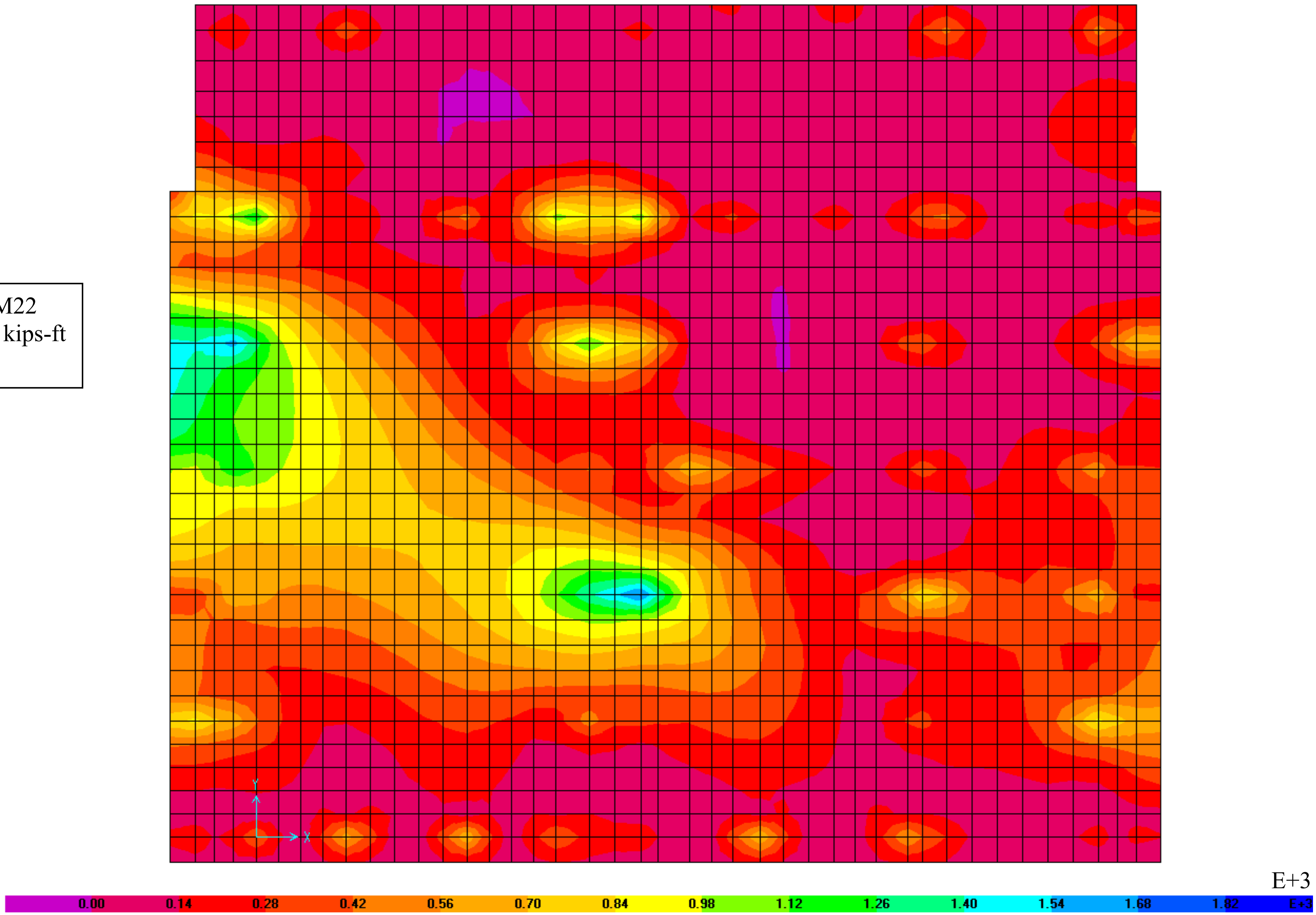

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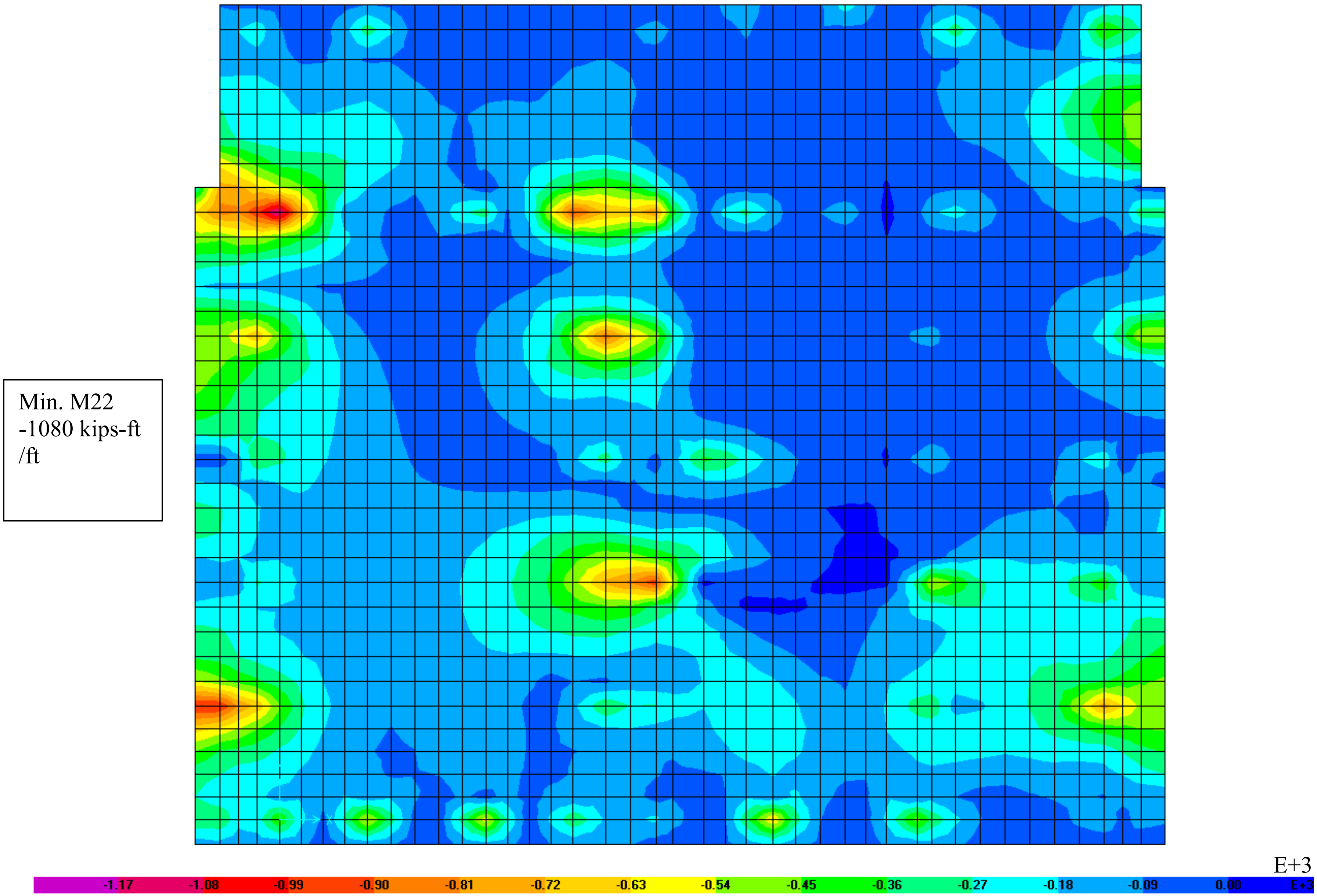

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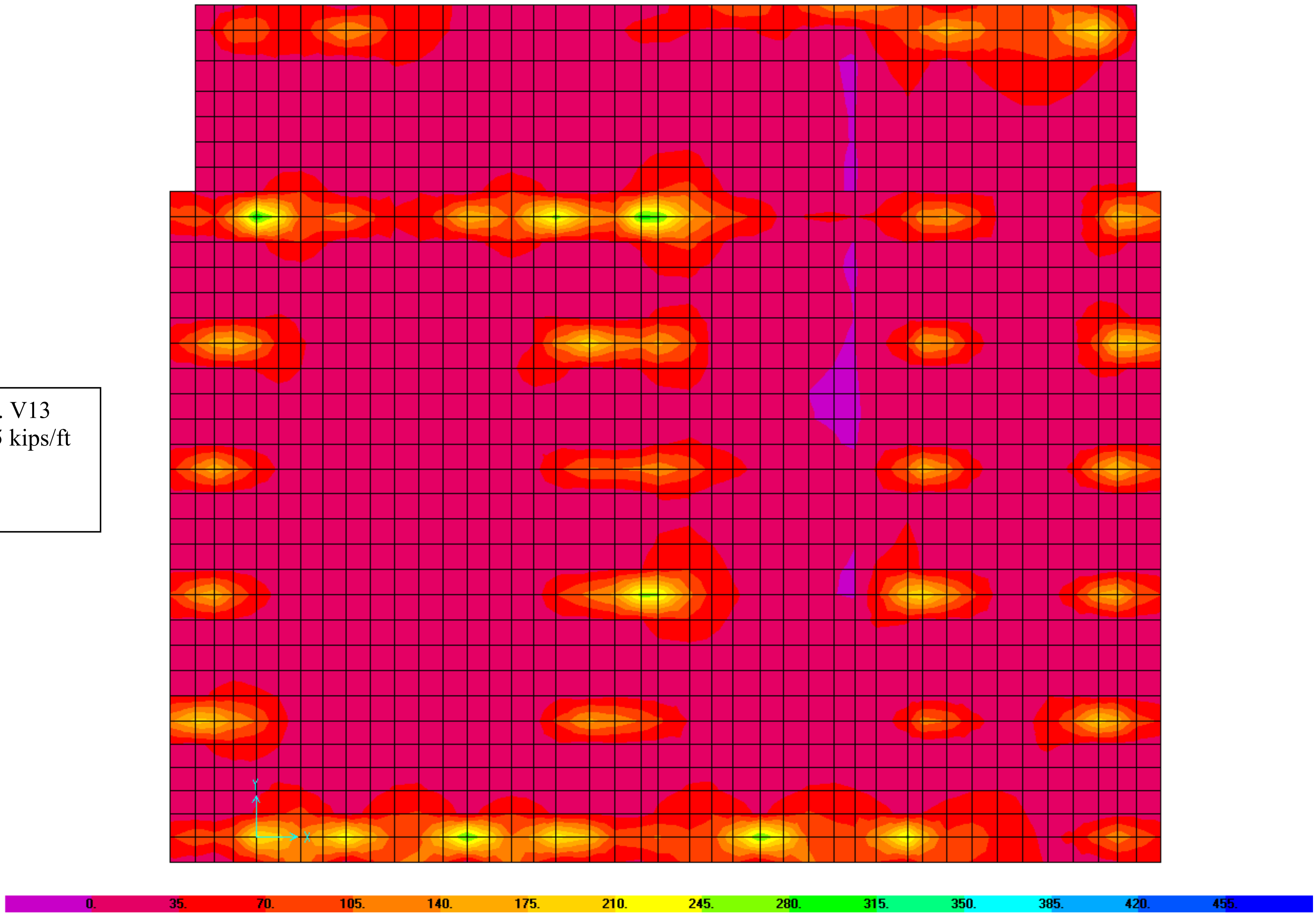
Max. V13
+175 kips/ft



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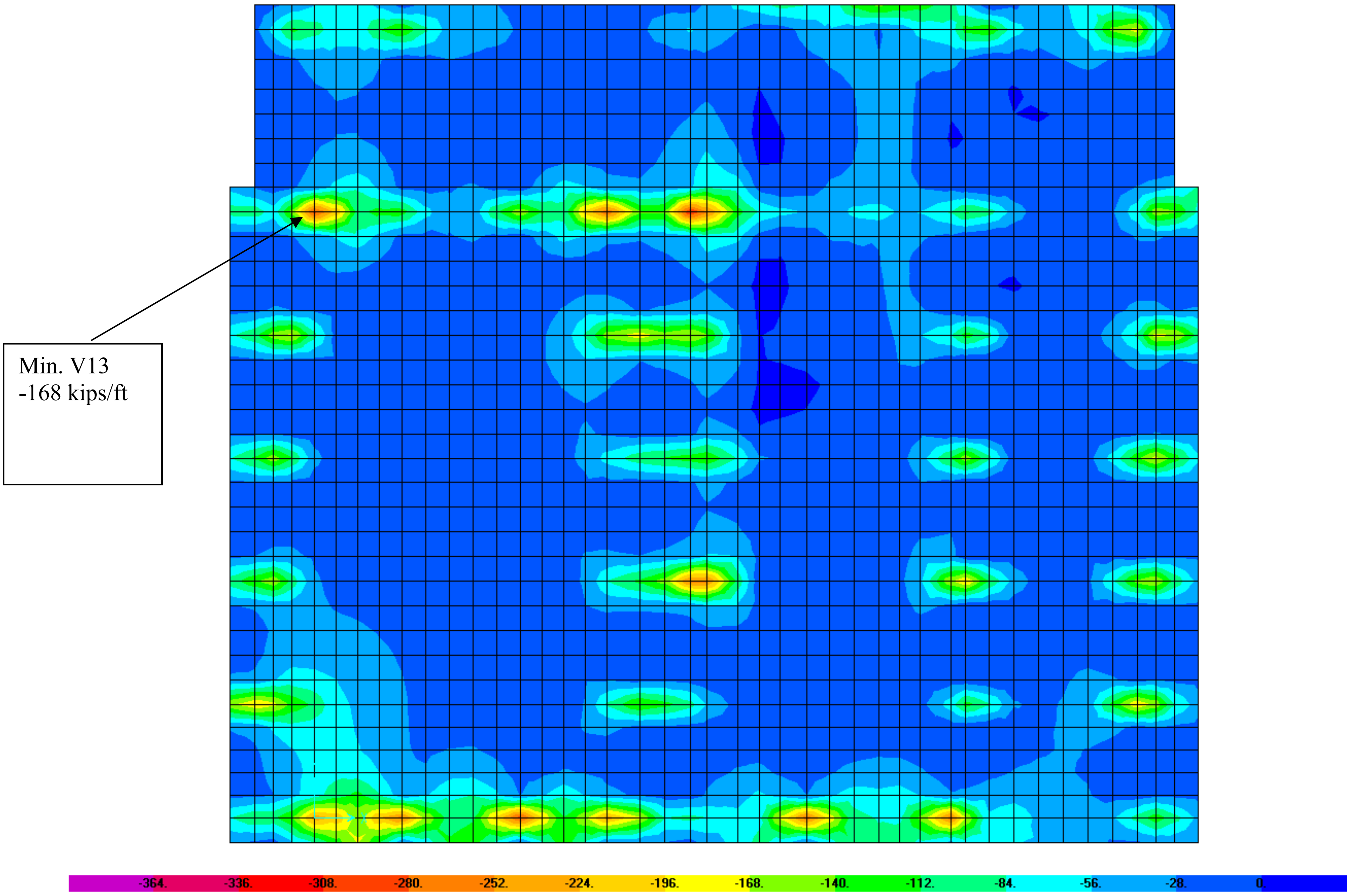

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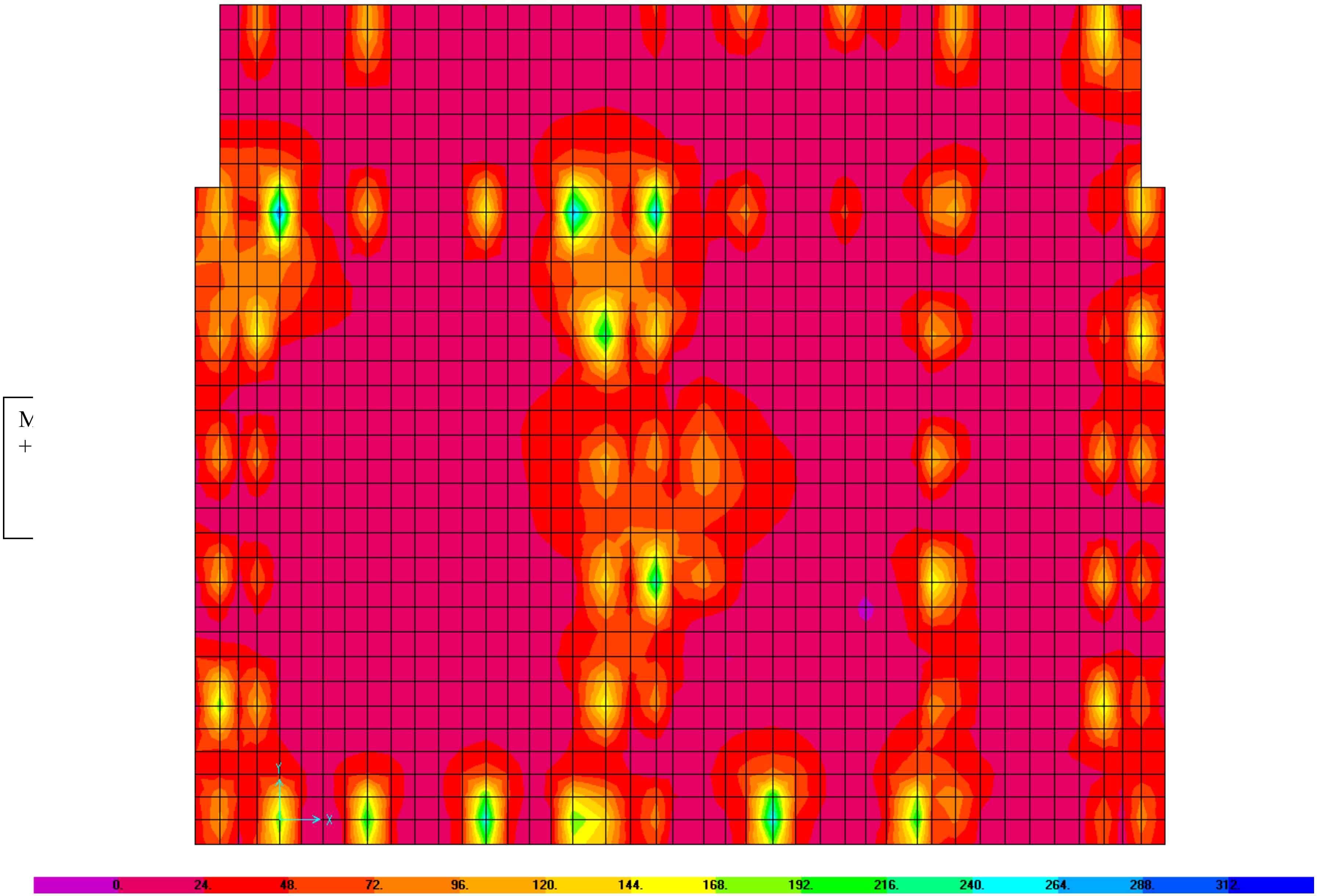

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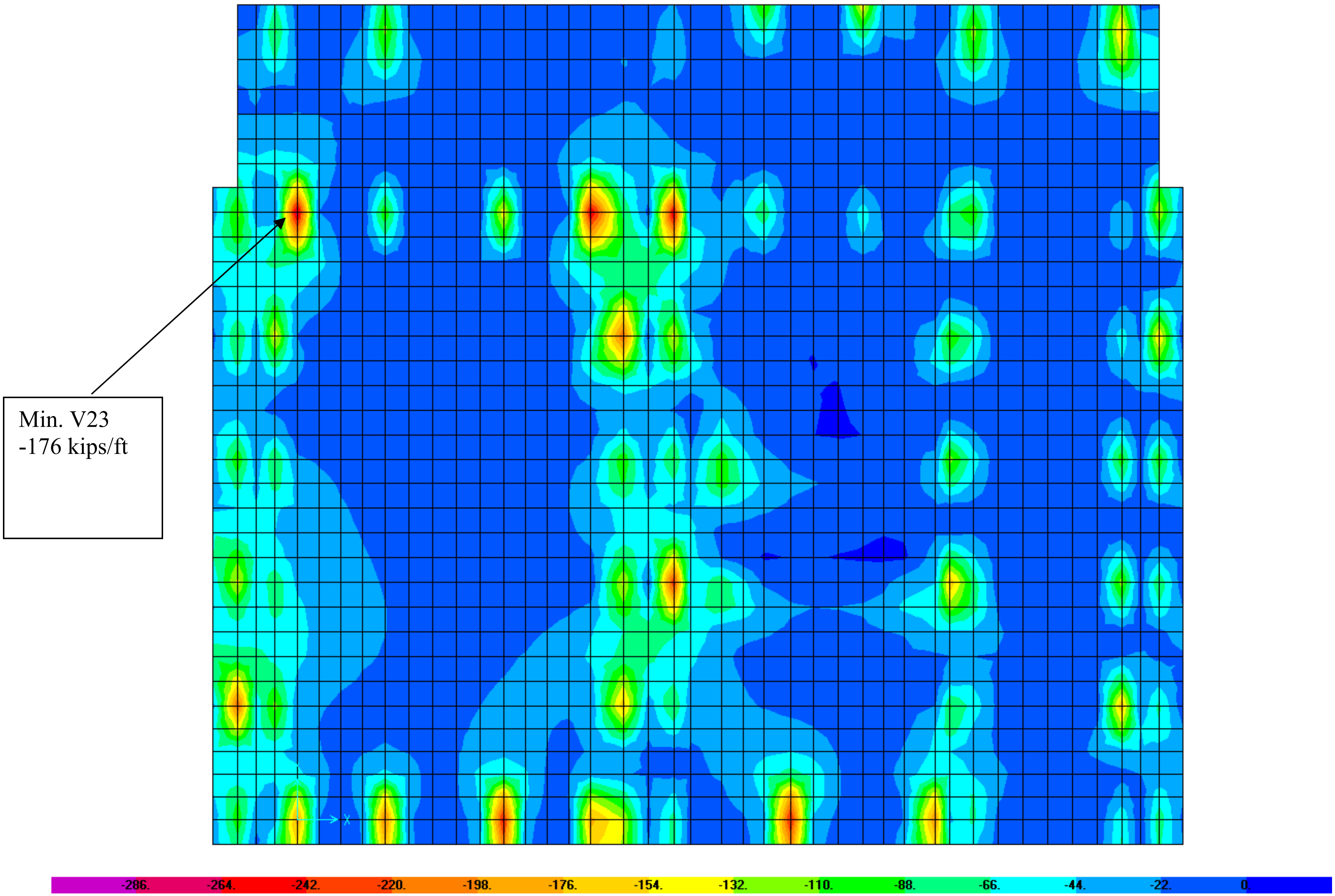

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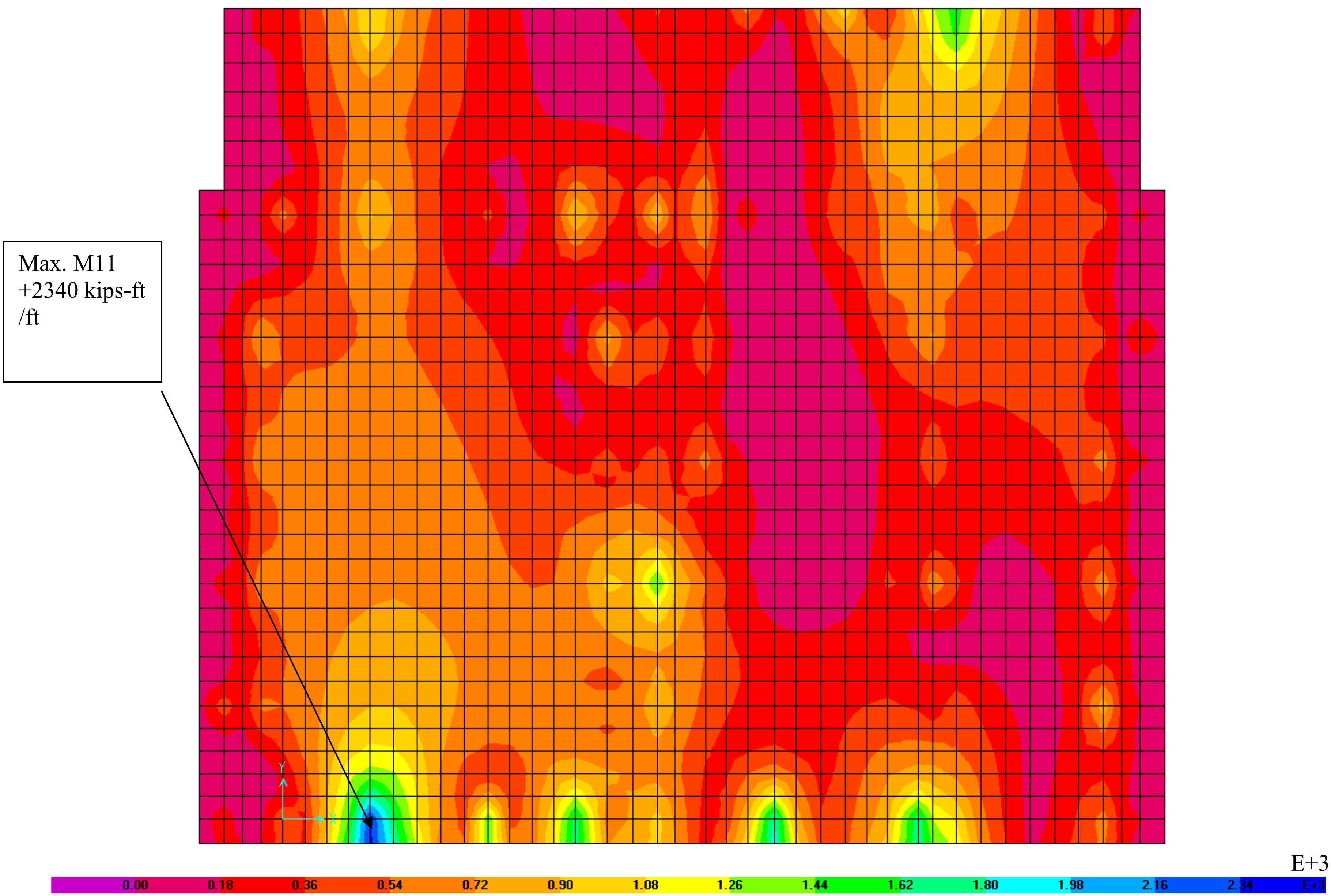

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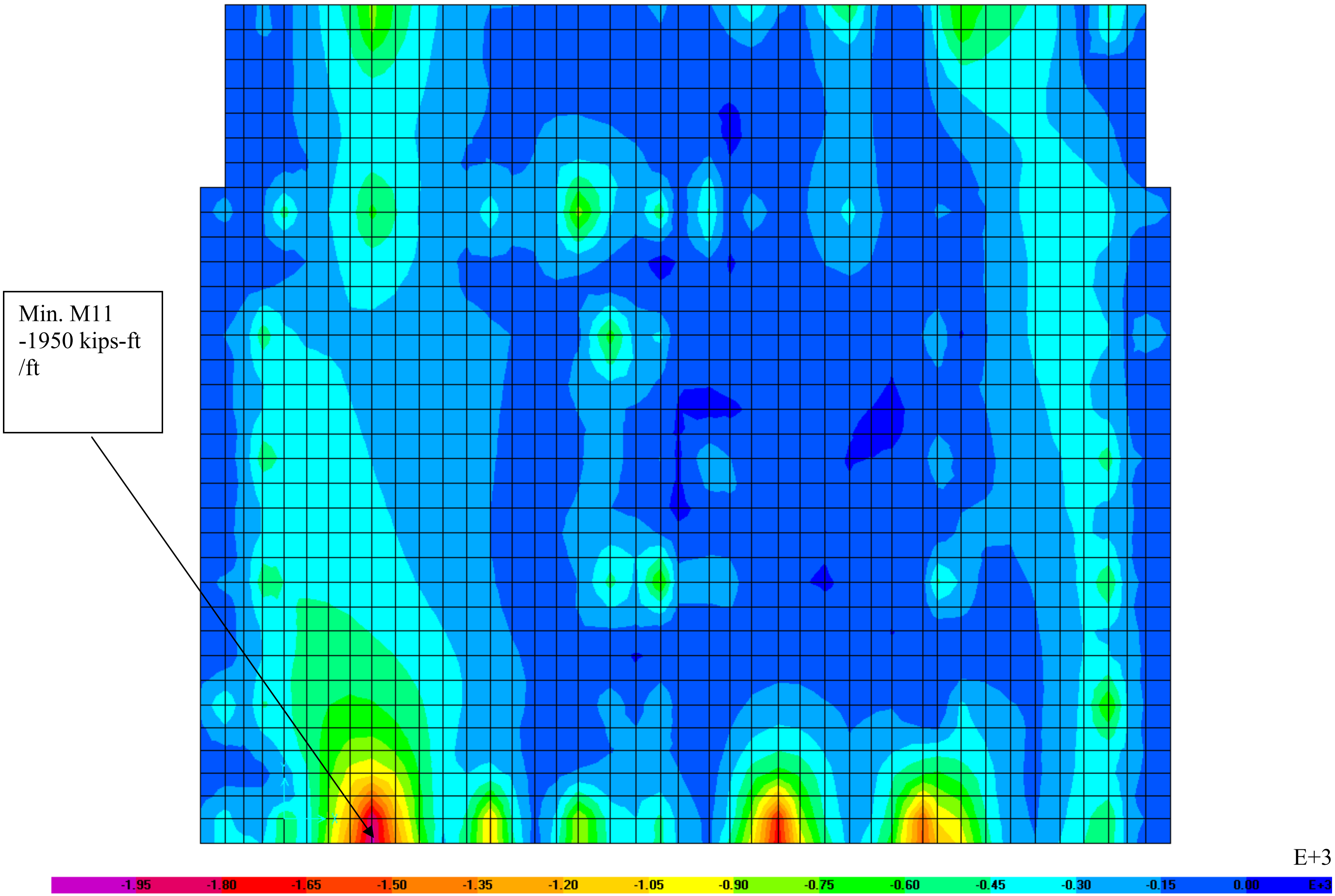

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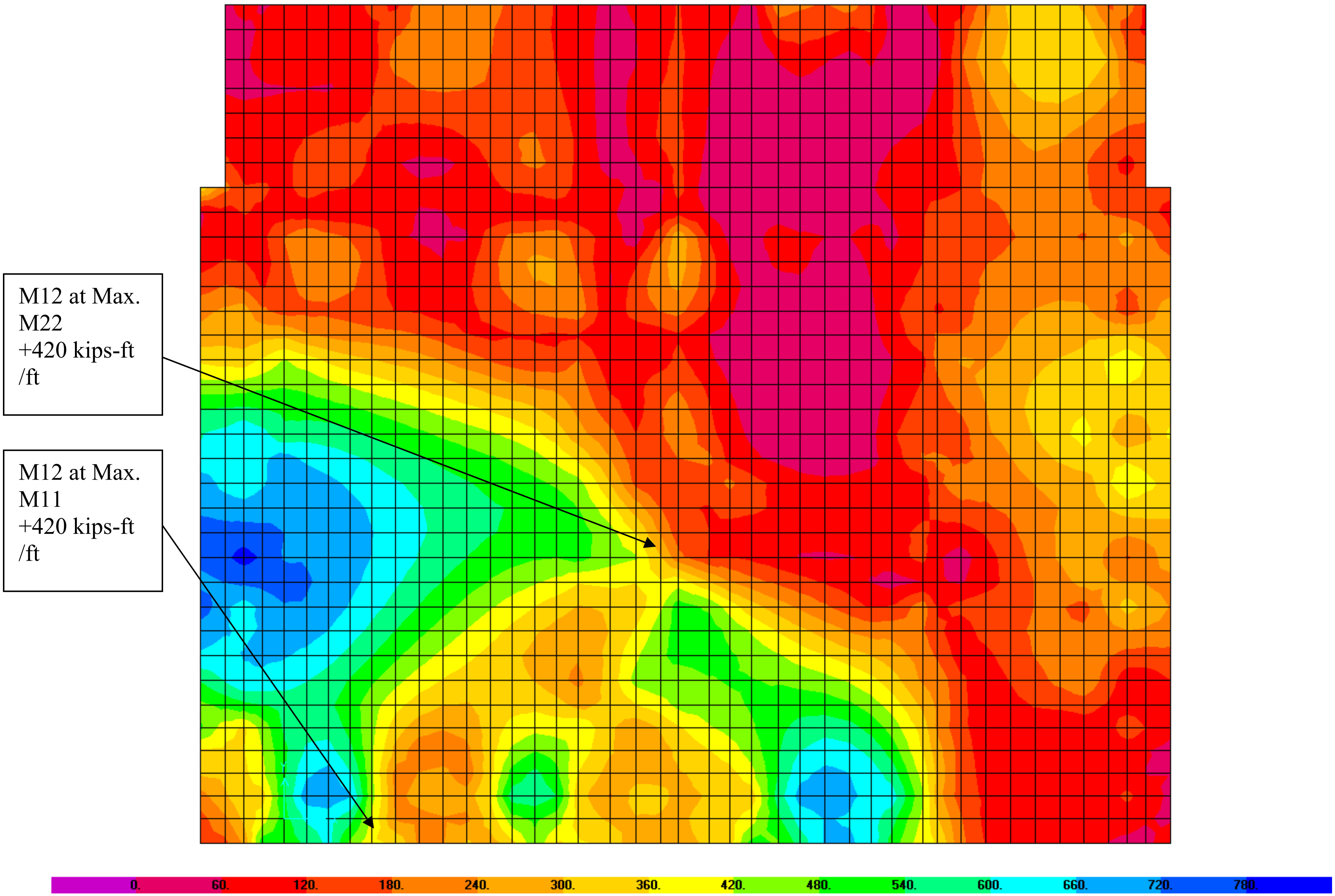

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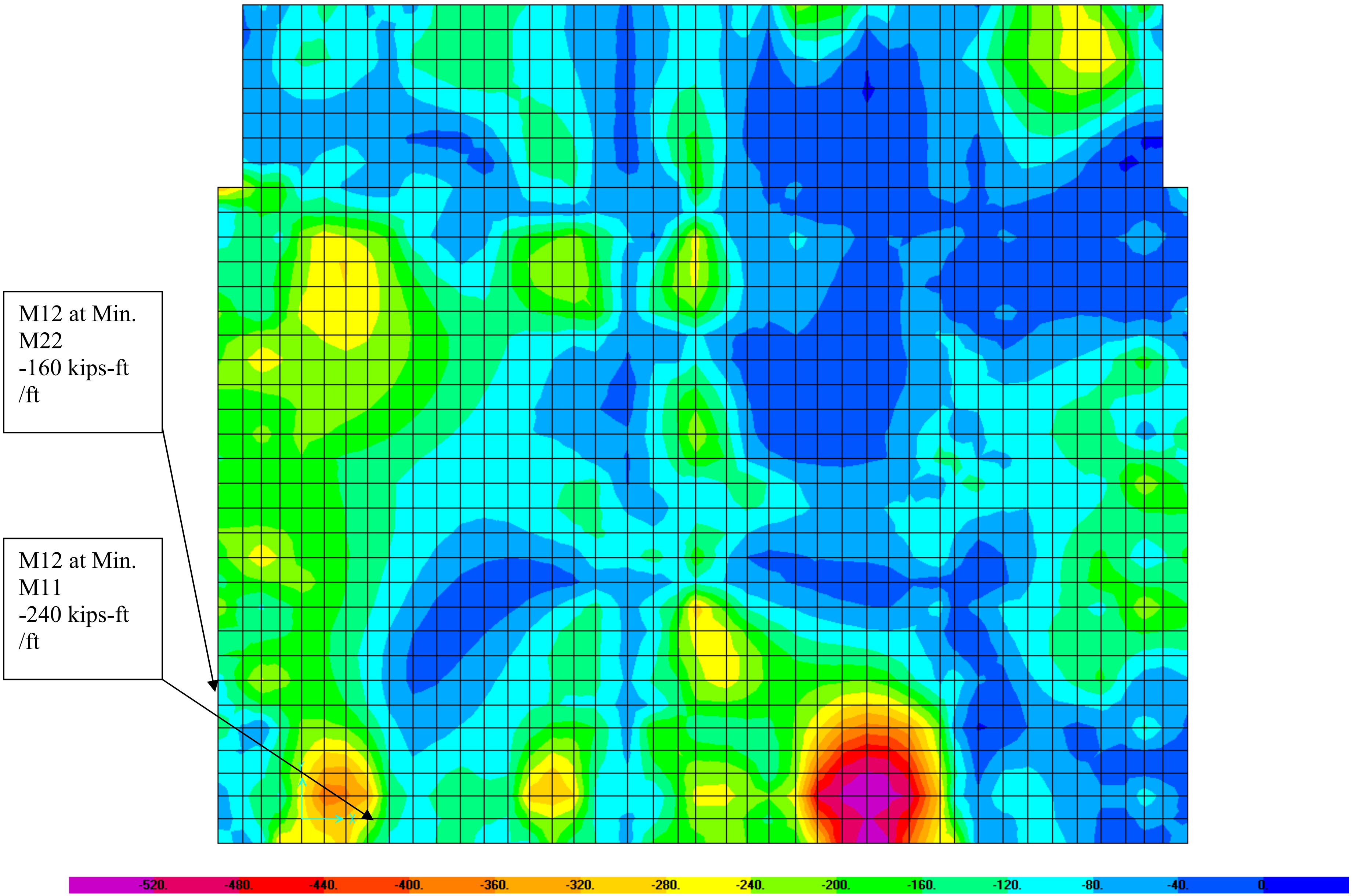

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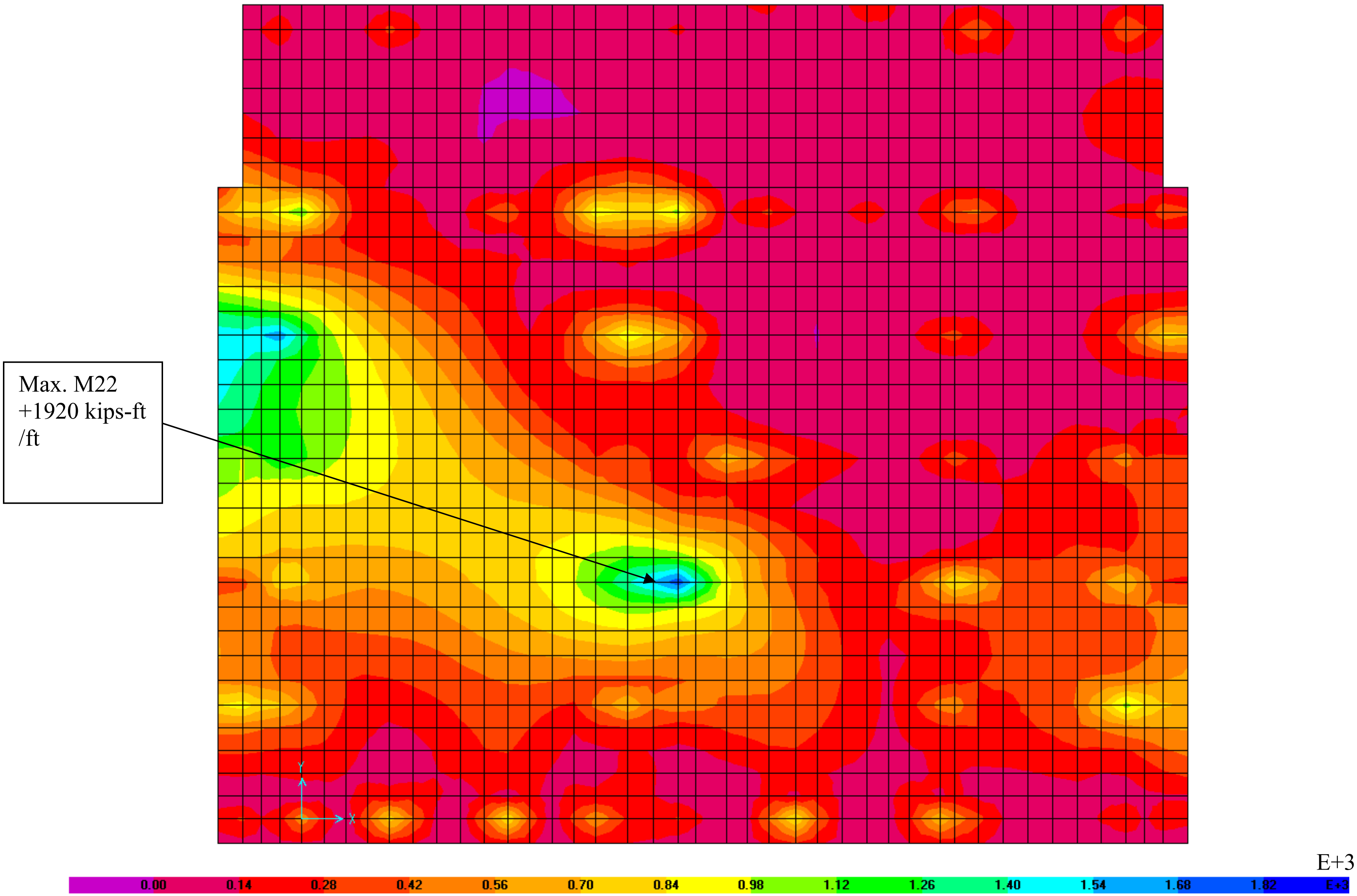

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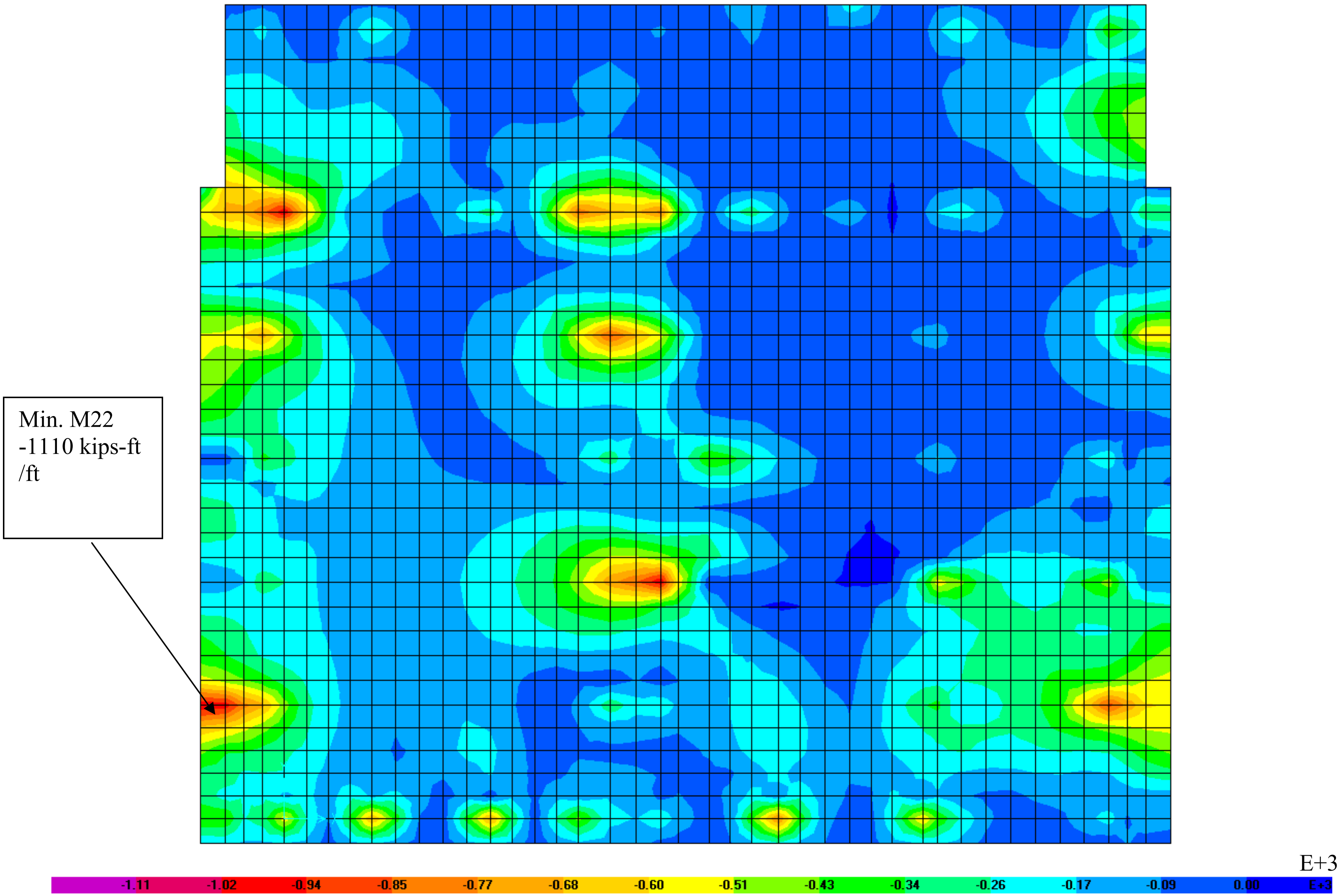

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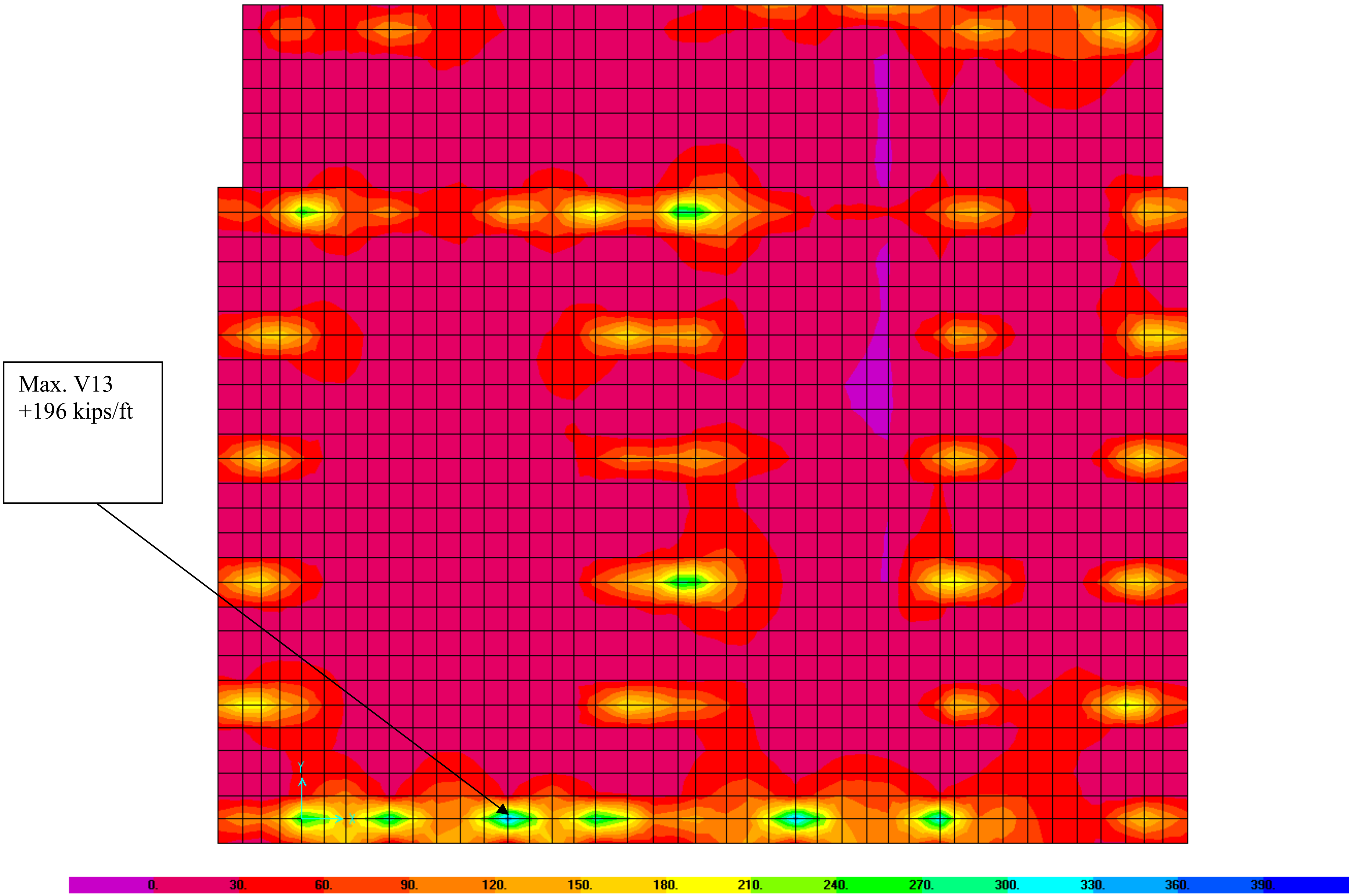

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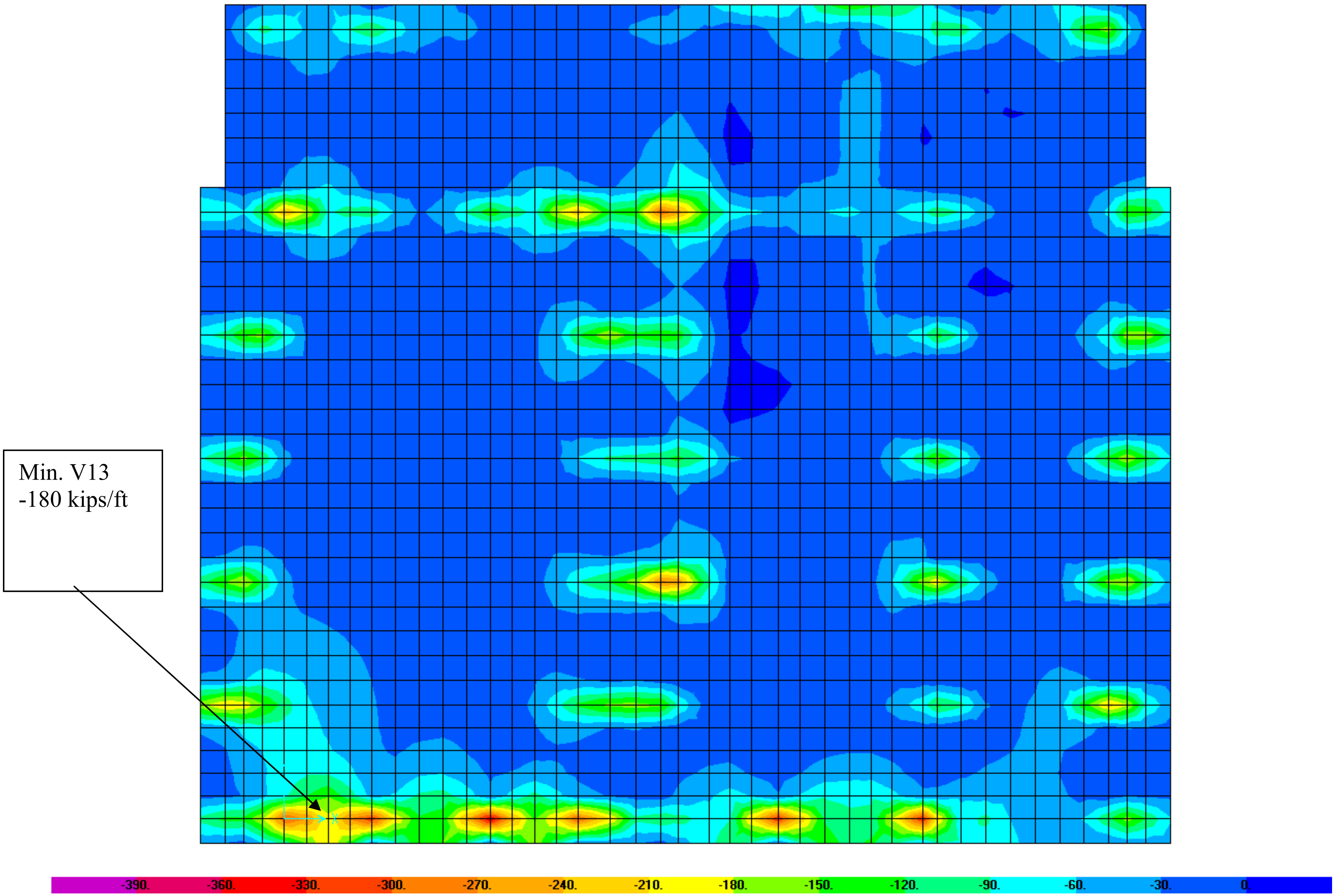

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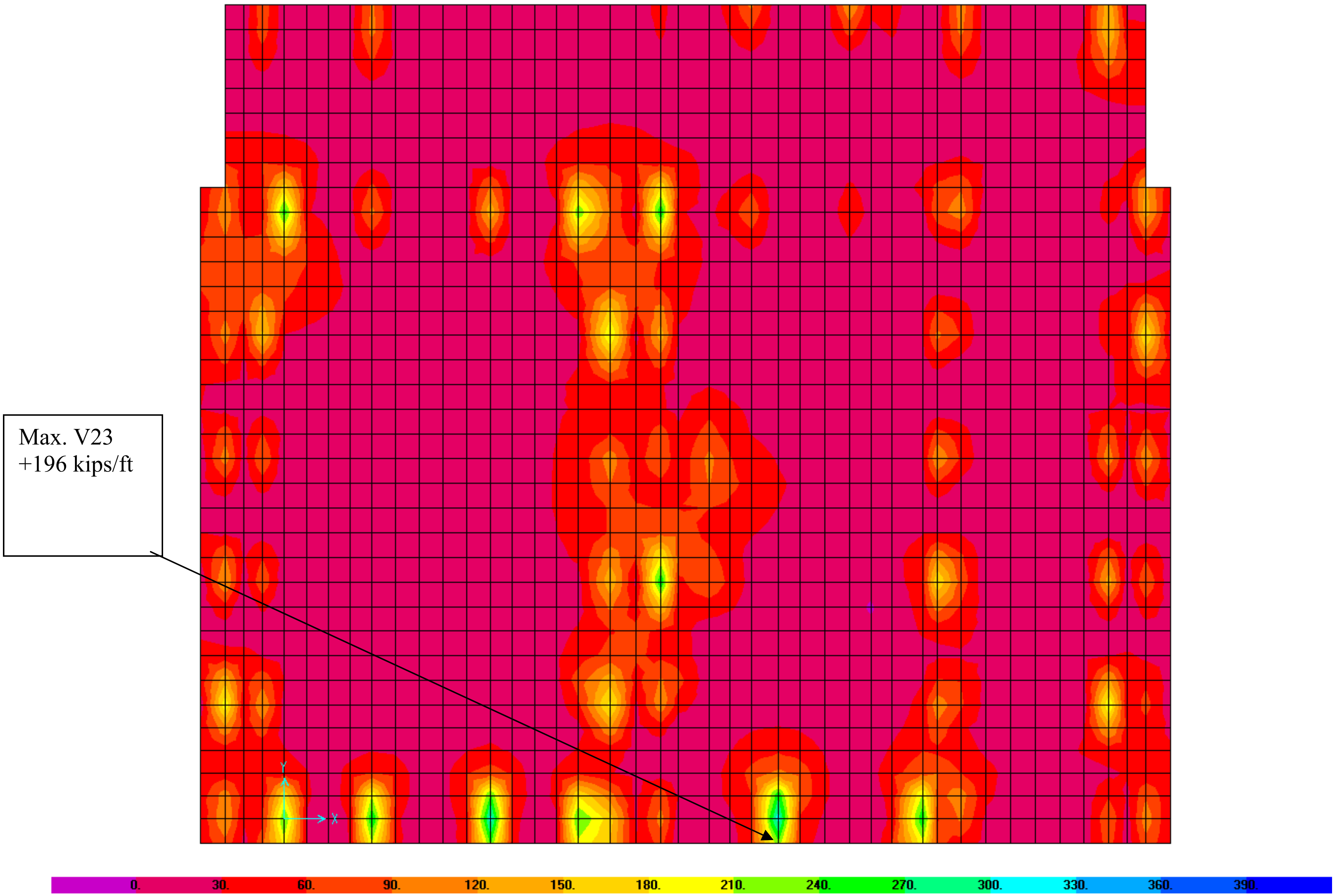

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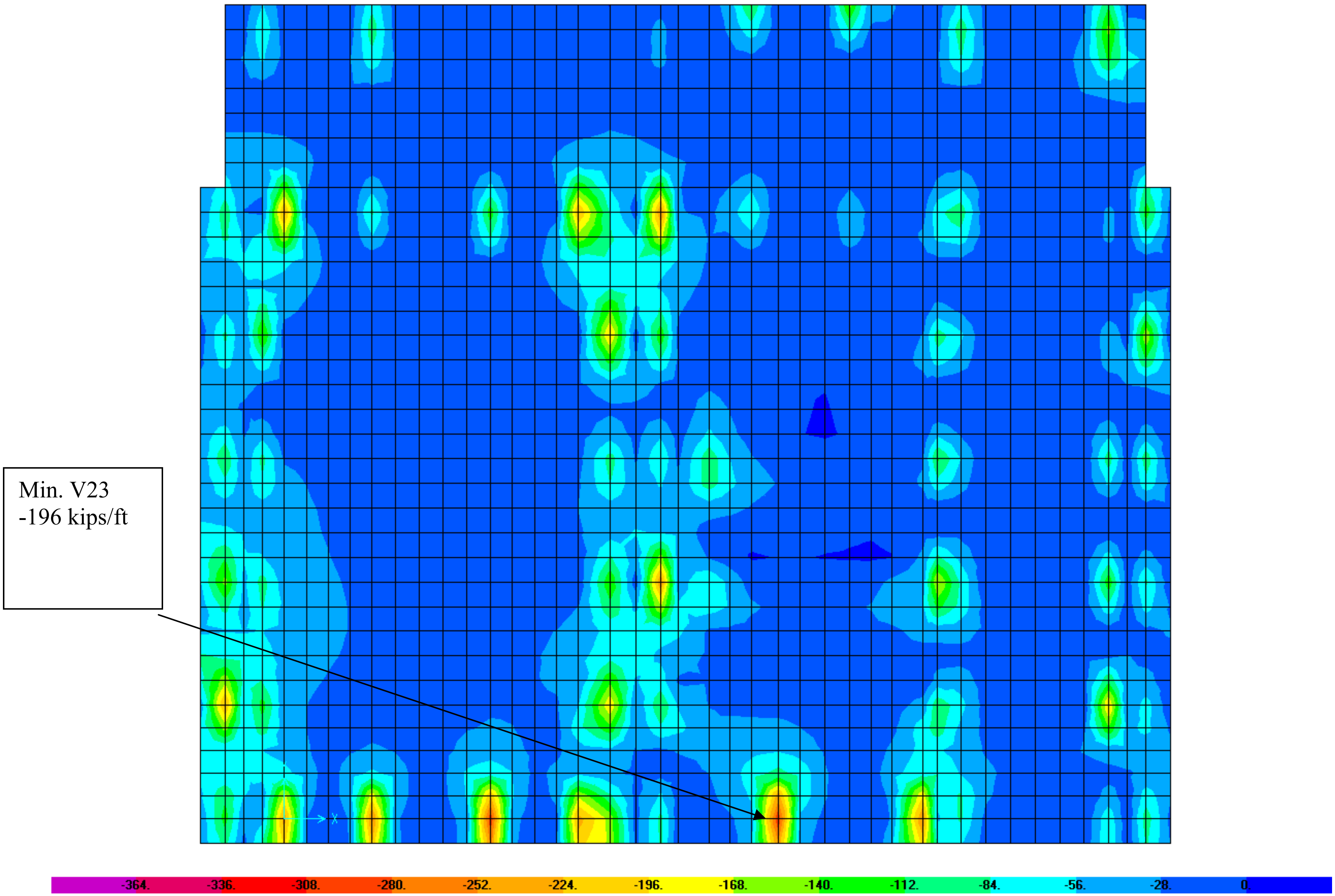

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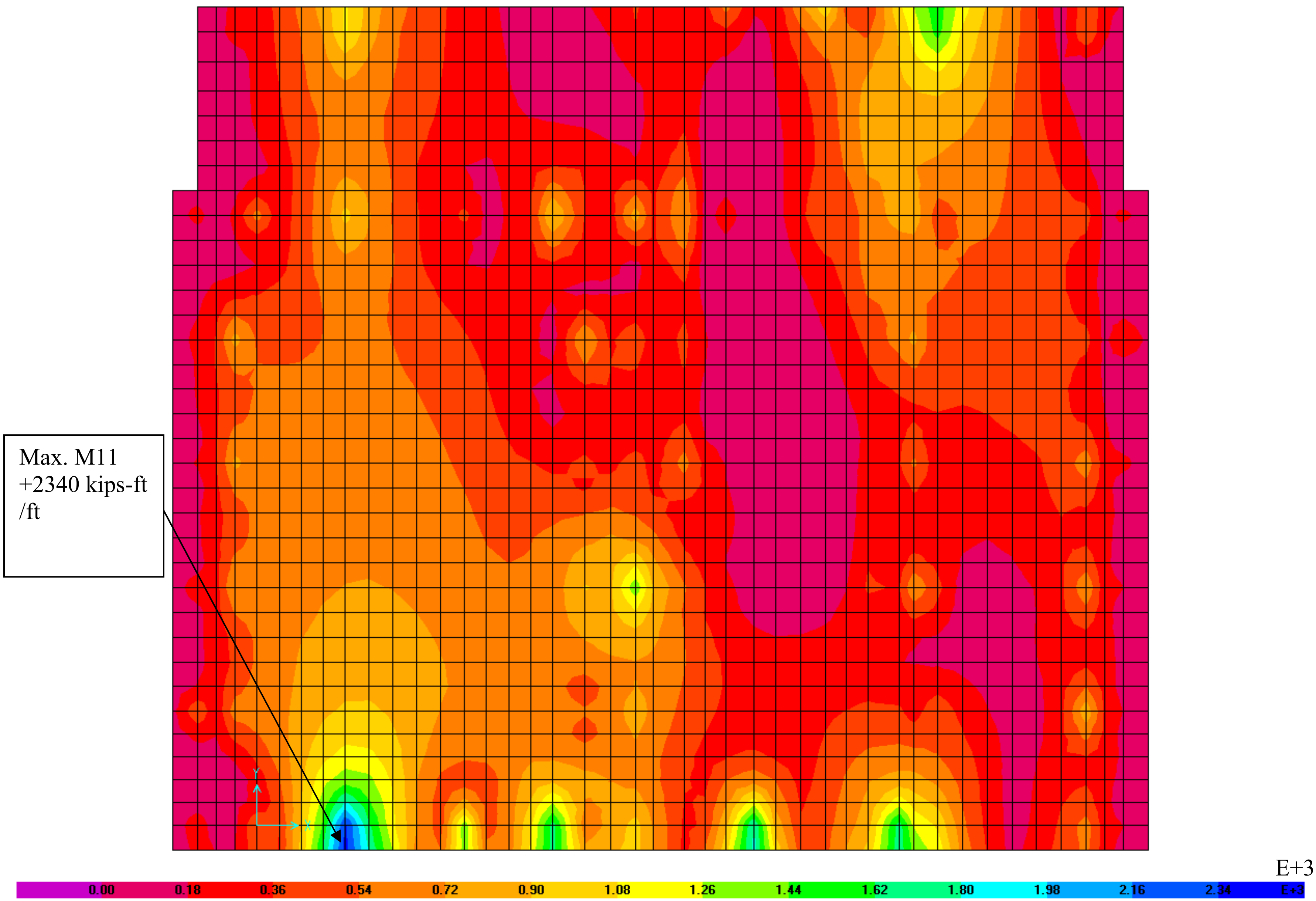

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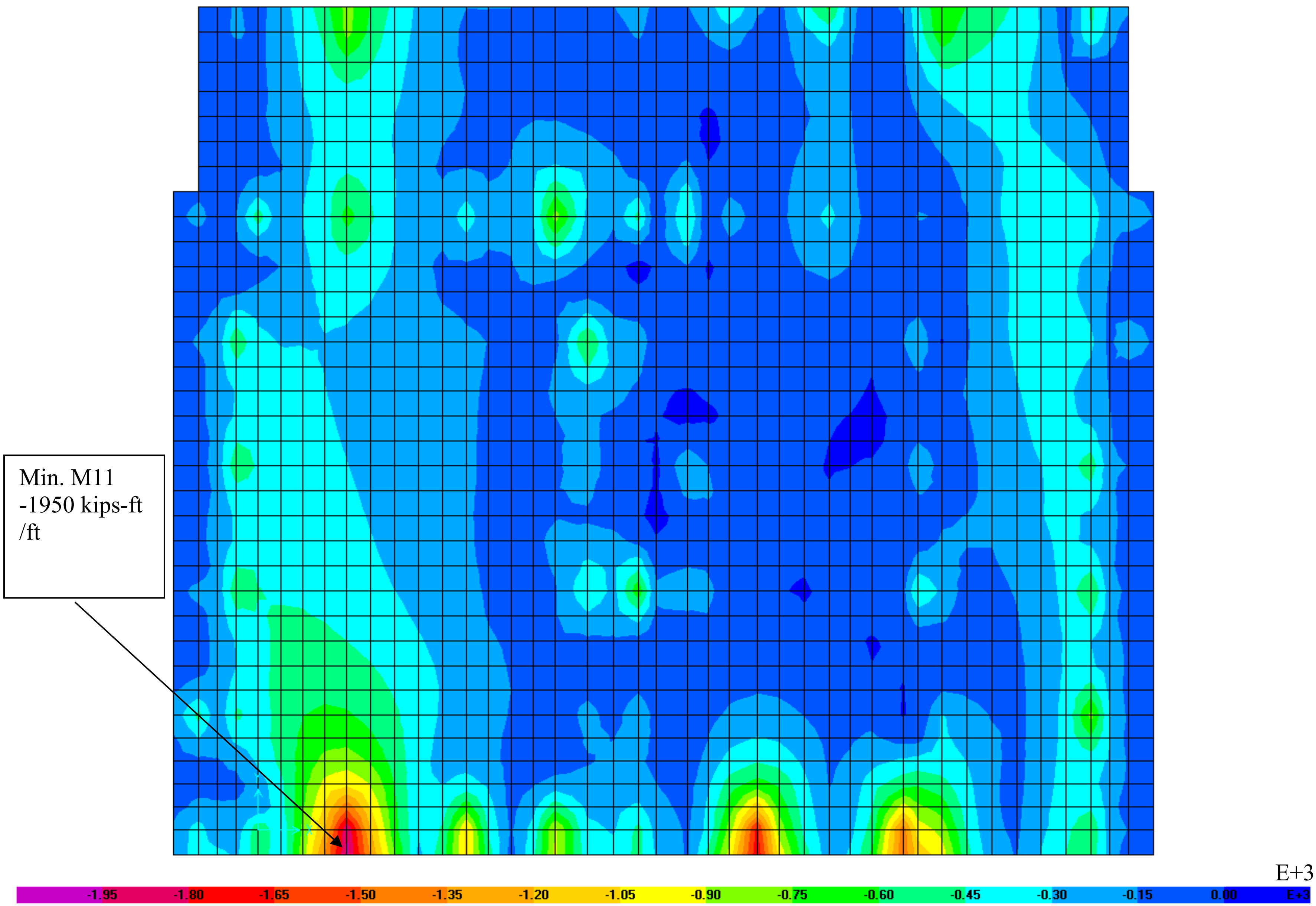

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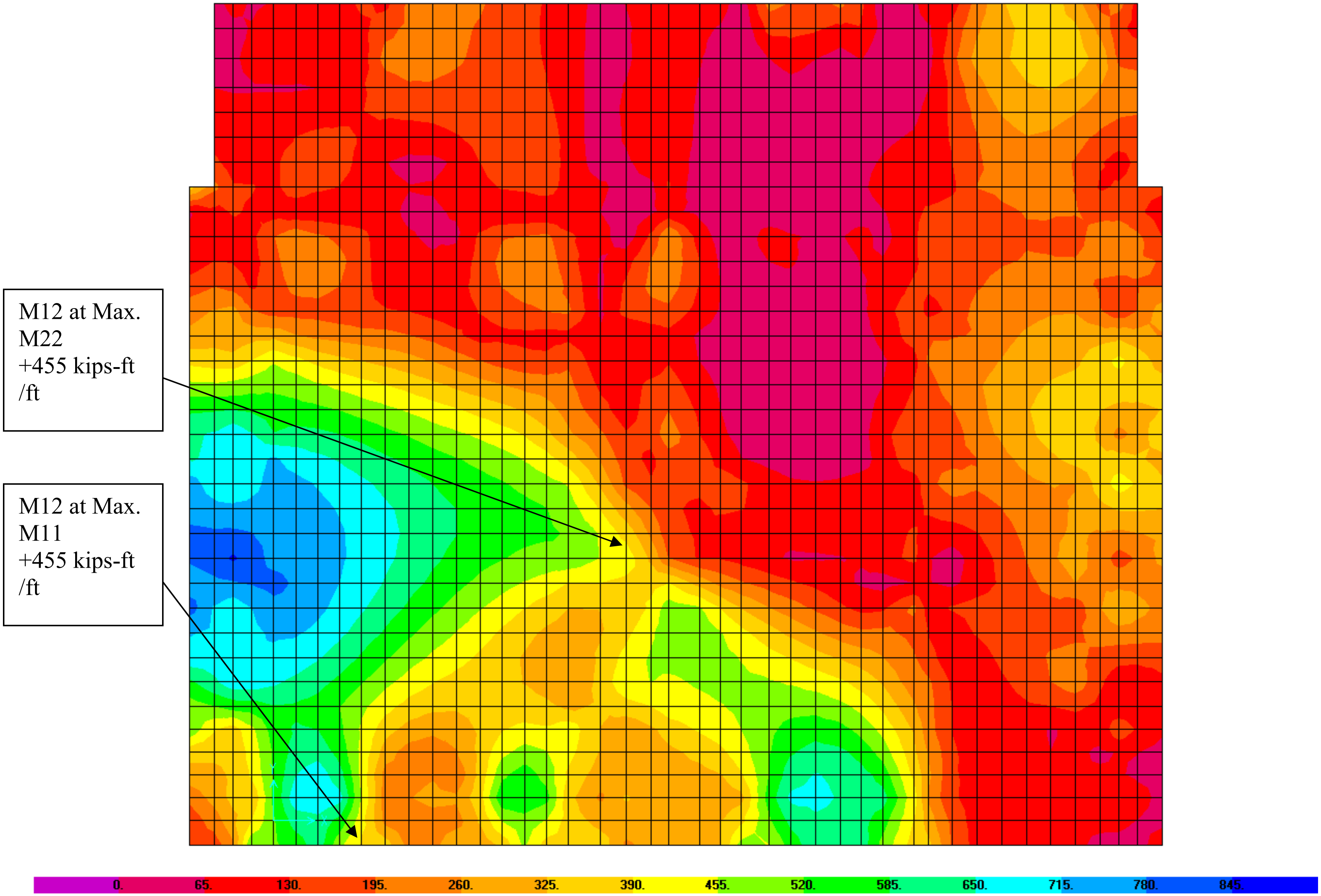

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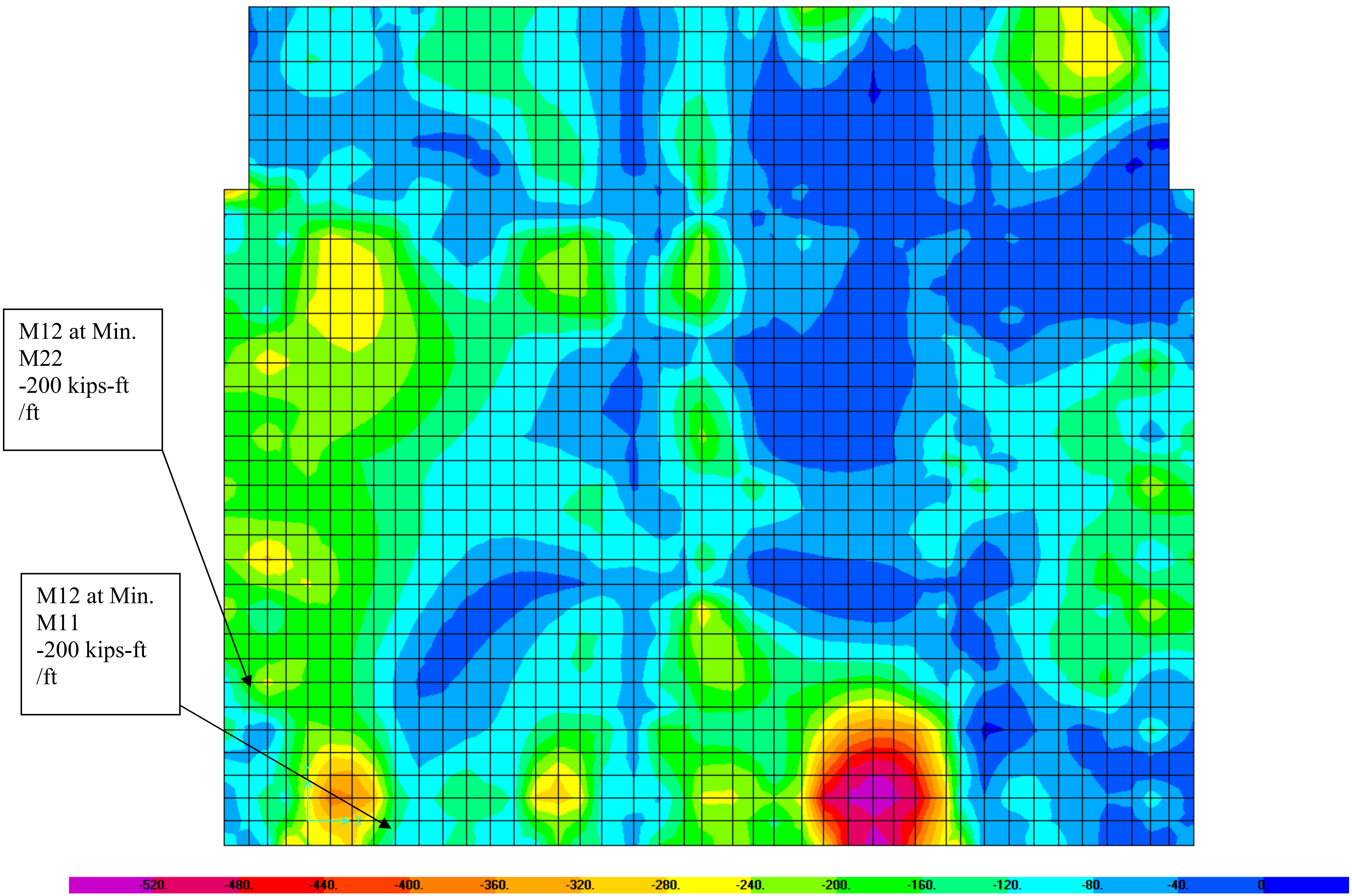

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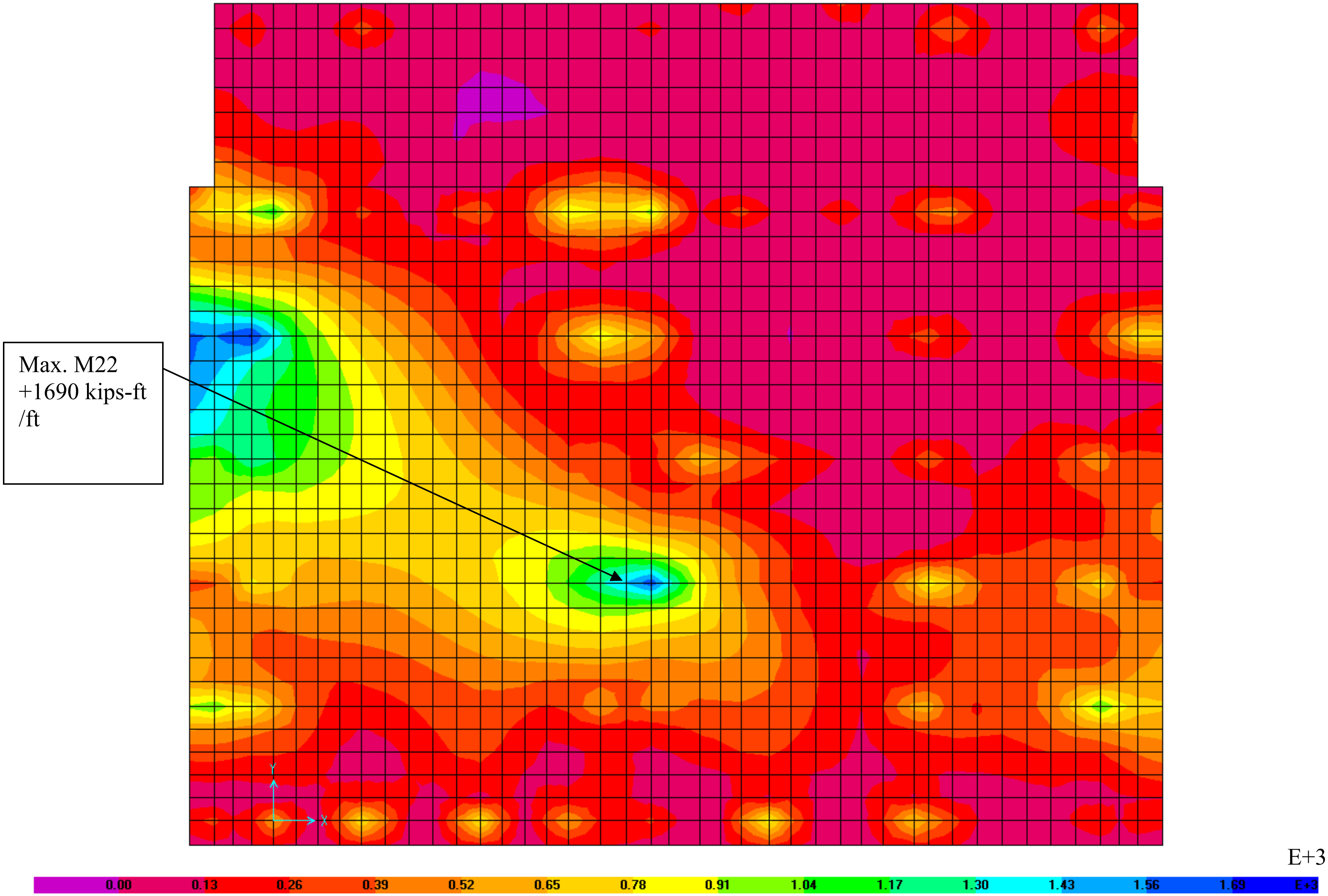

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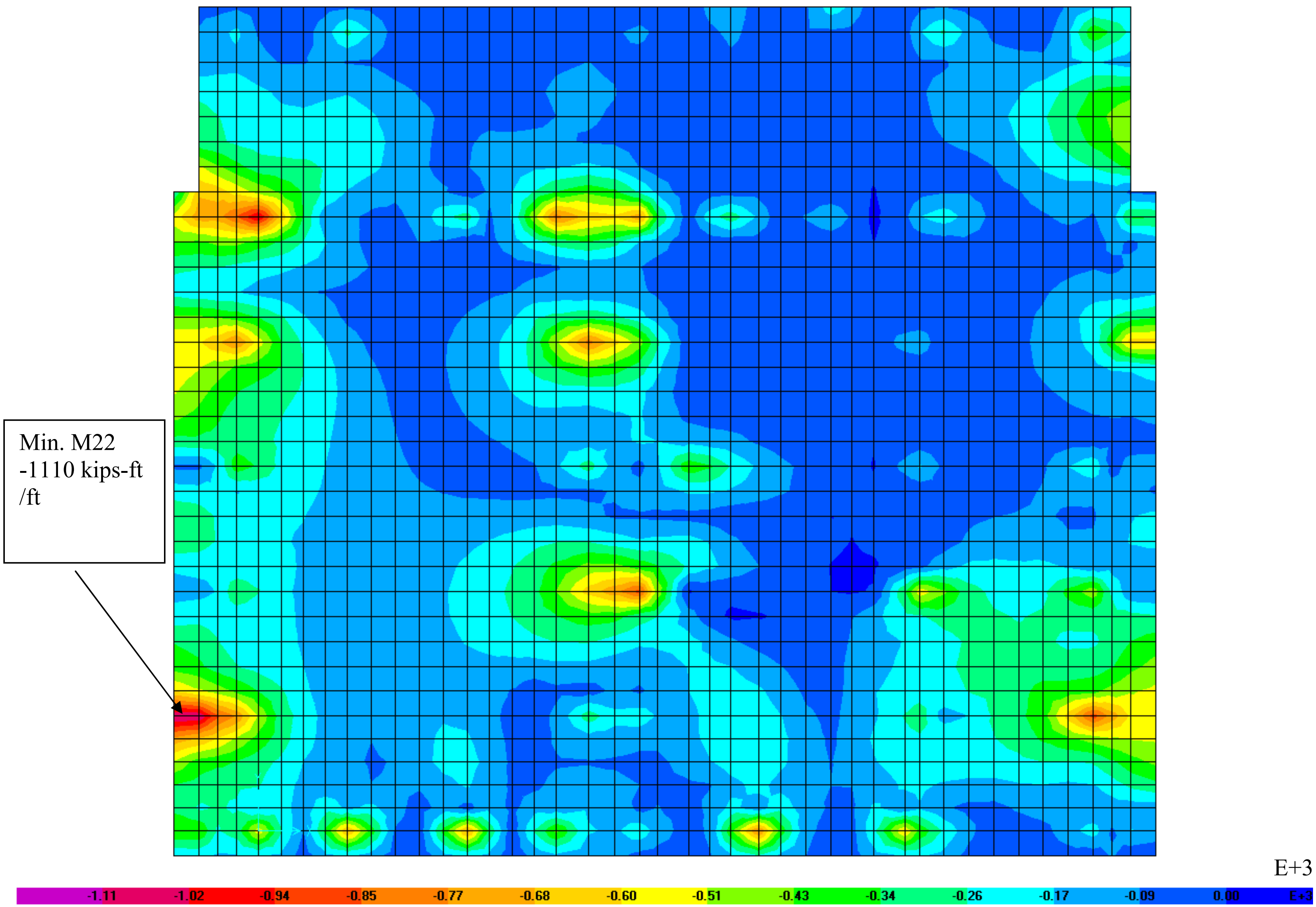

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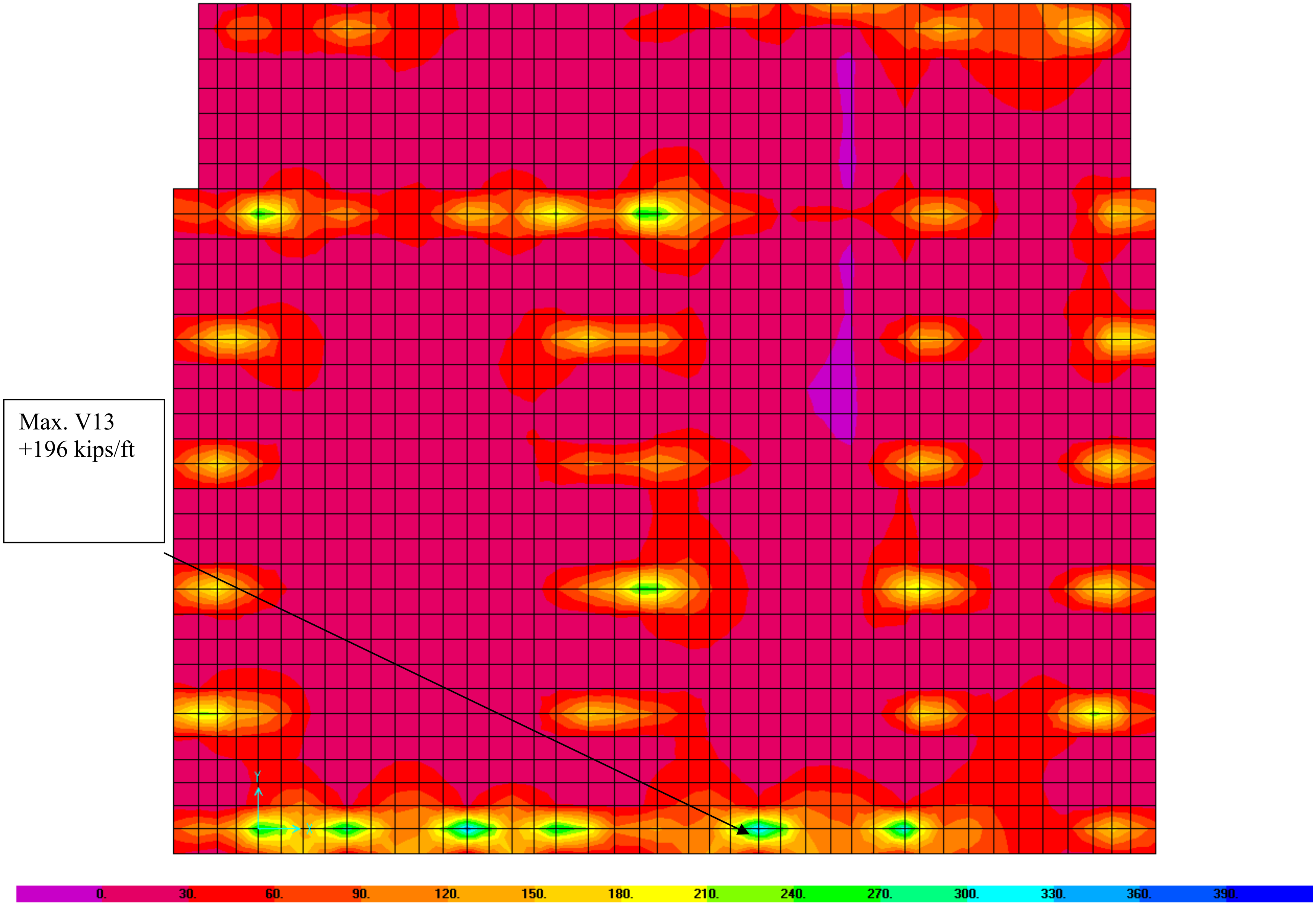

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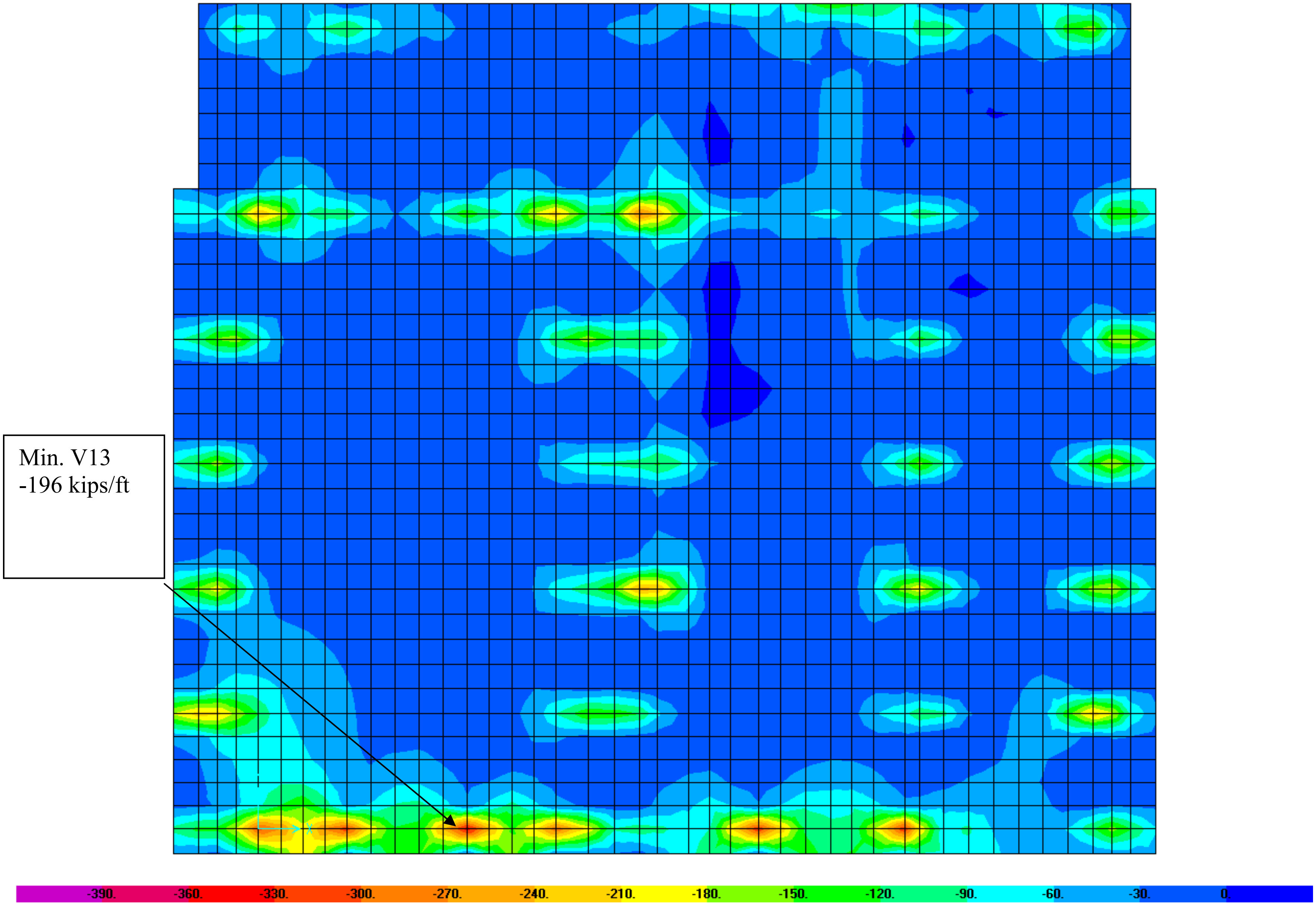

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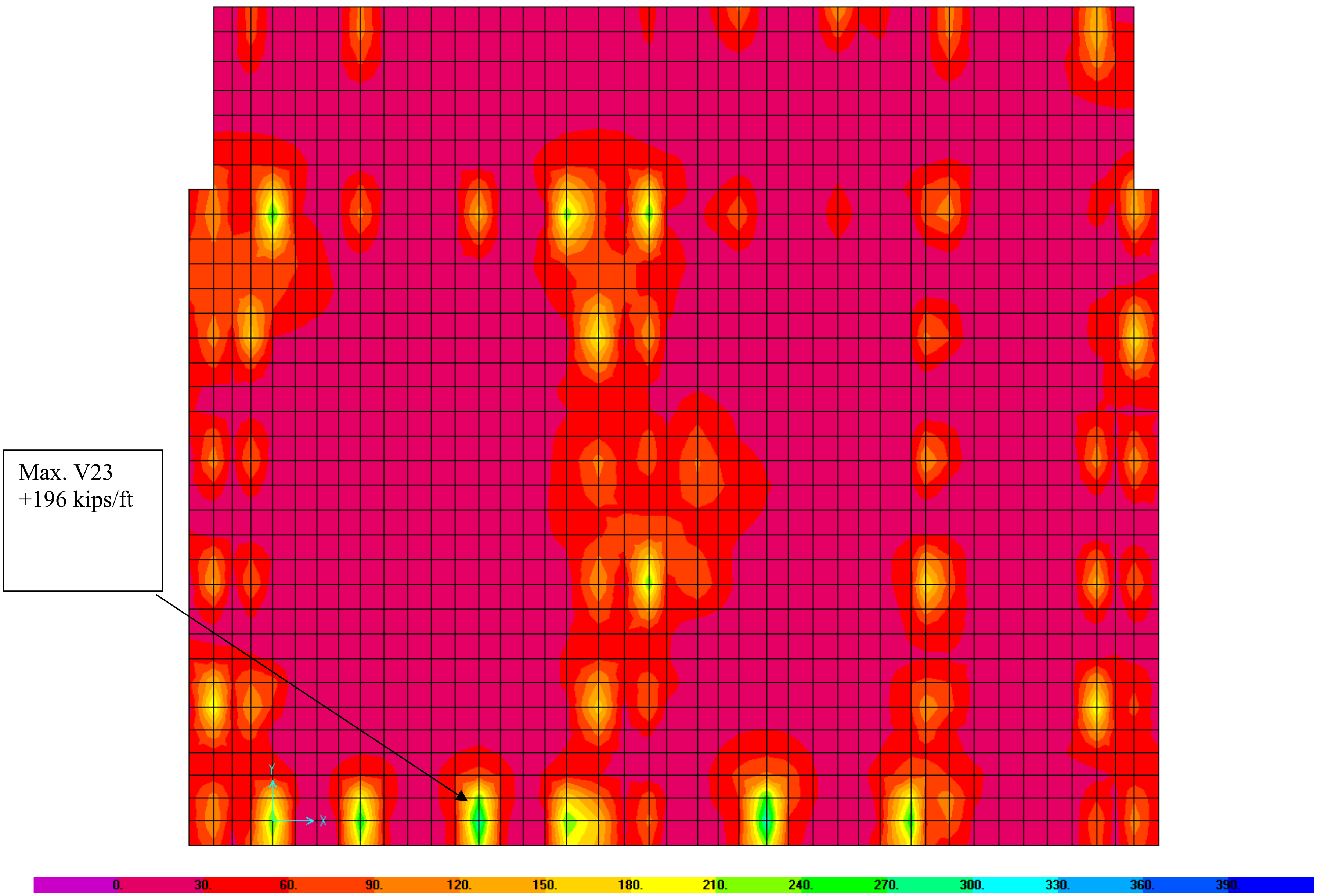

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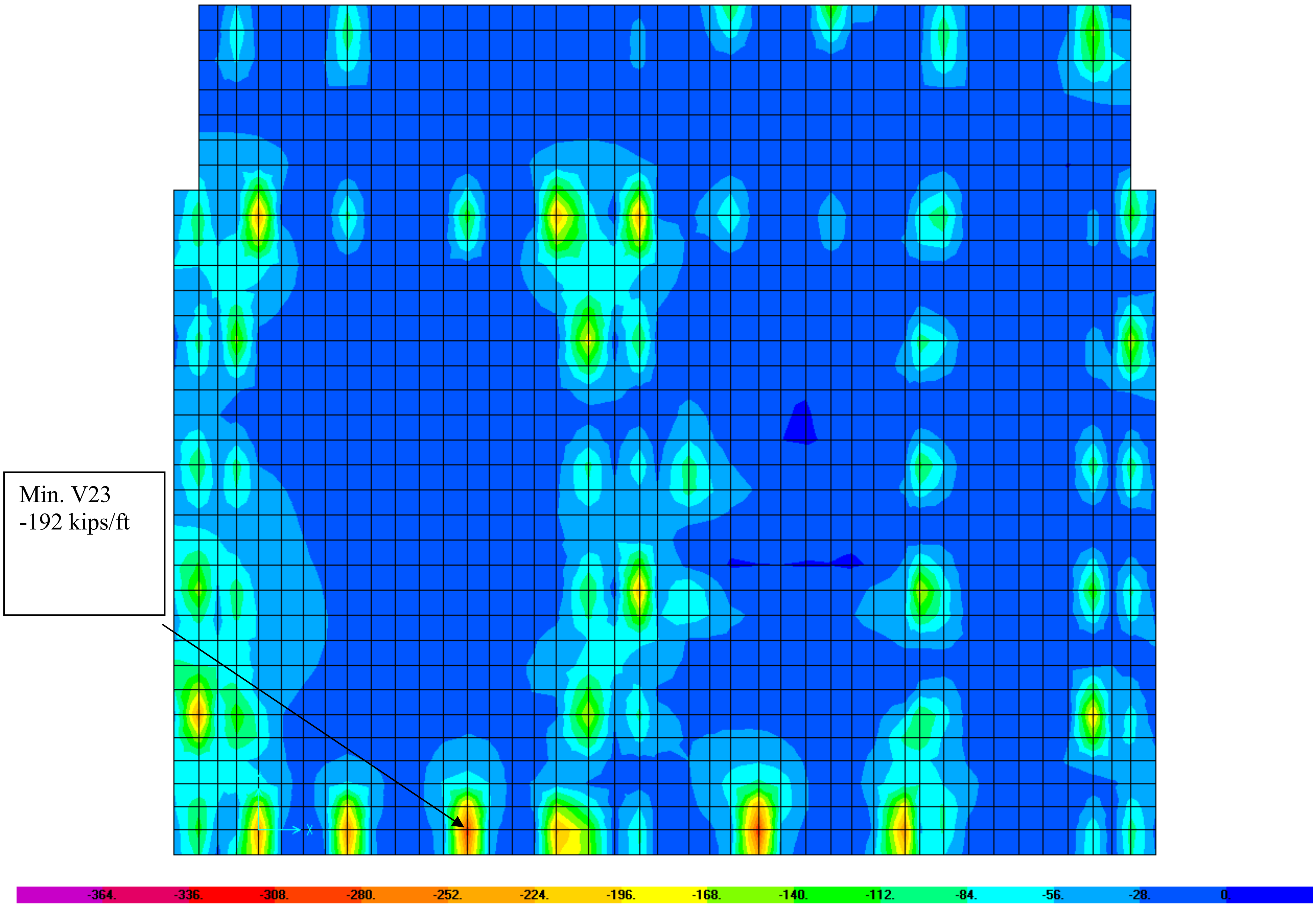

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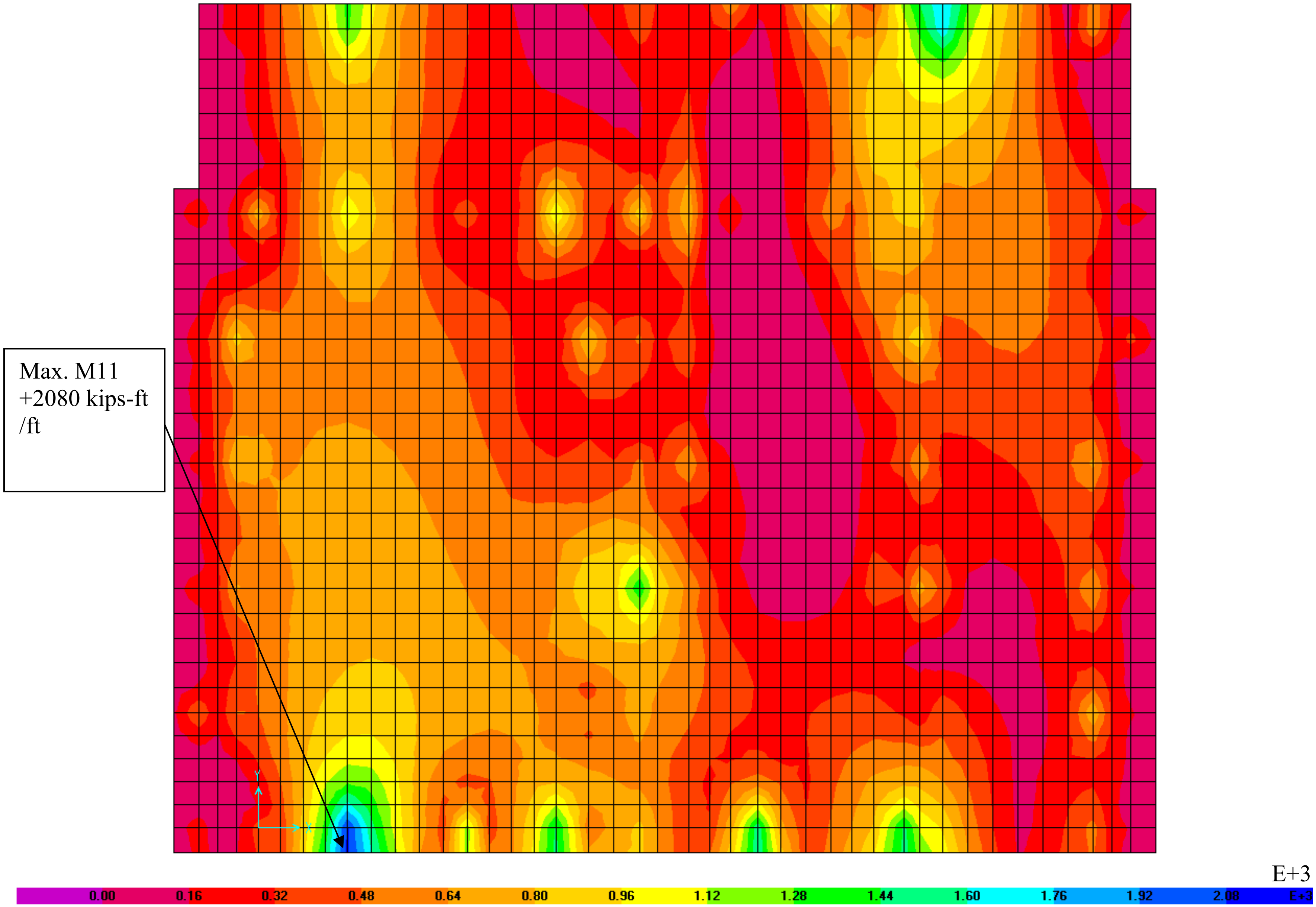

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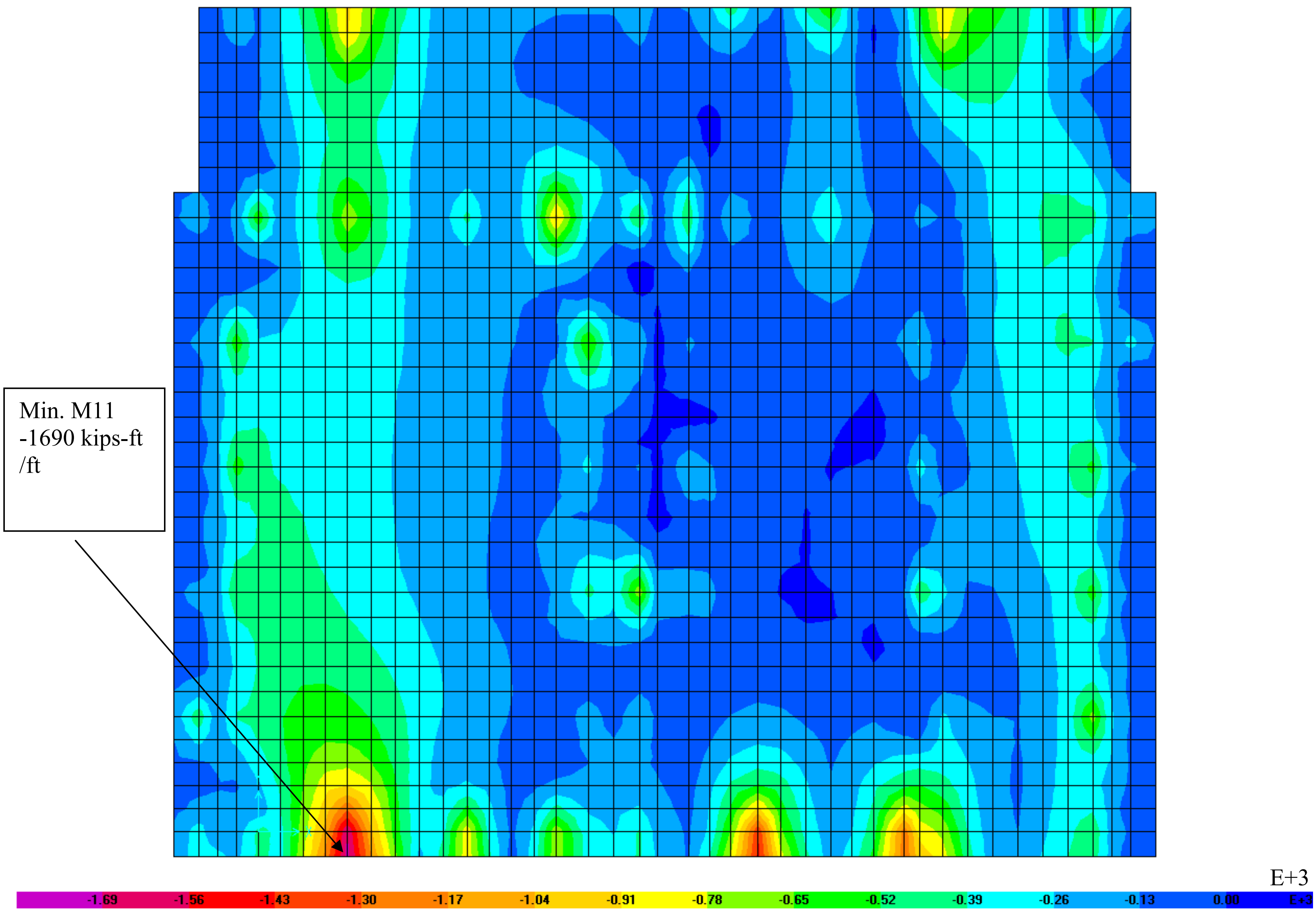

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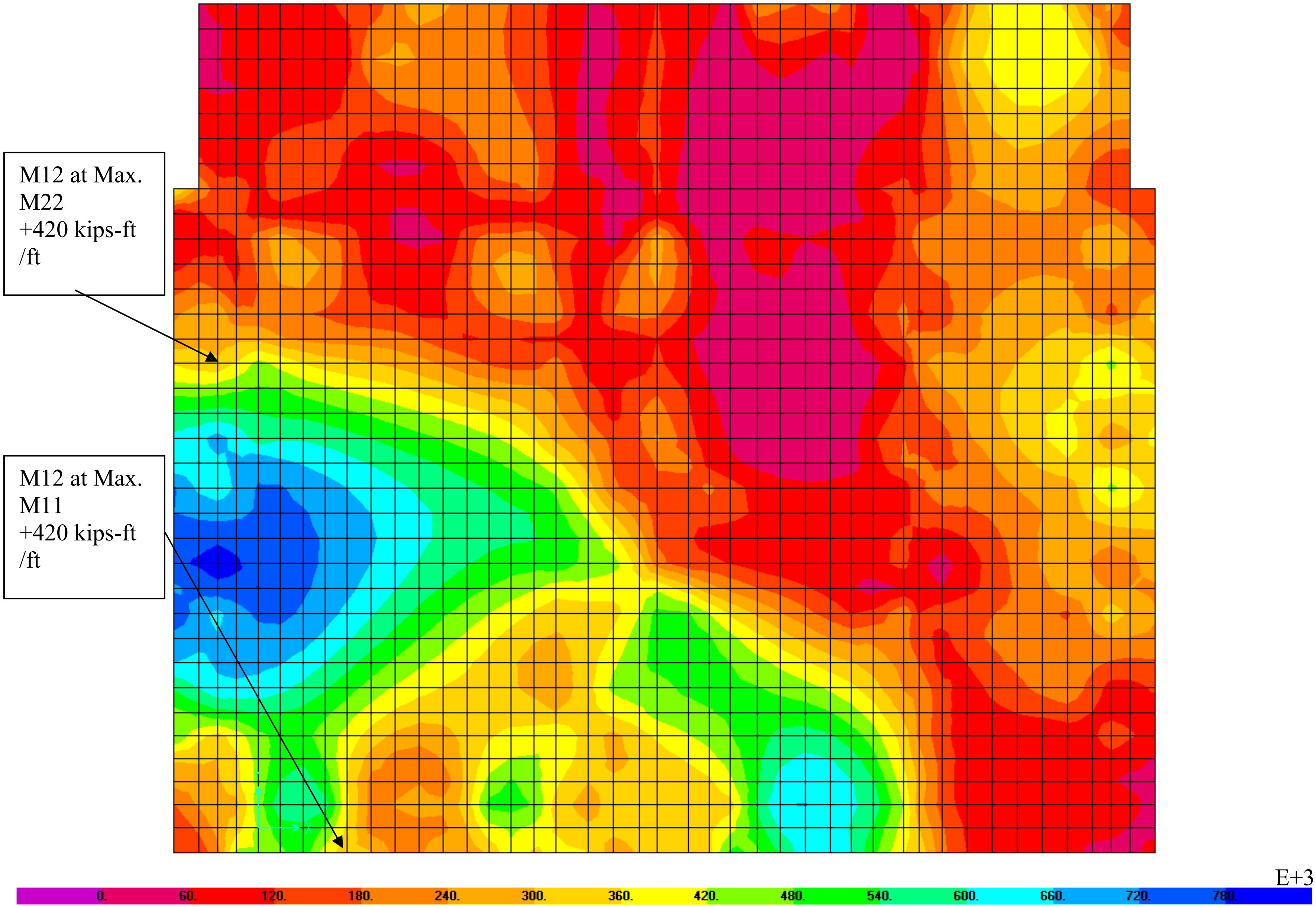

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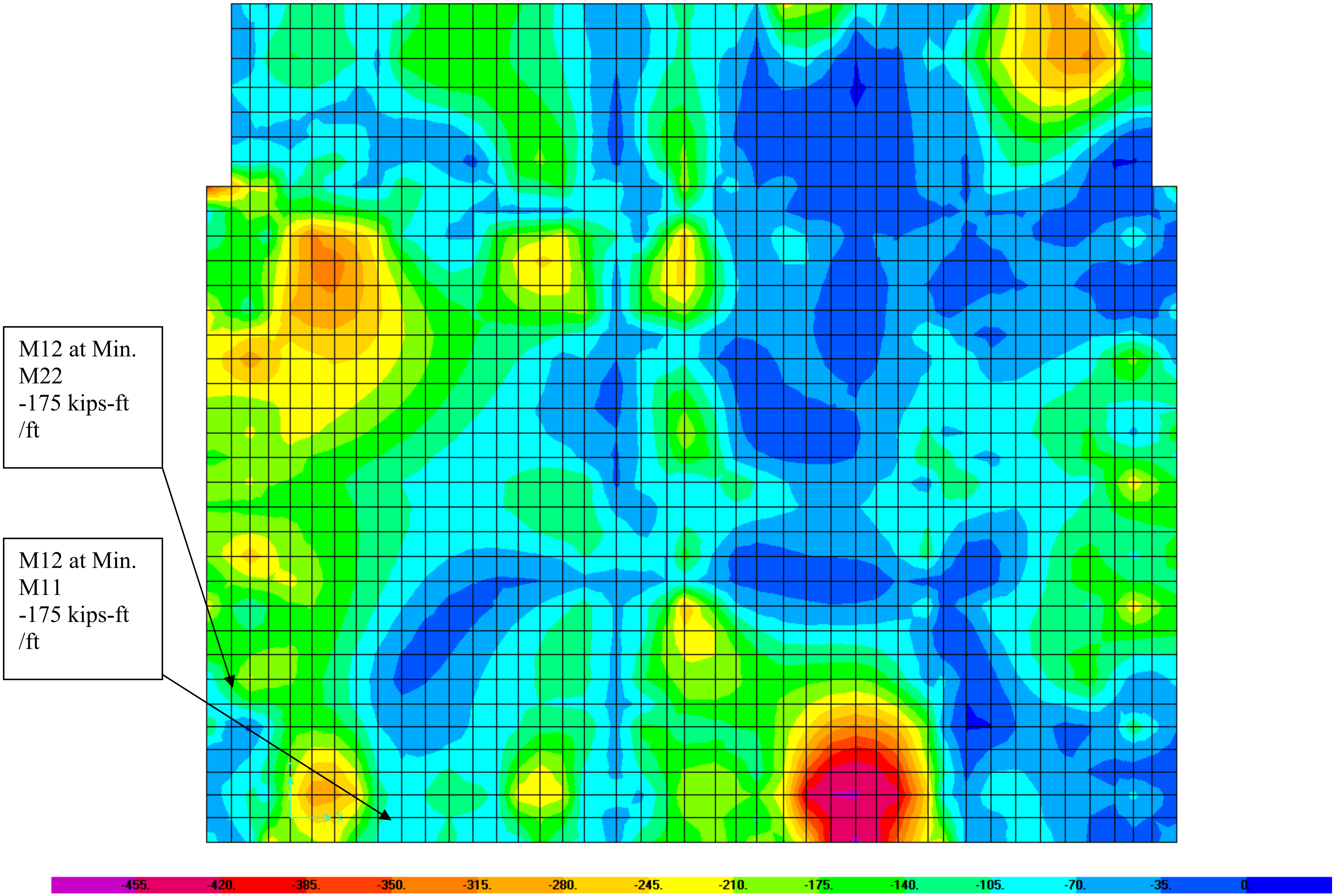

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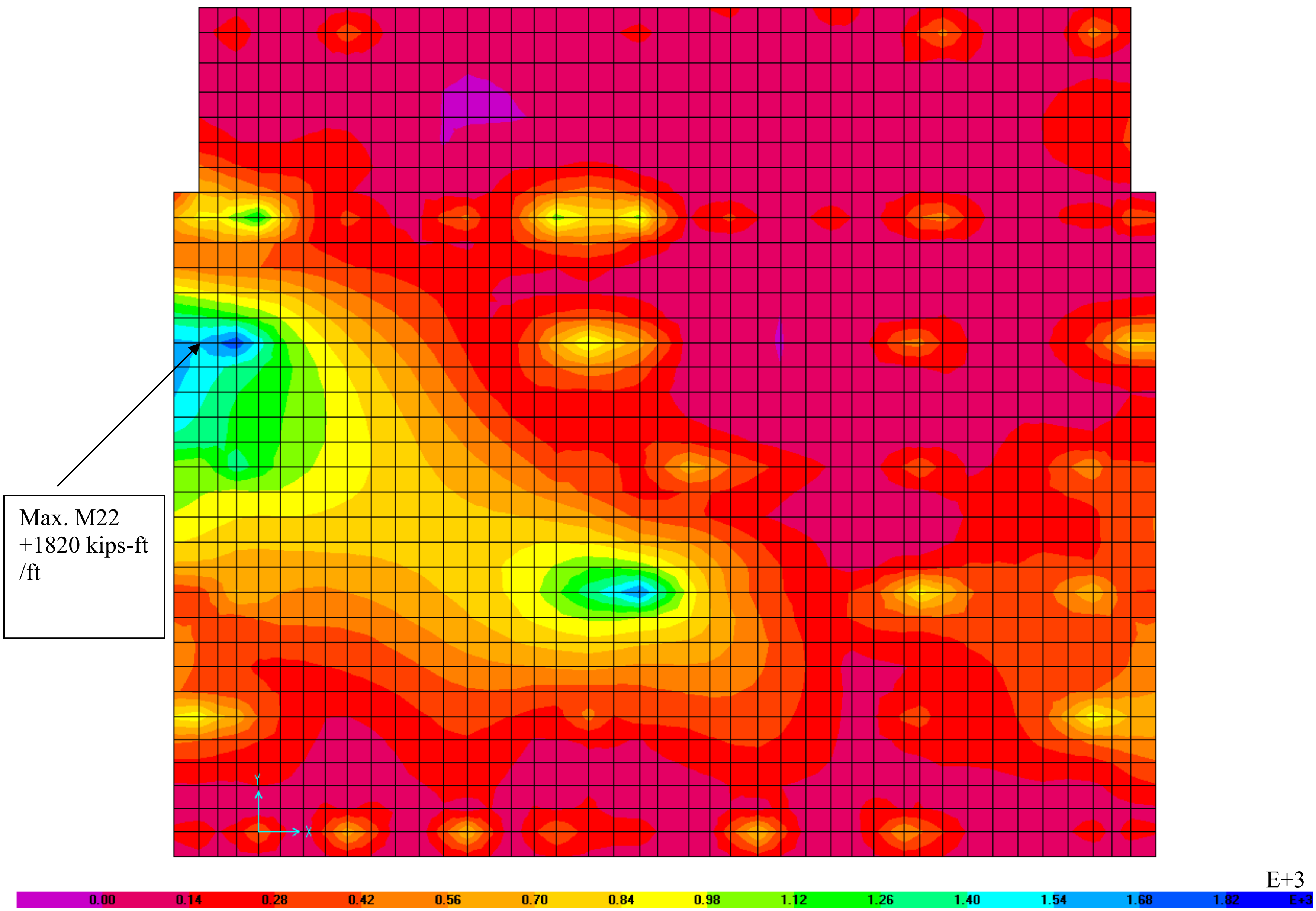

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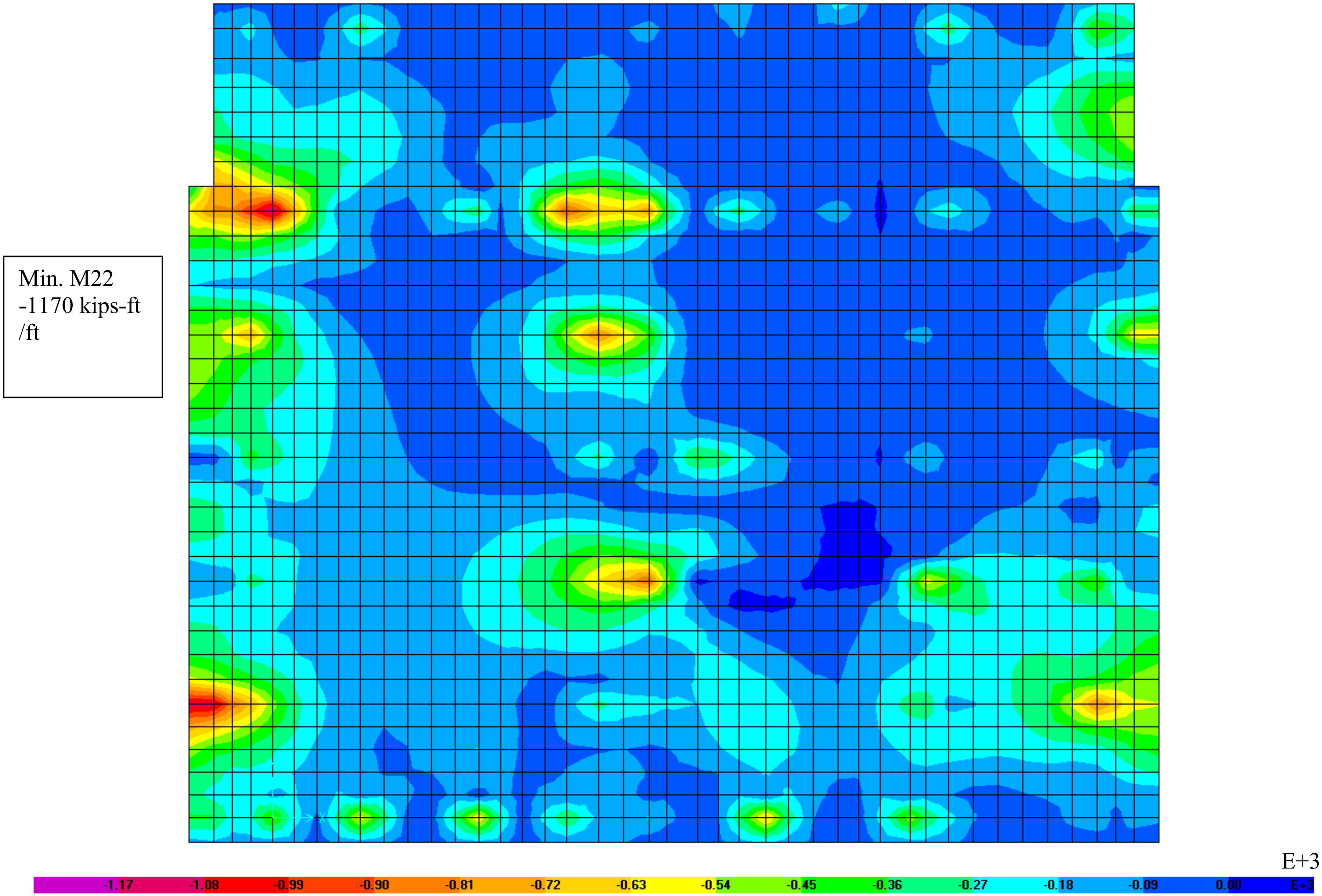

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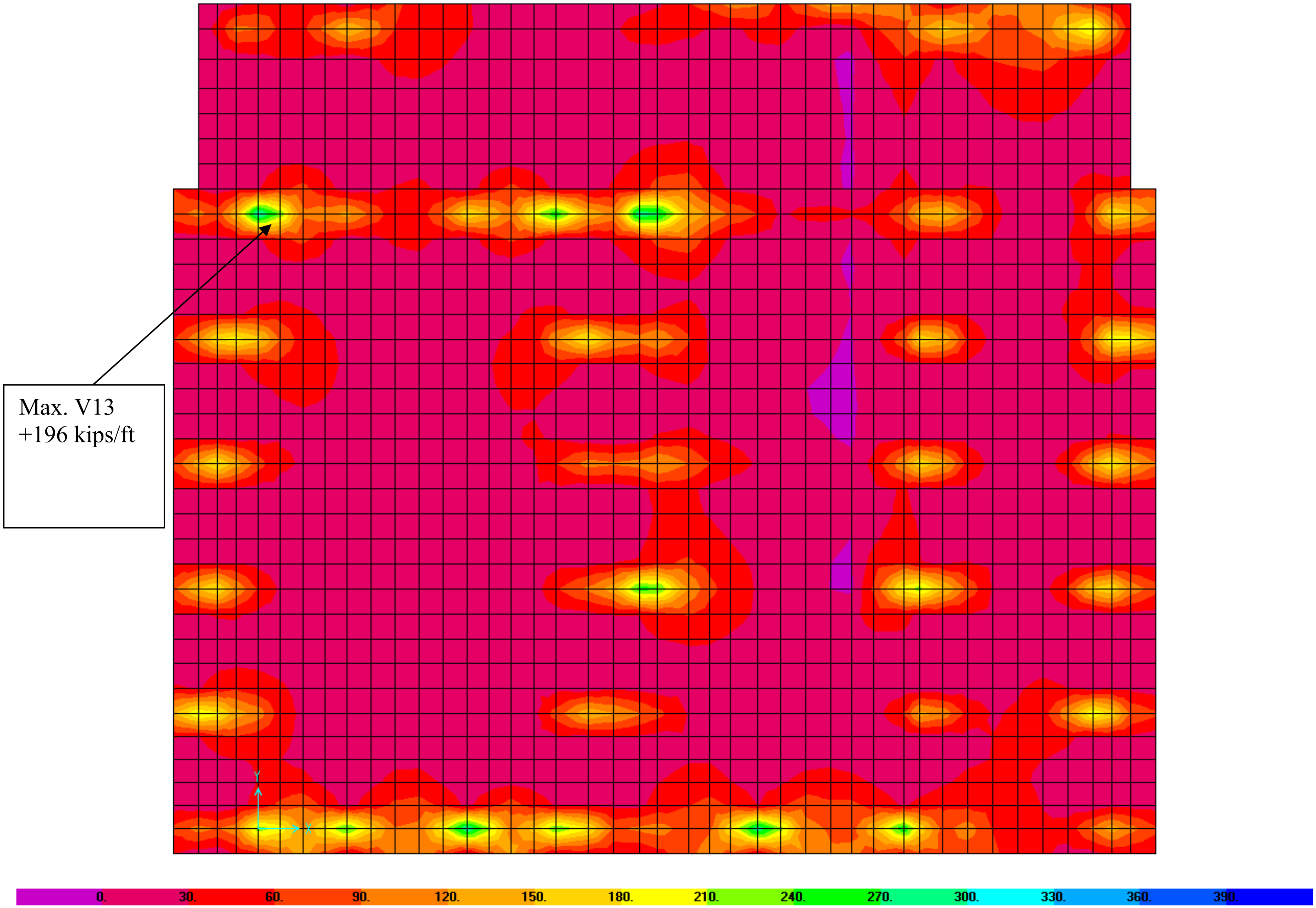

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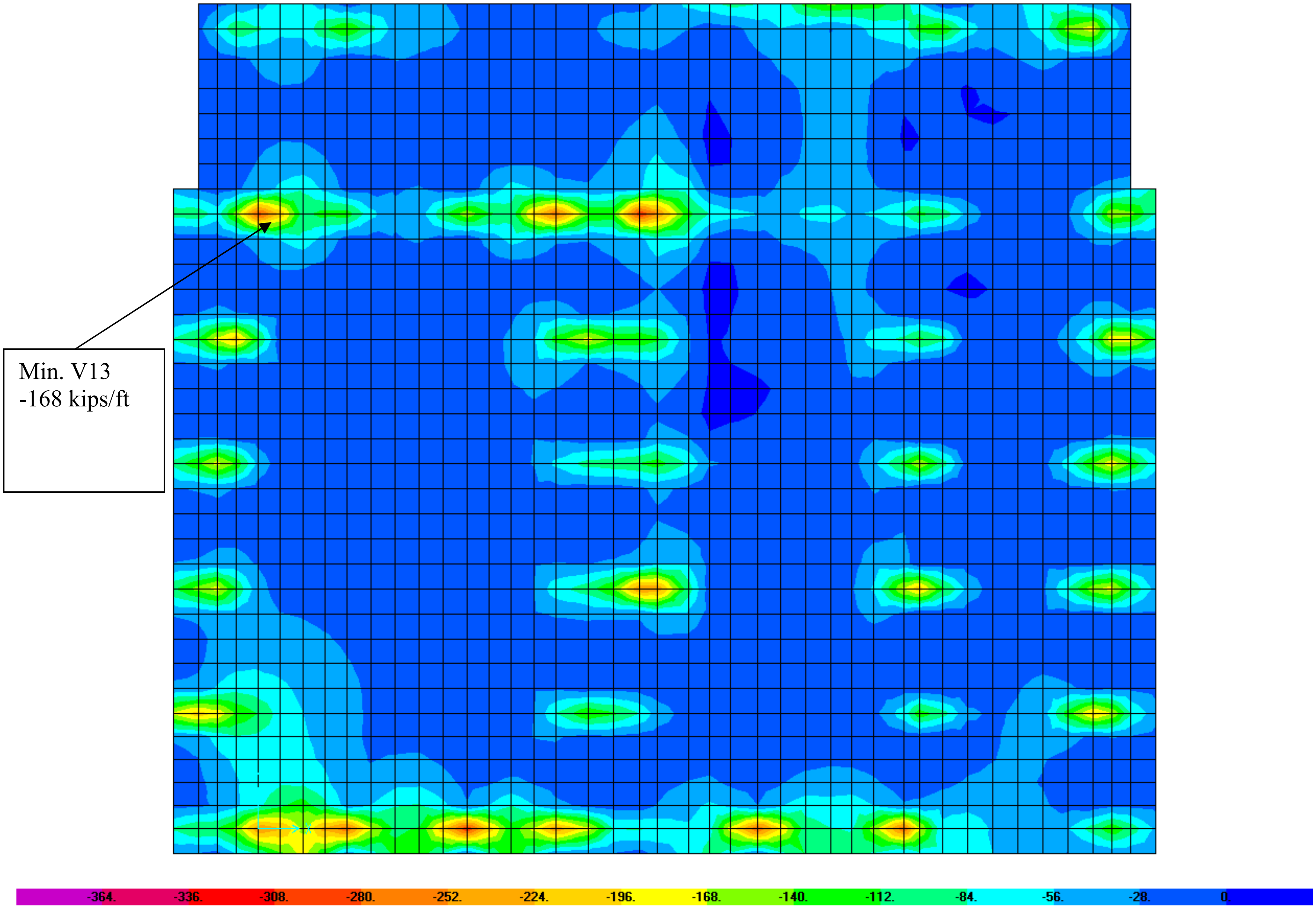

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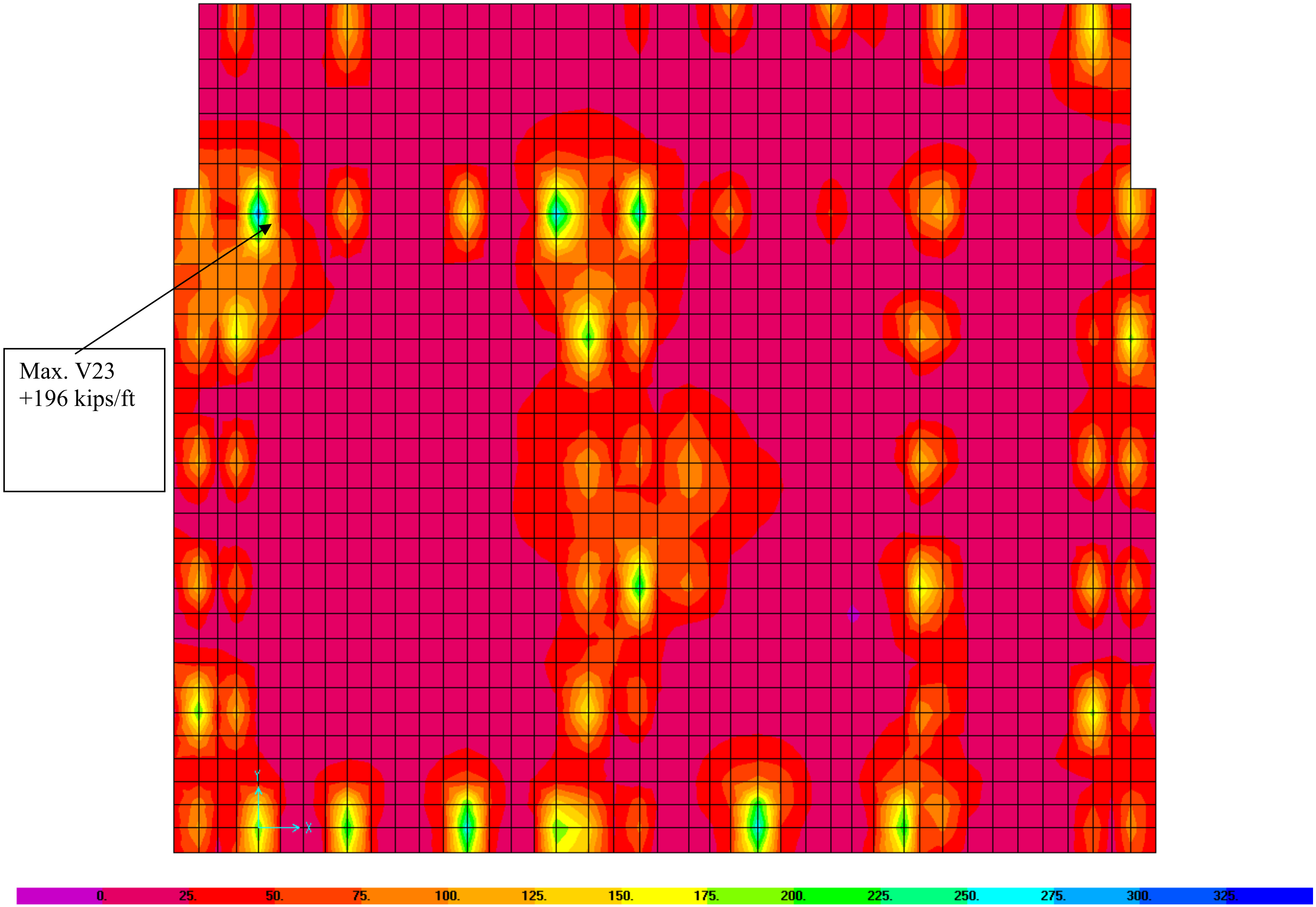

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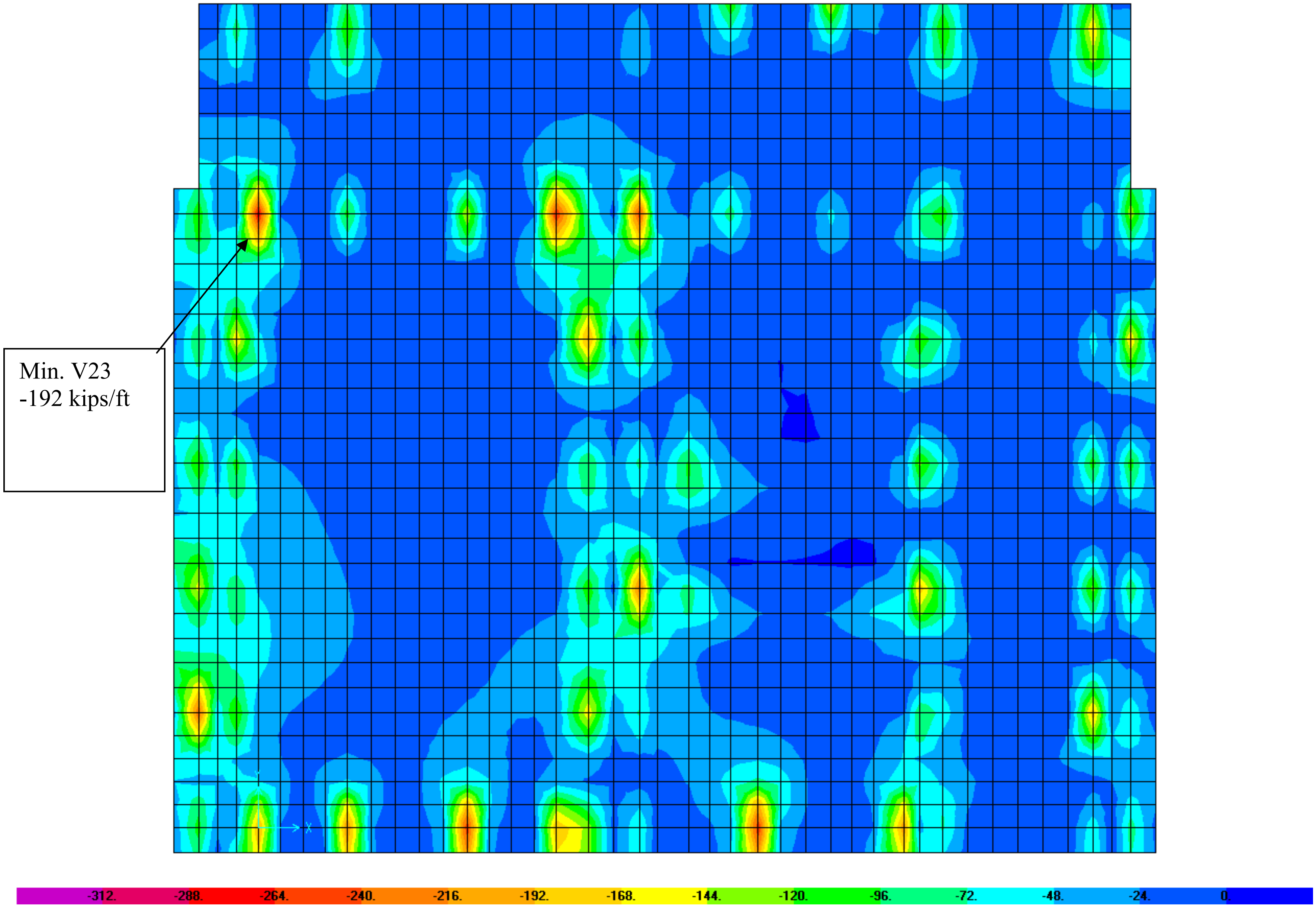

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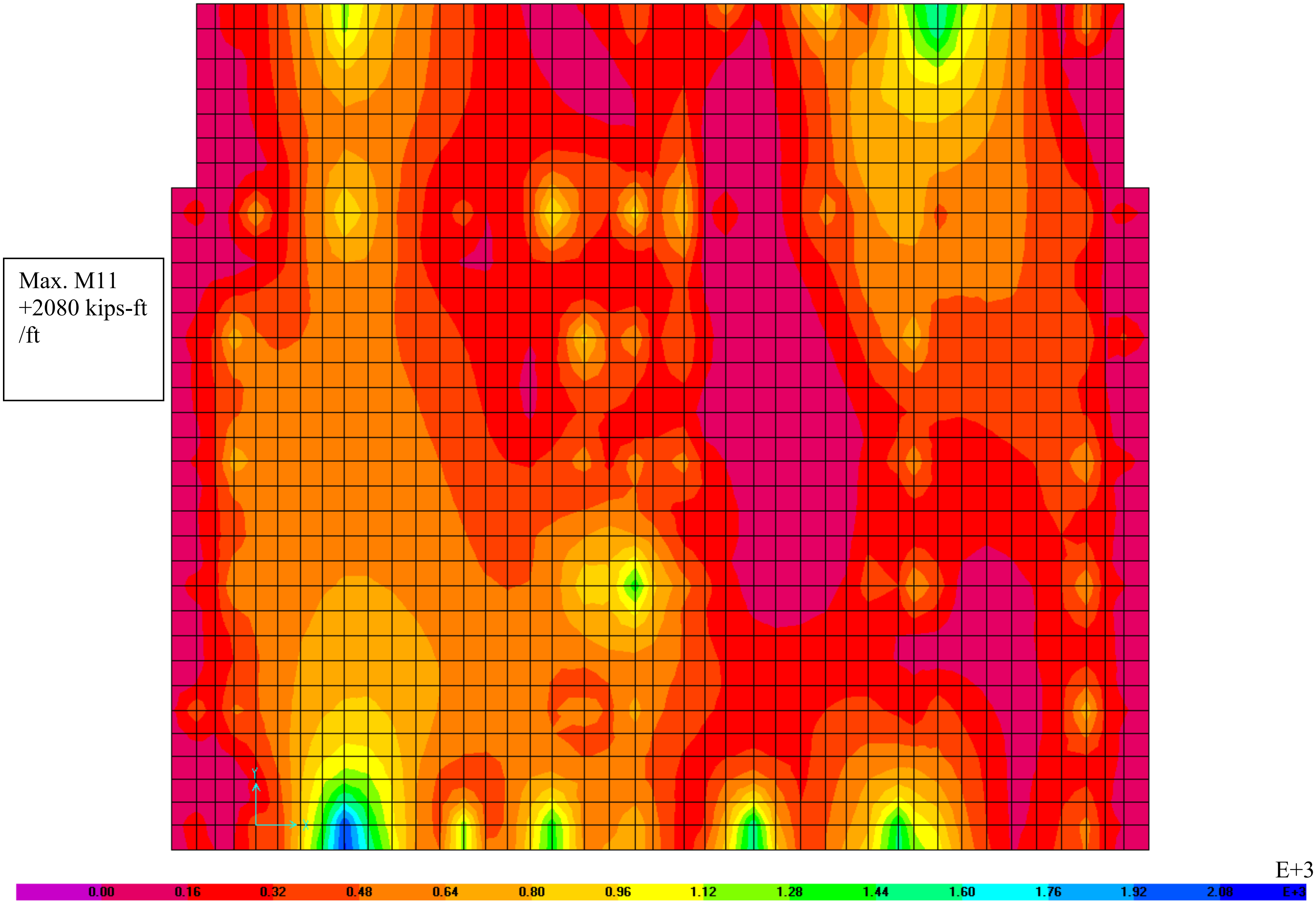
Max. M11
+2080 kips-ft
/ft



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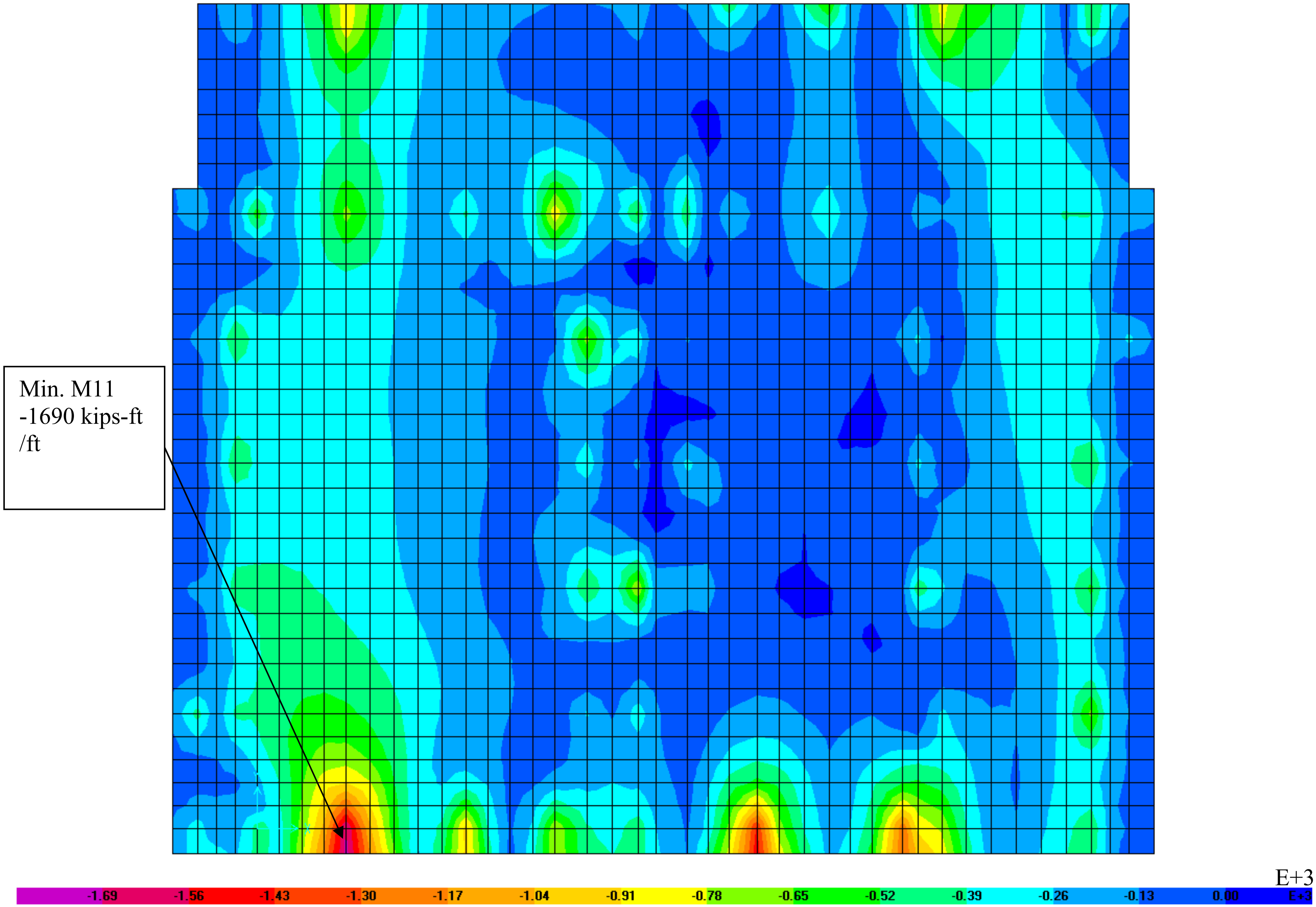

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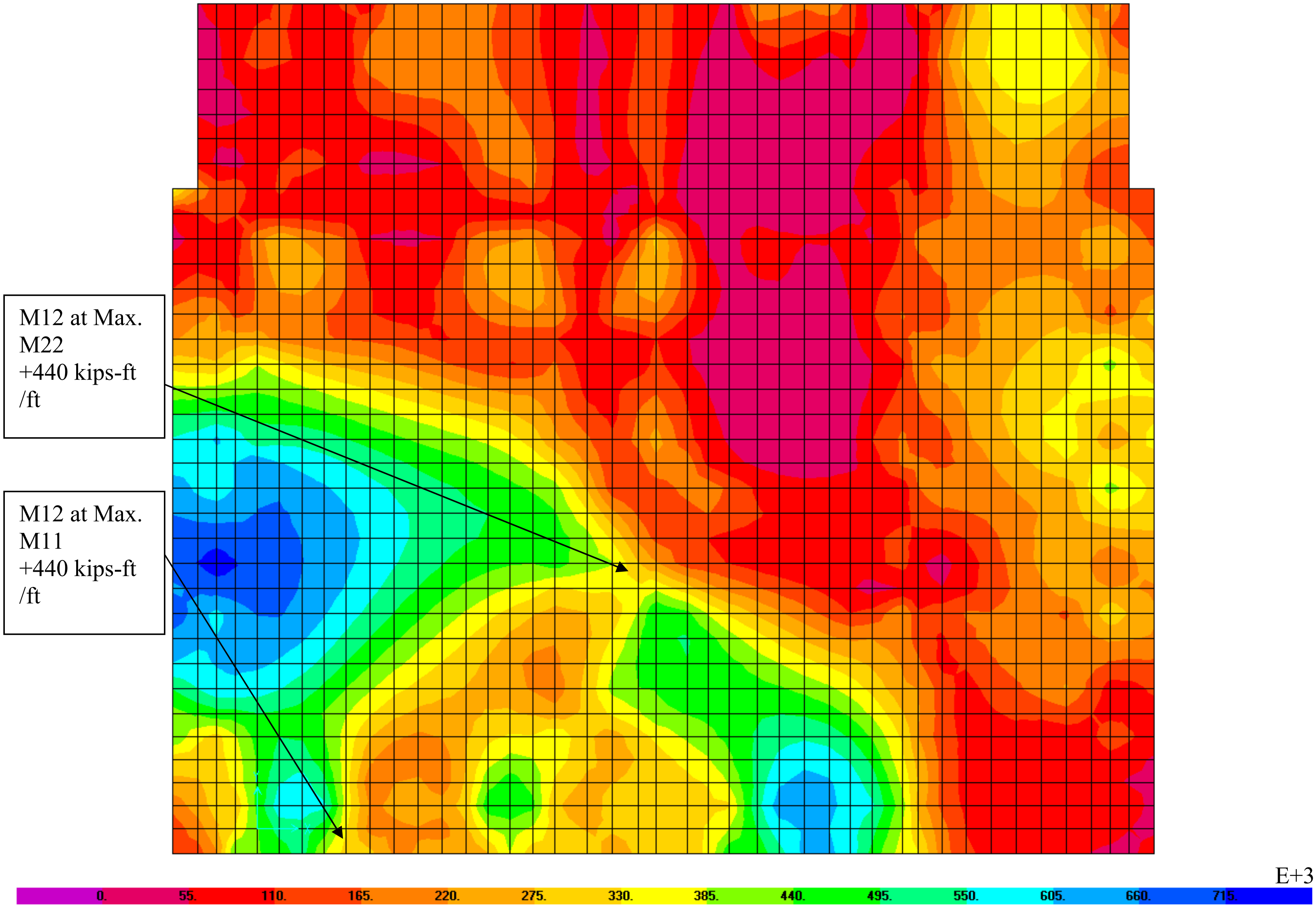

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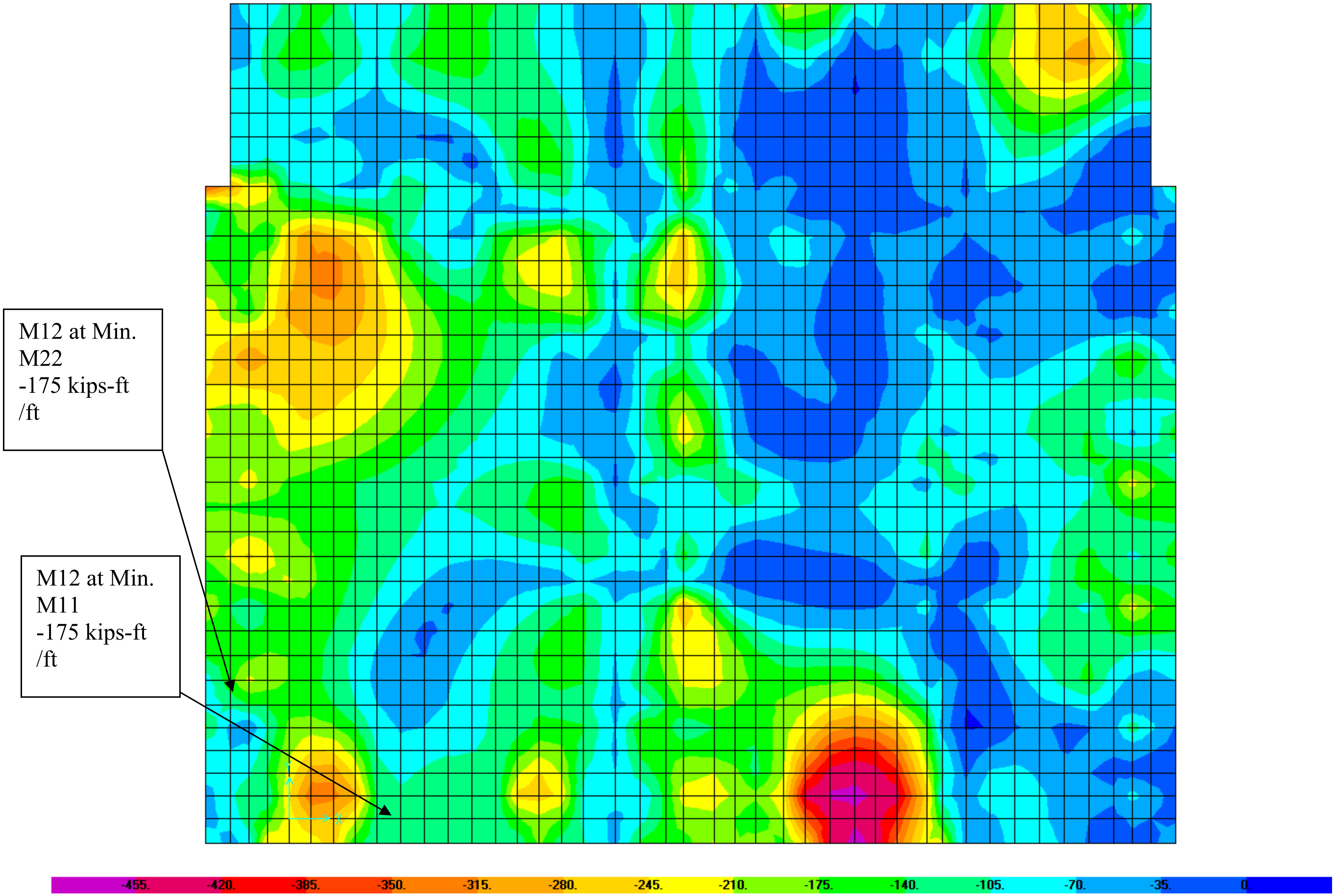

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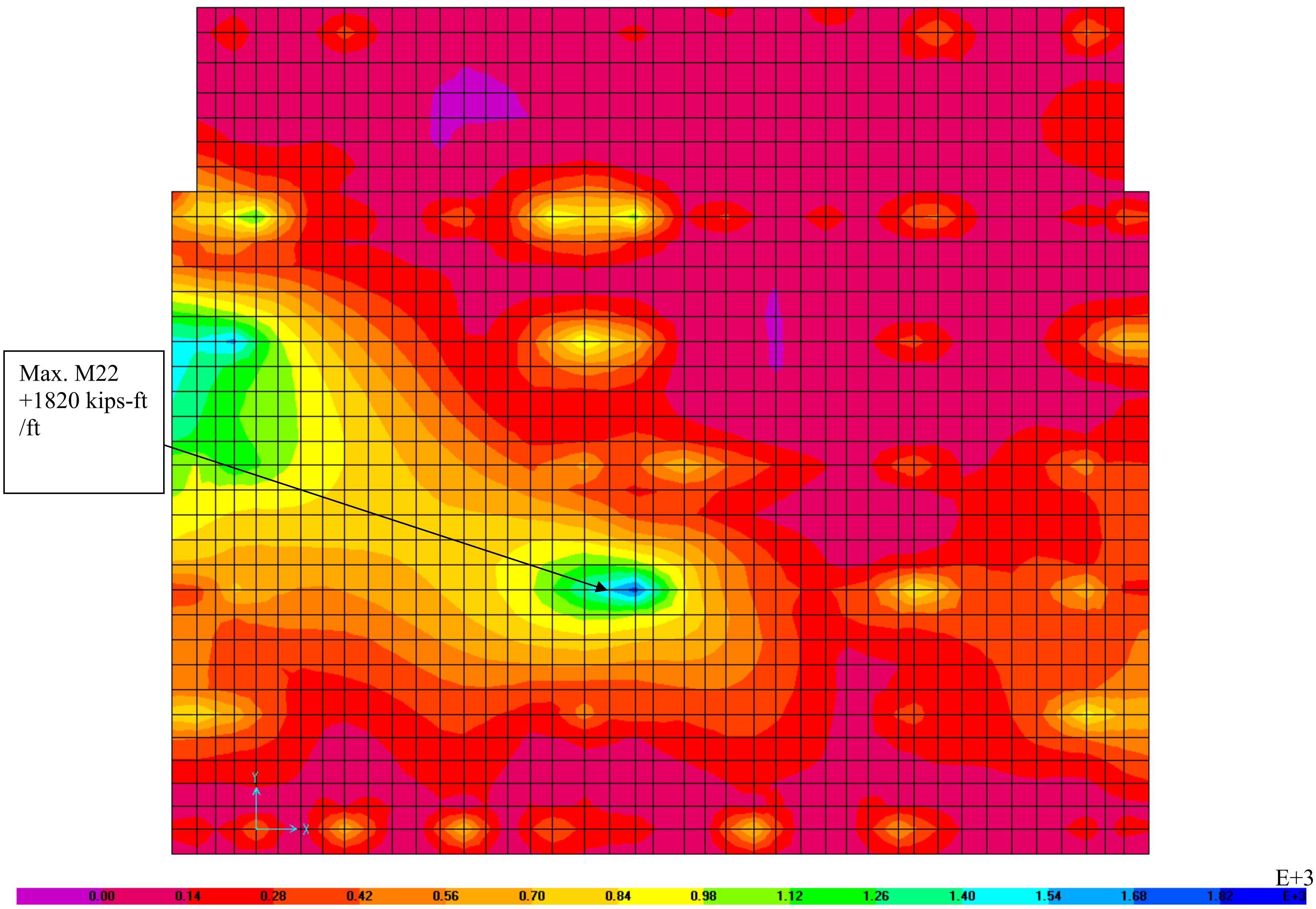

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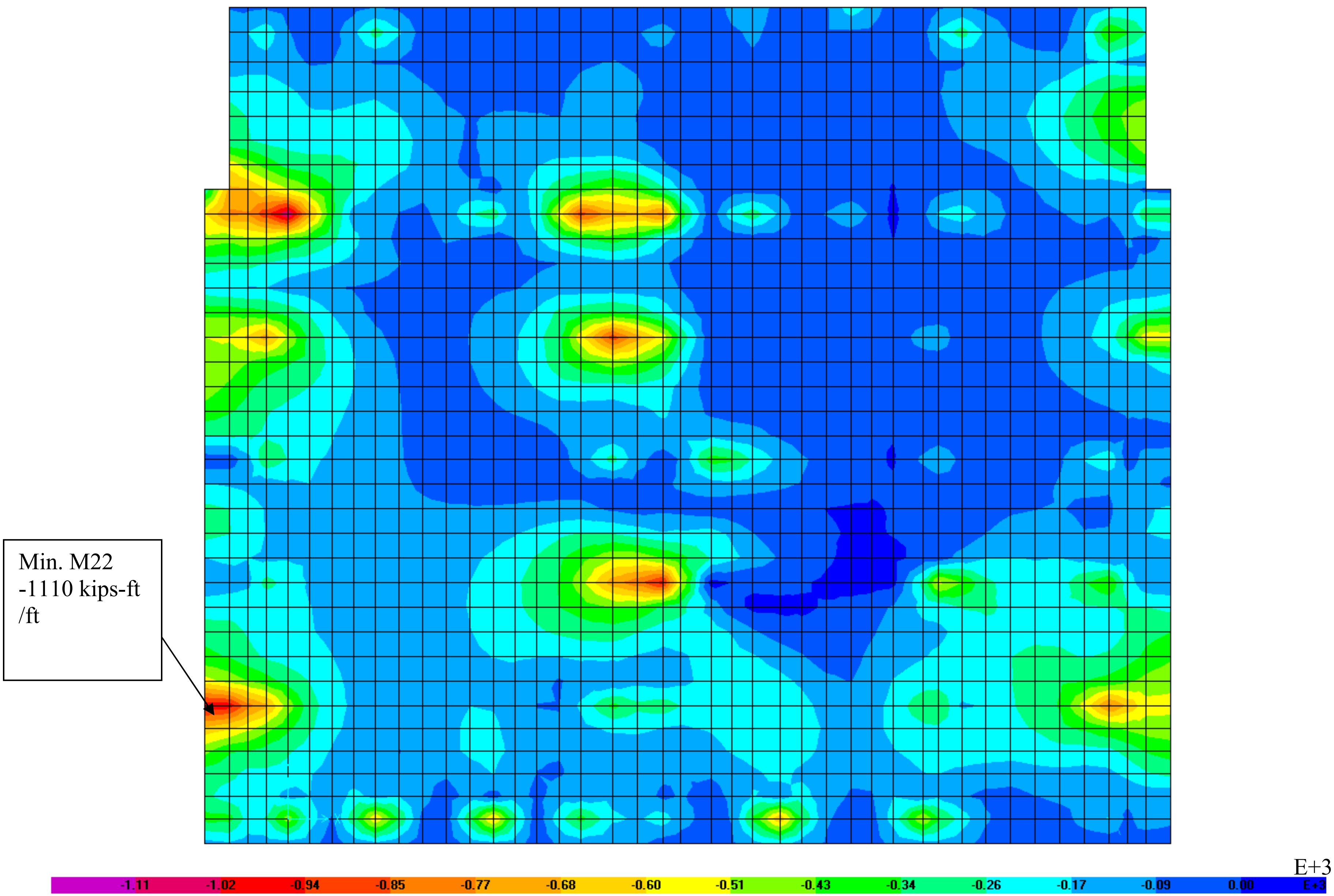

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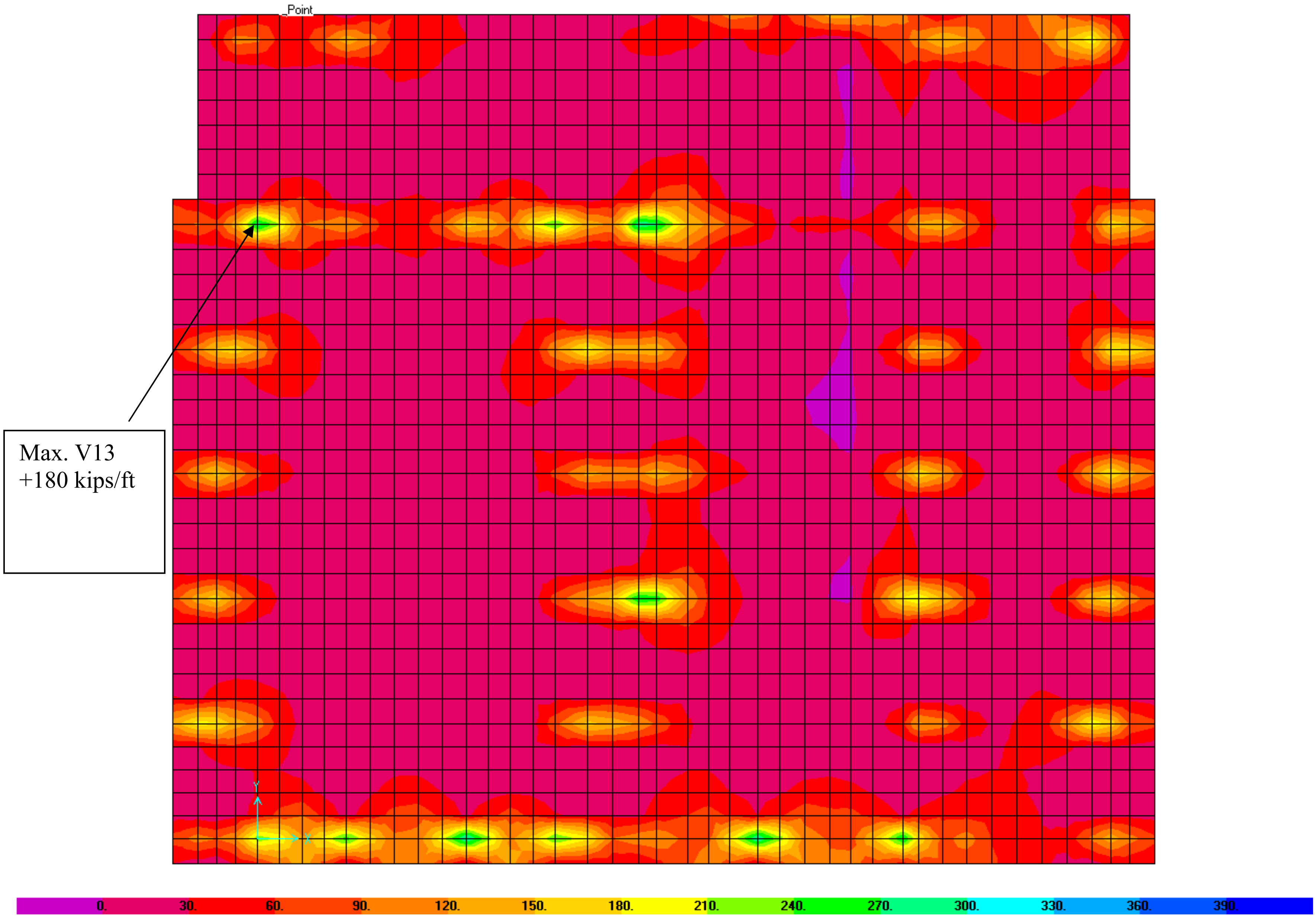

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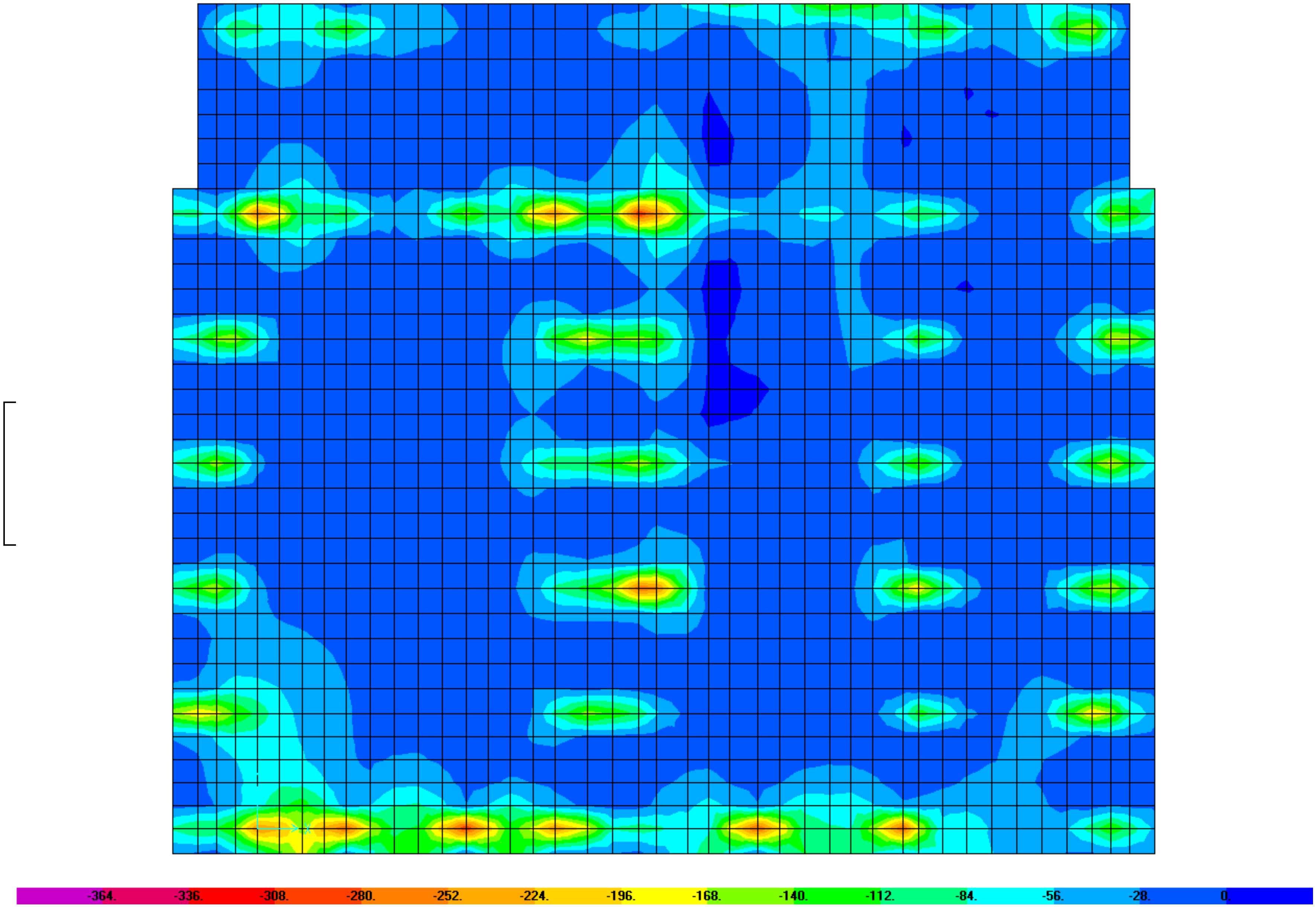

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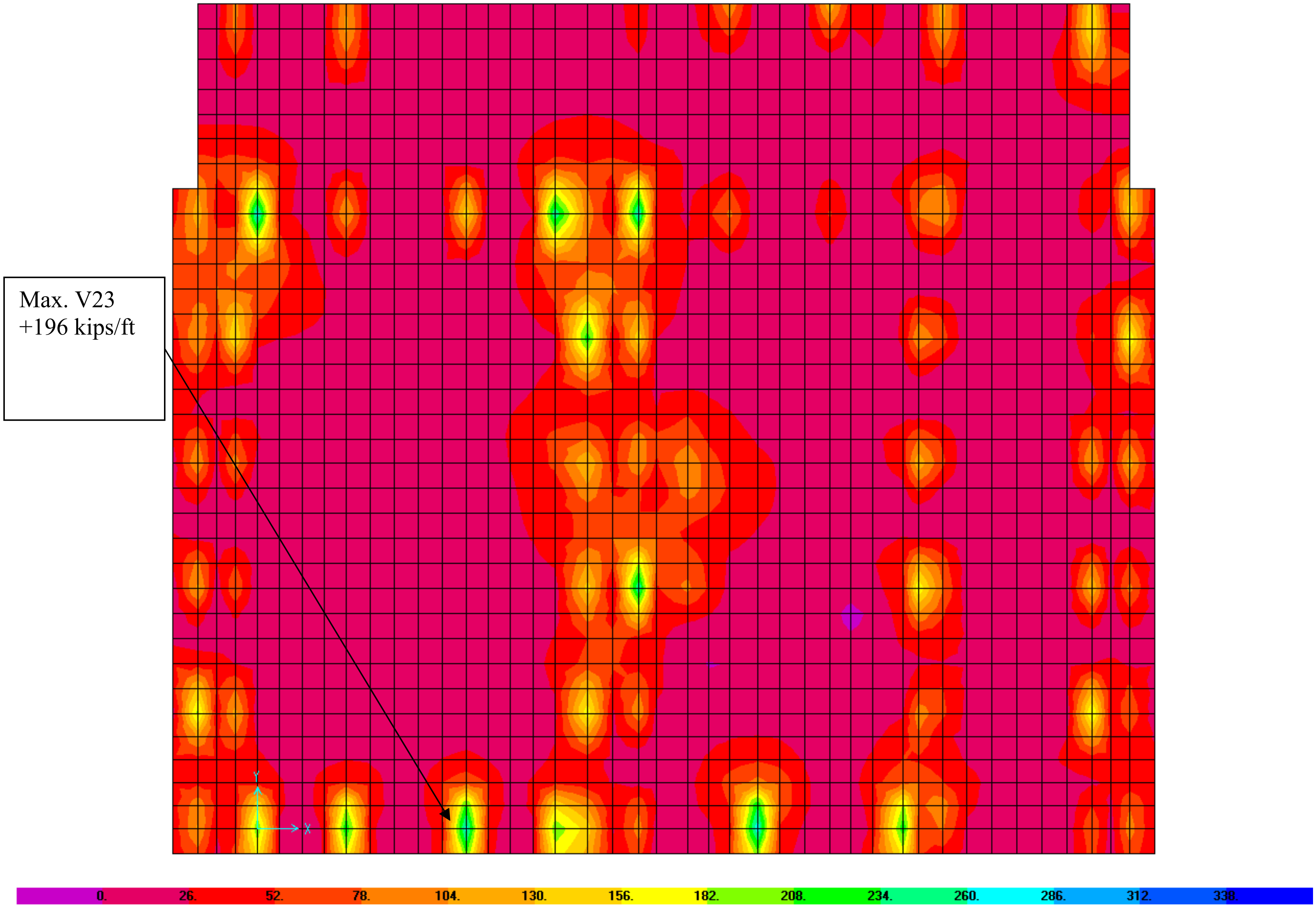

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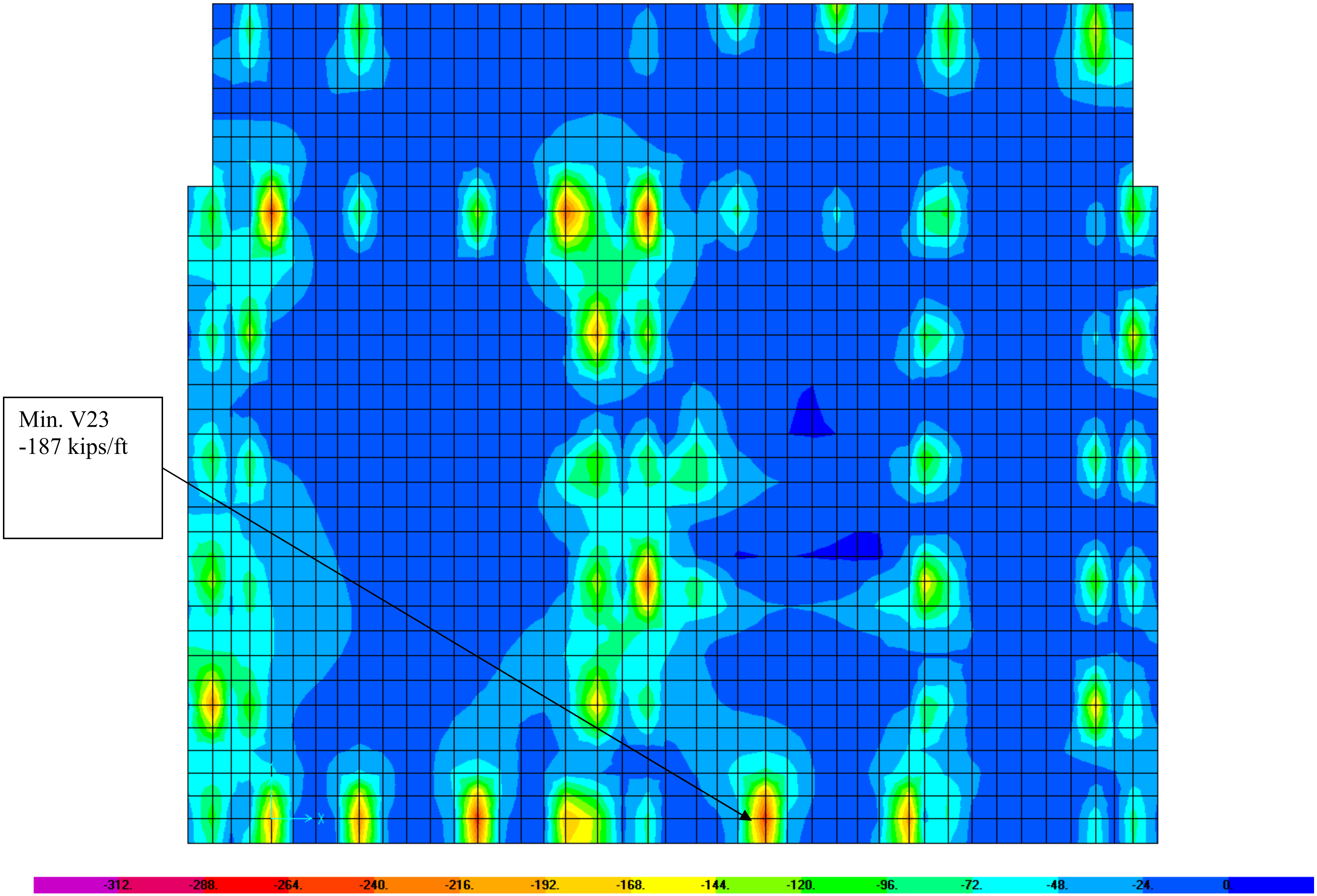

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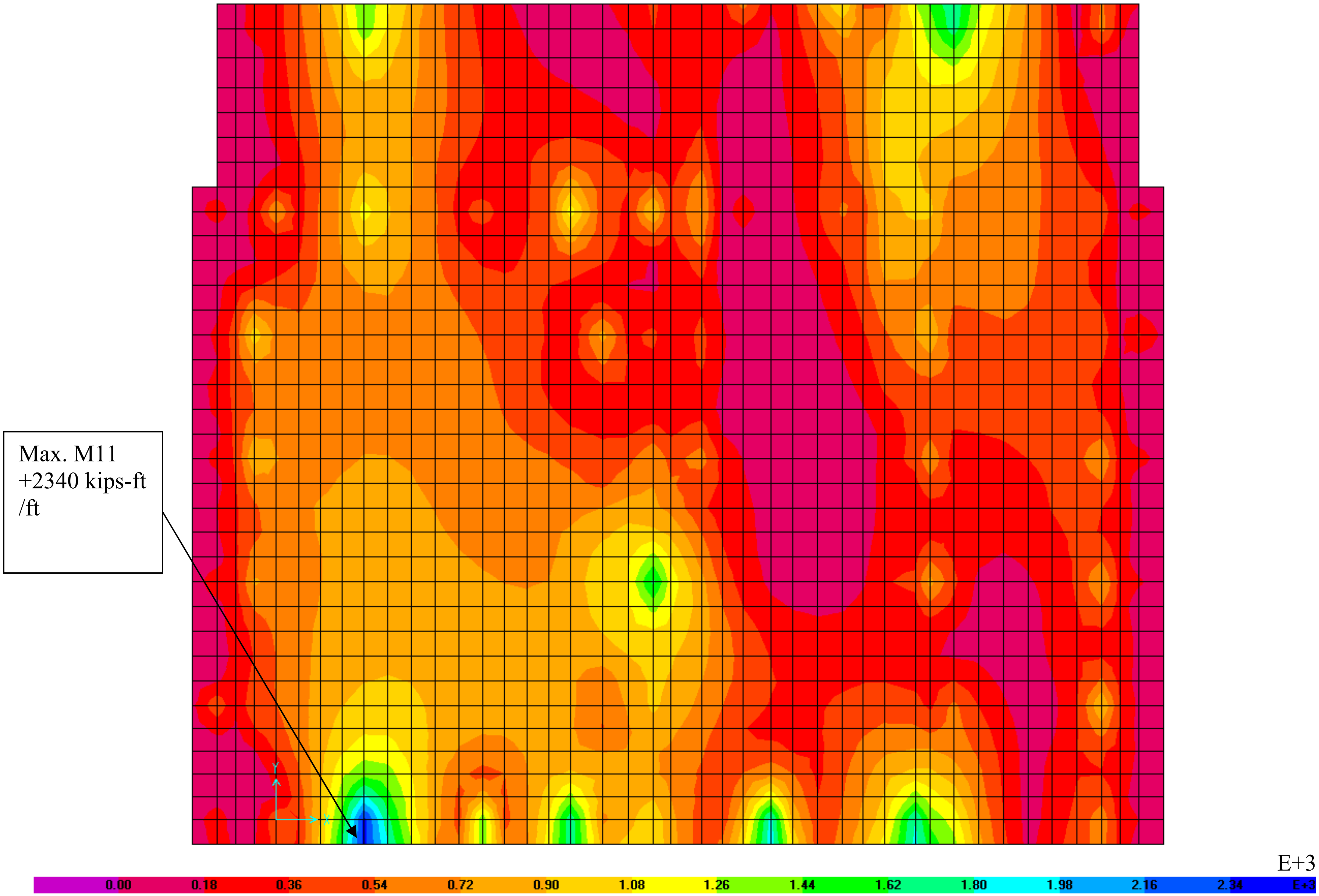

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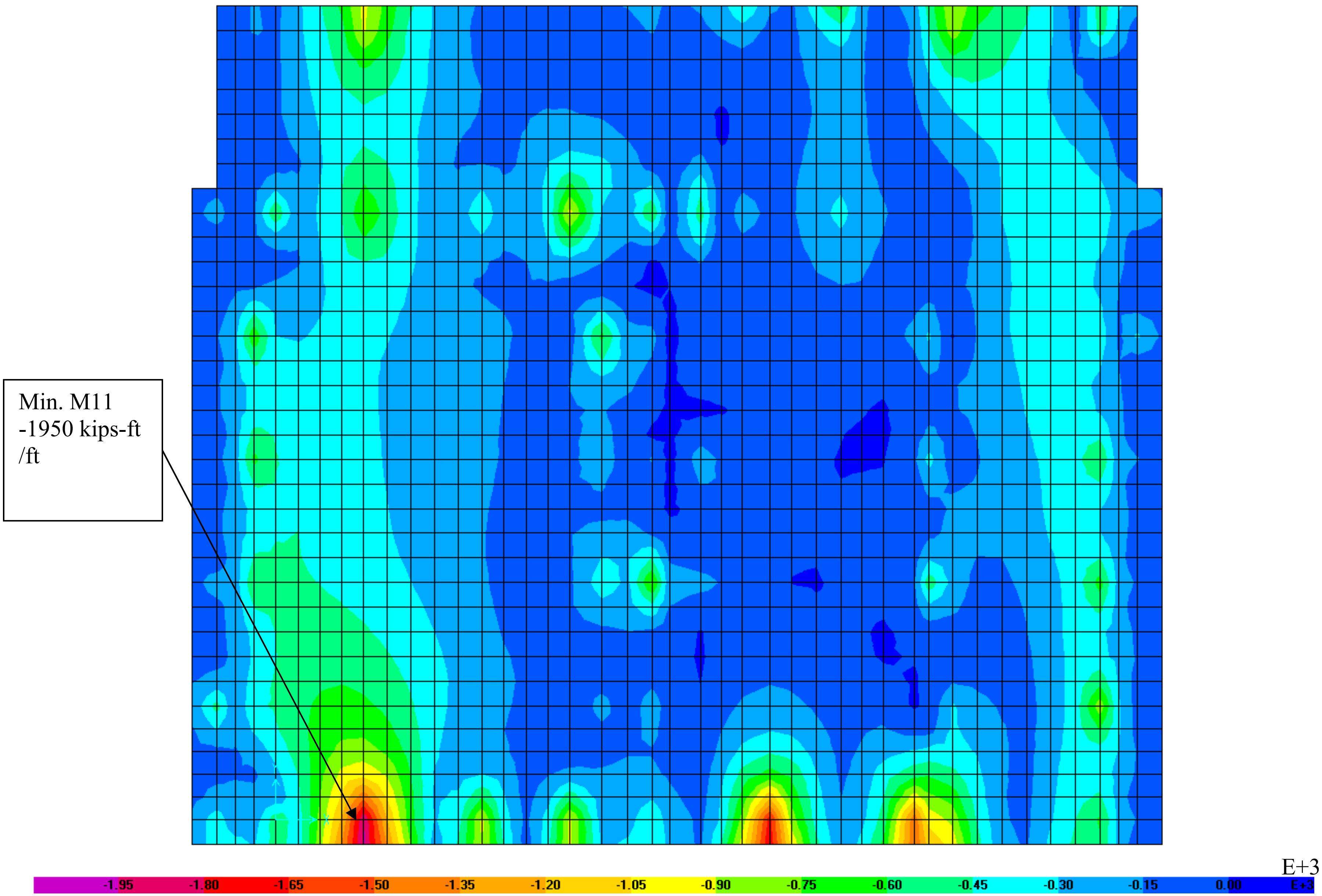

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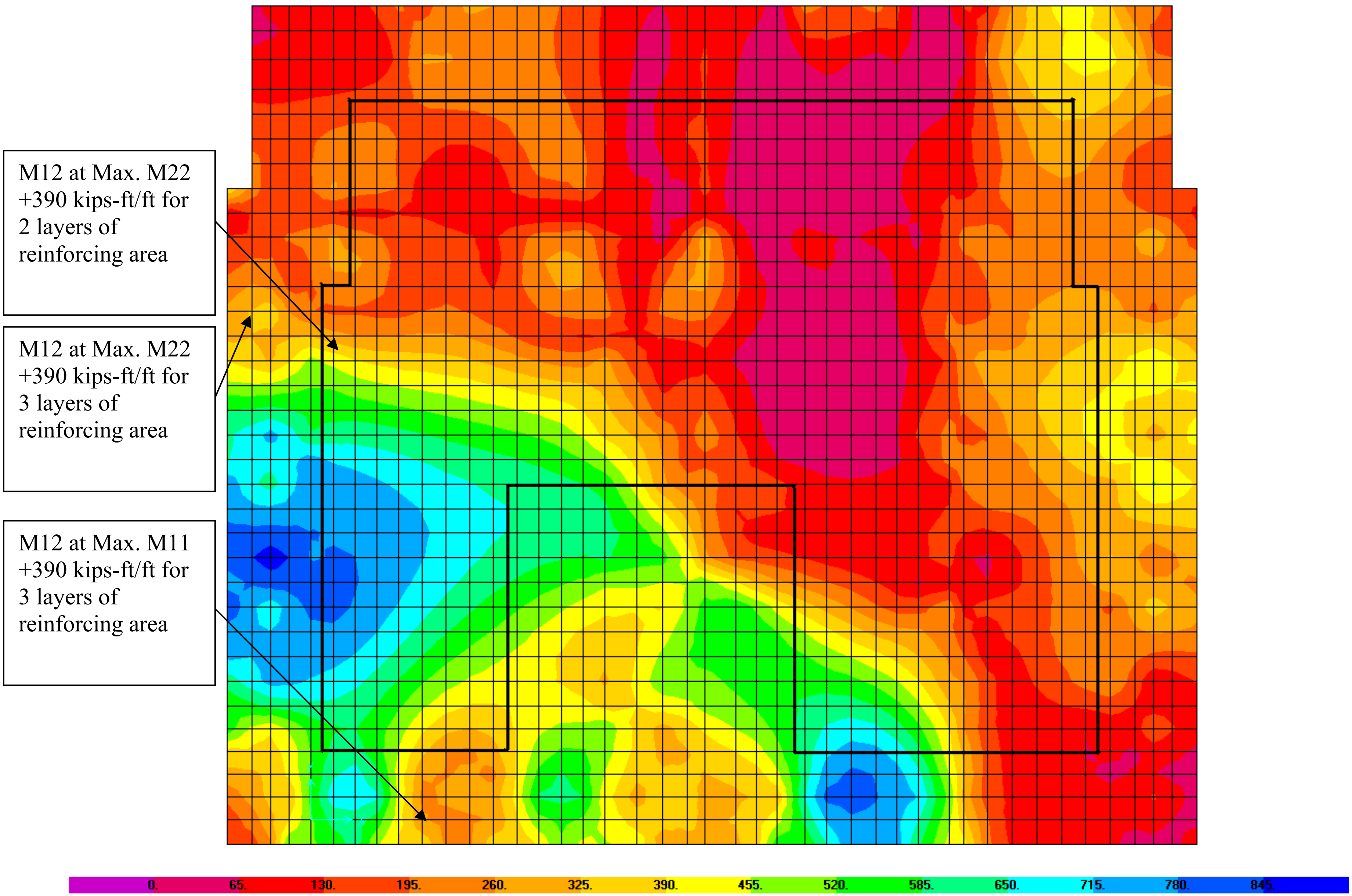

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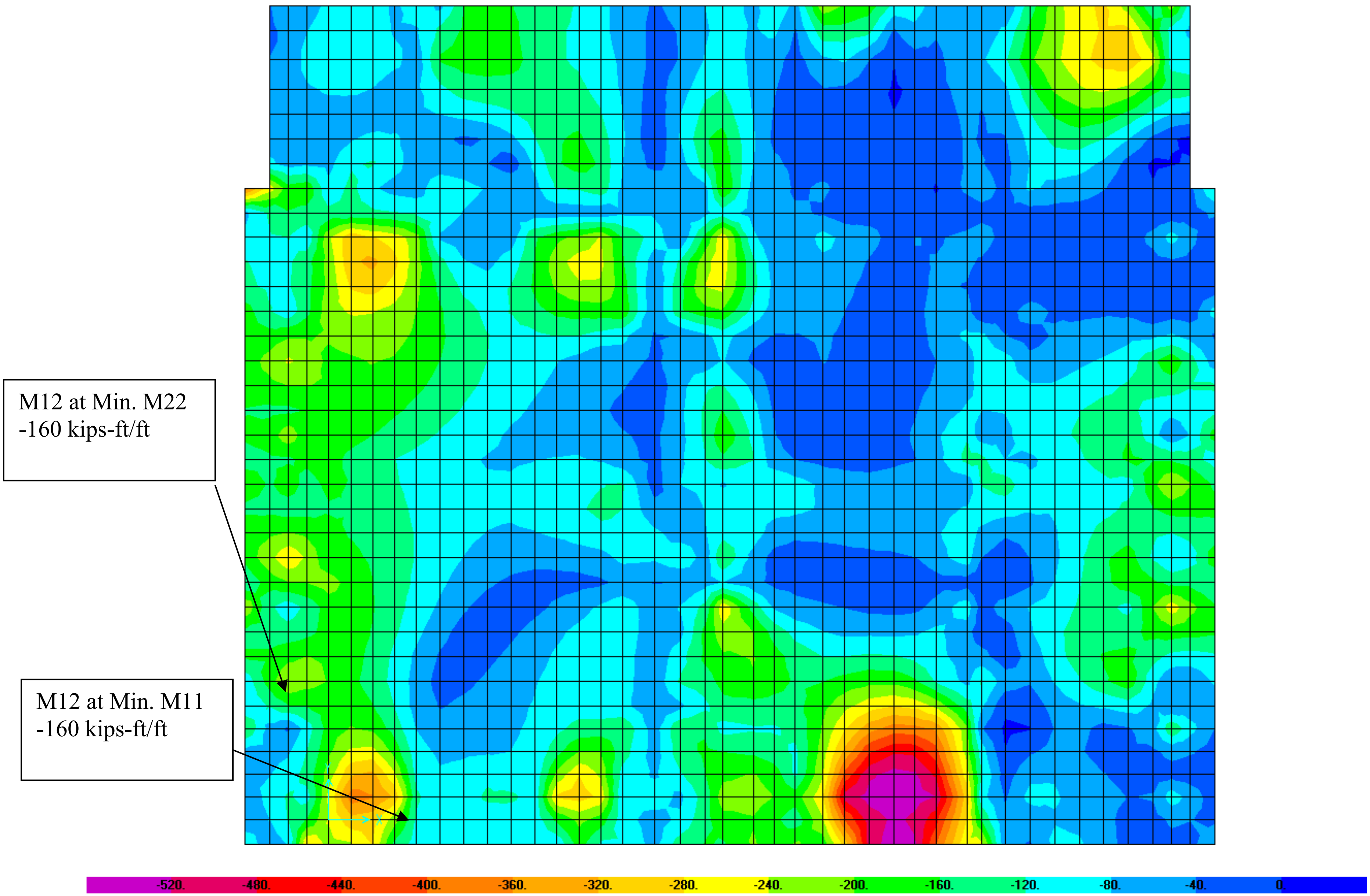

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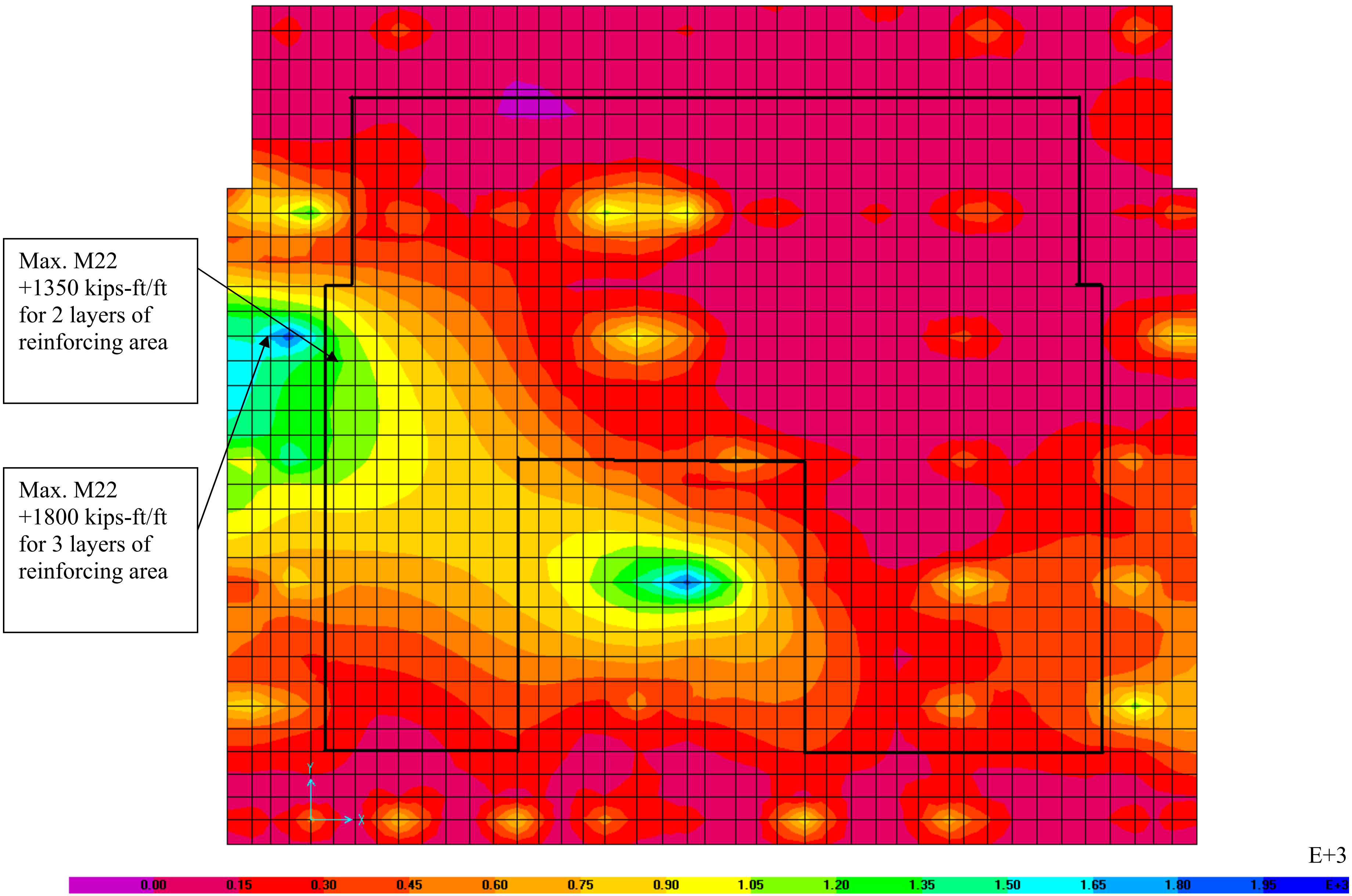

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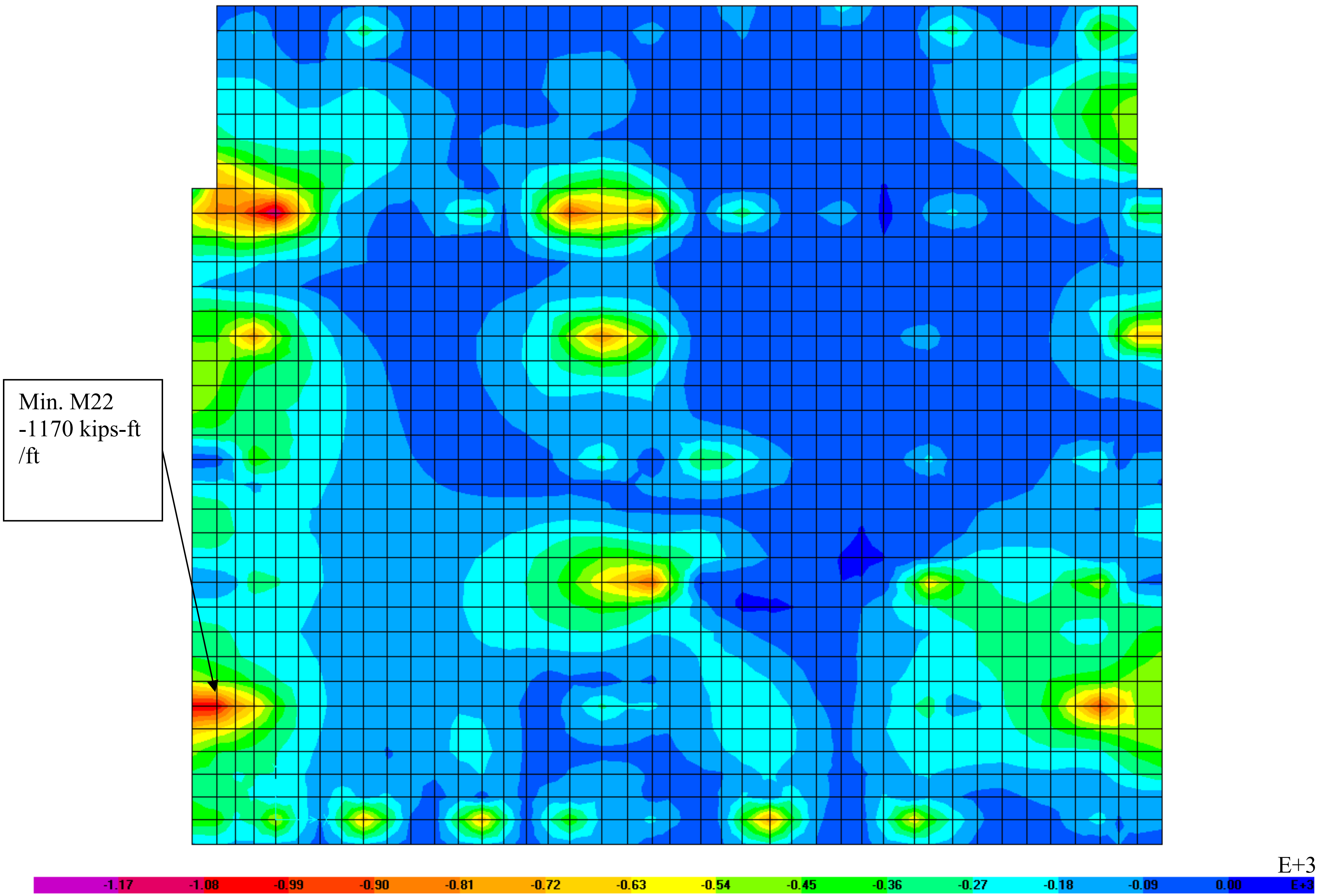

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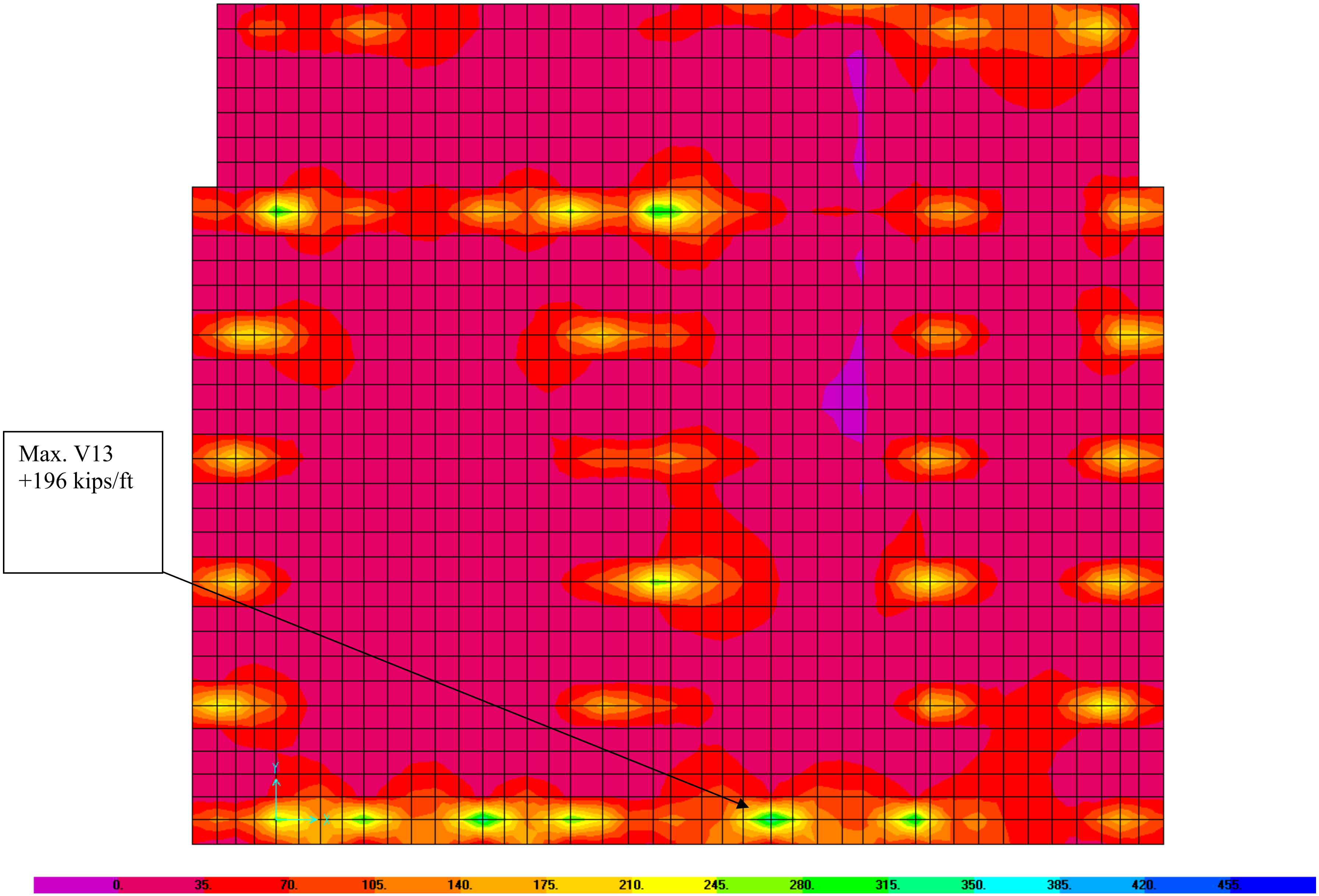

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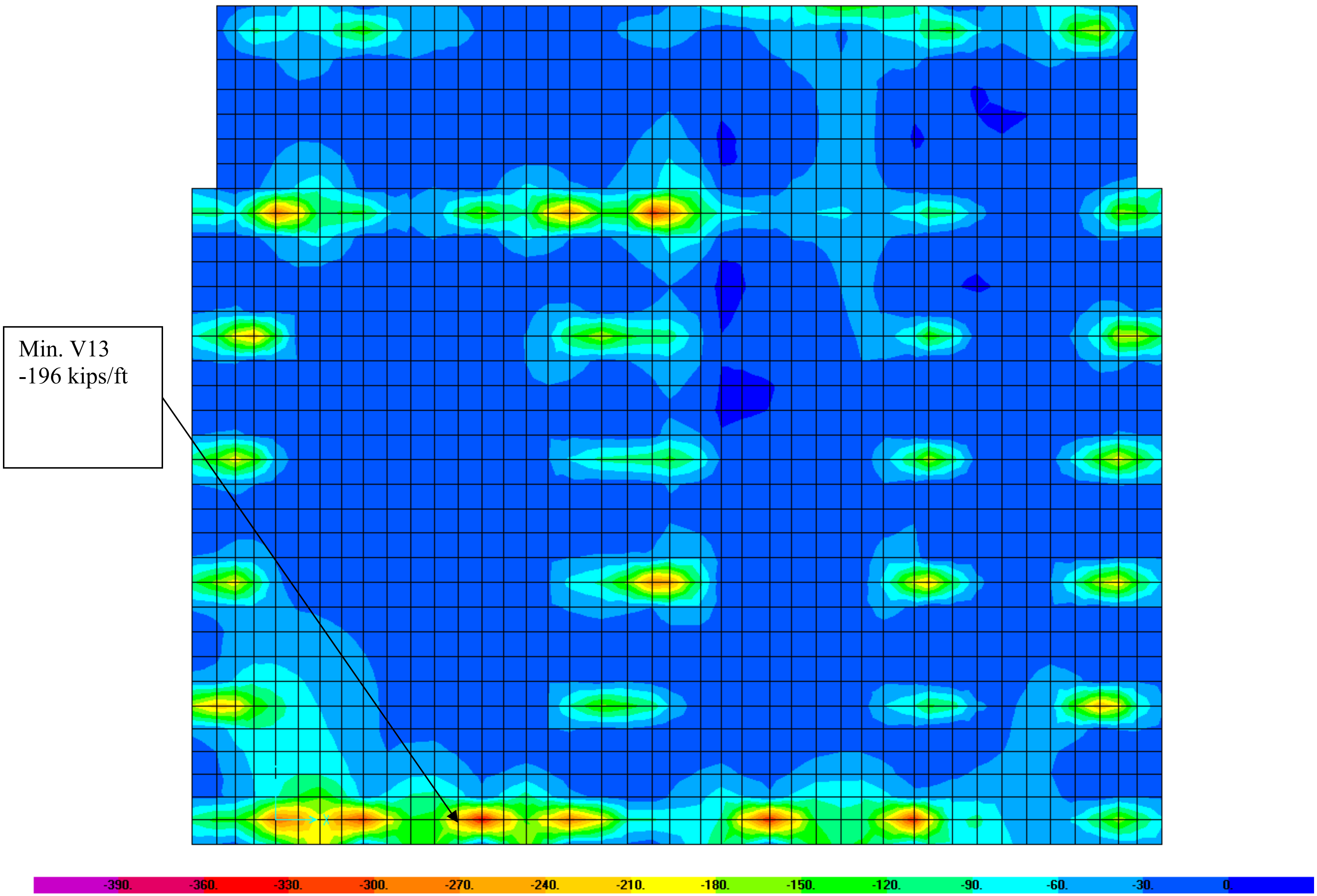

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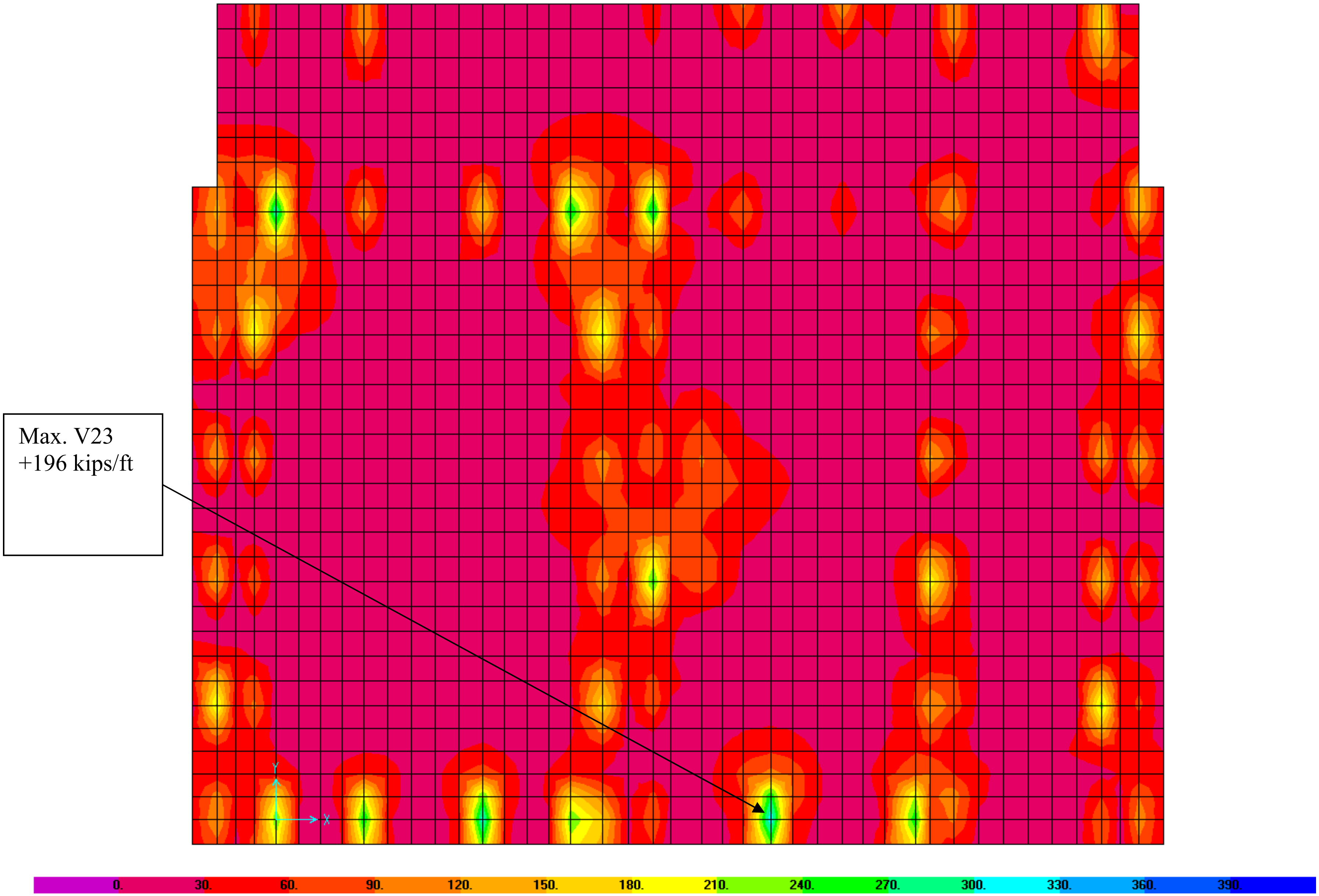

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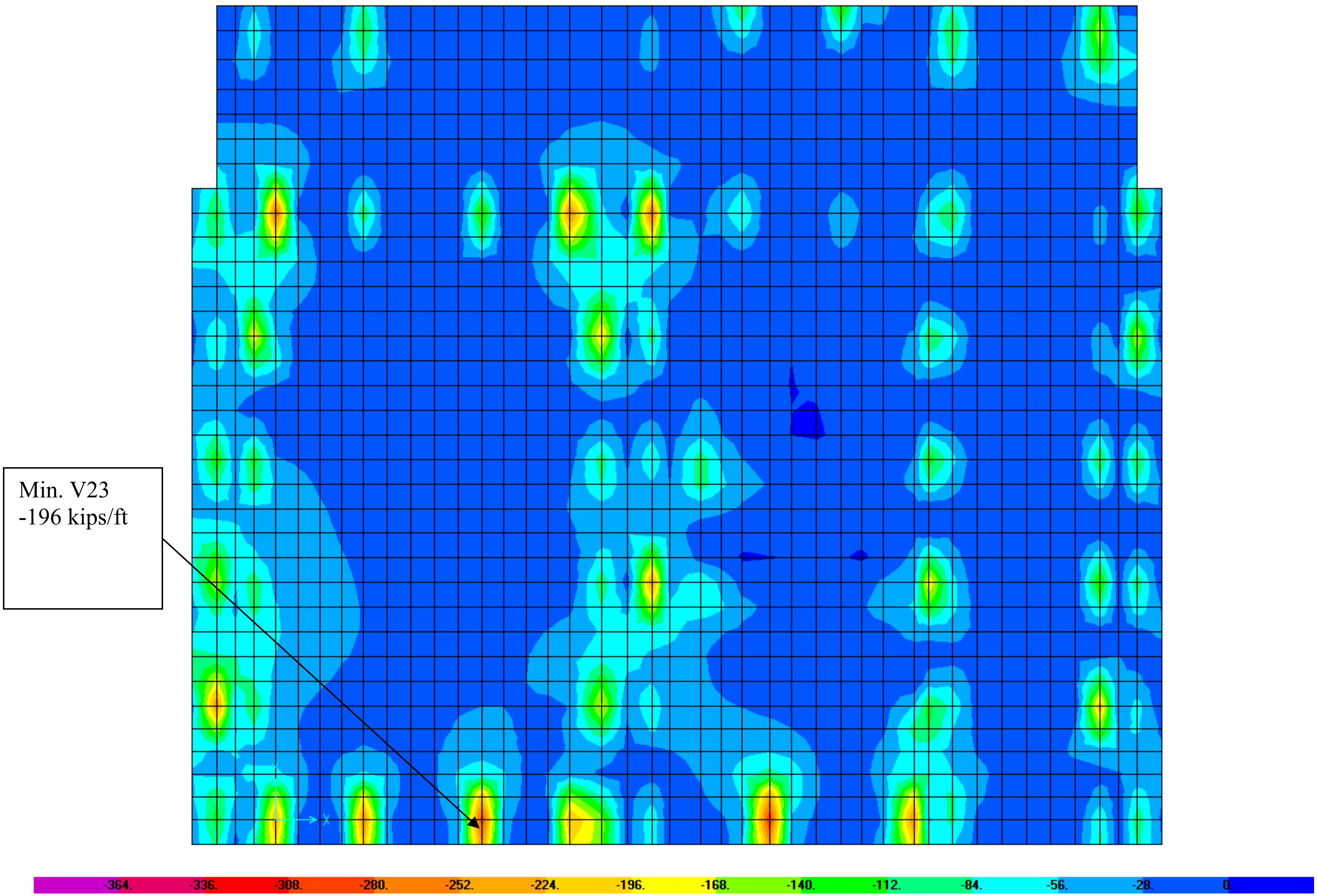

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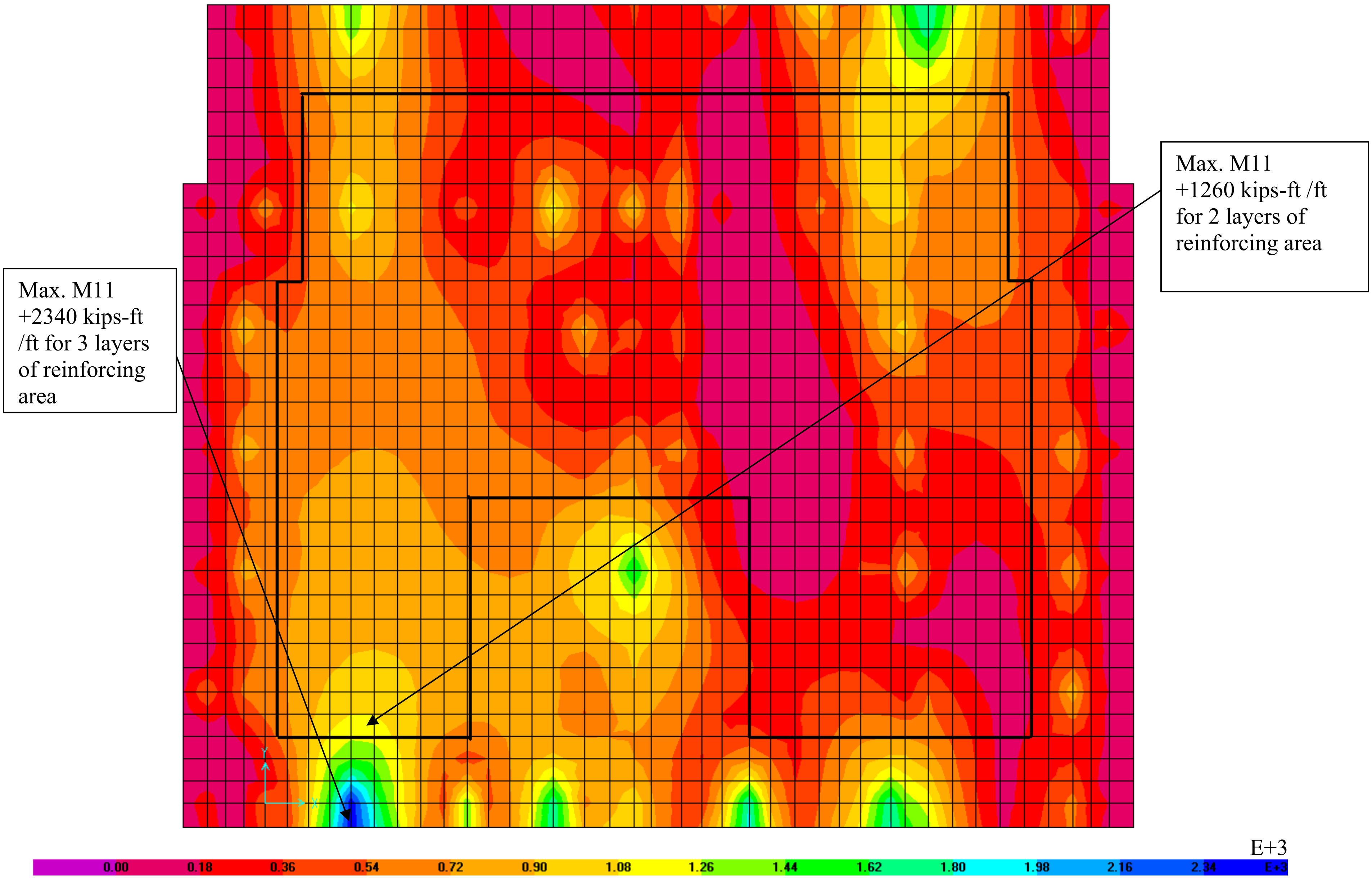
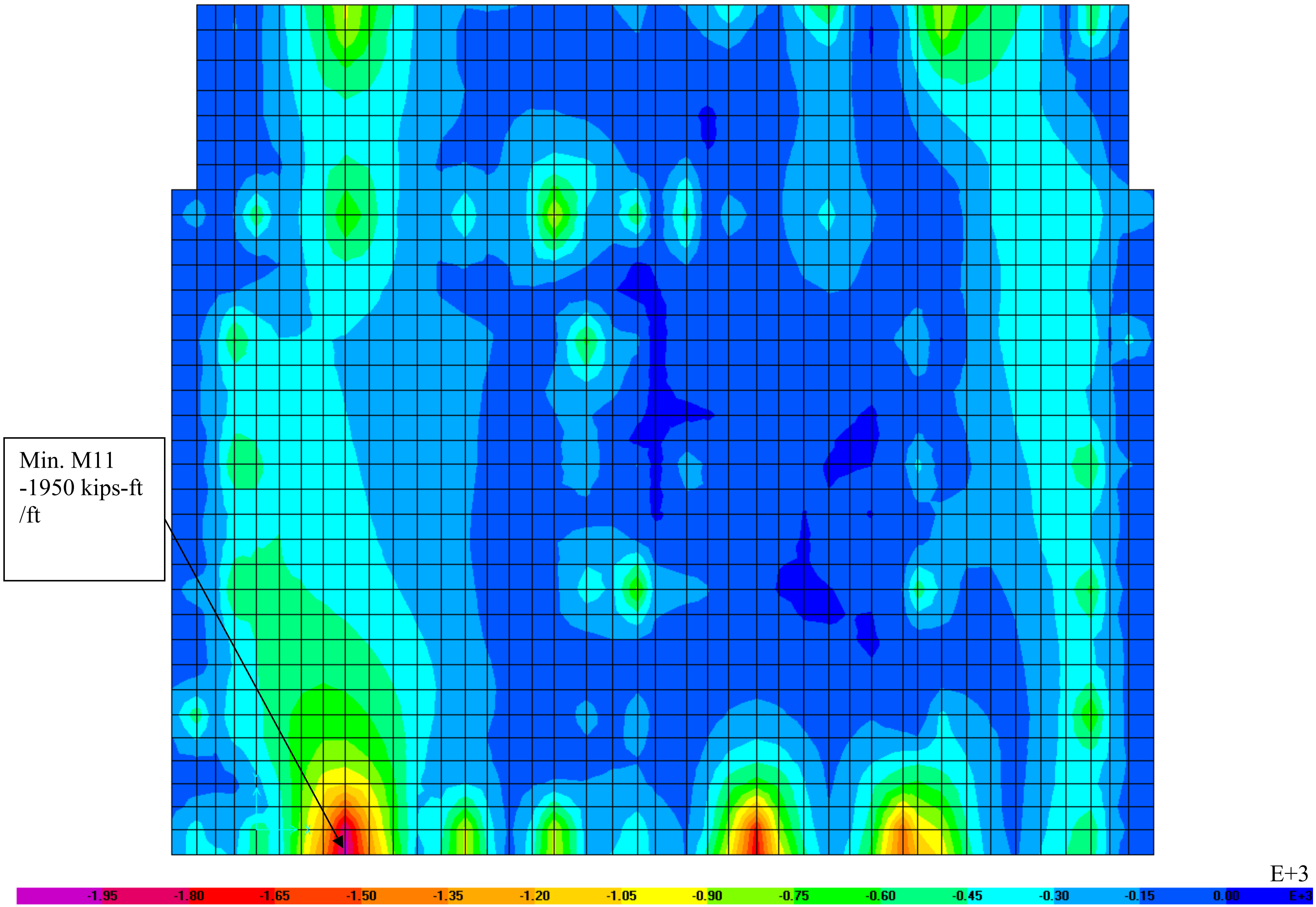
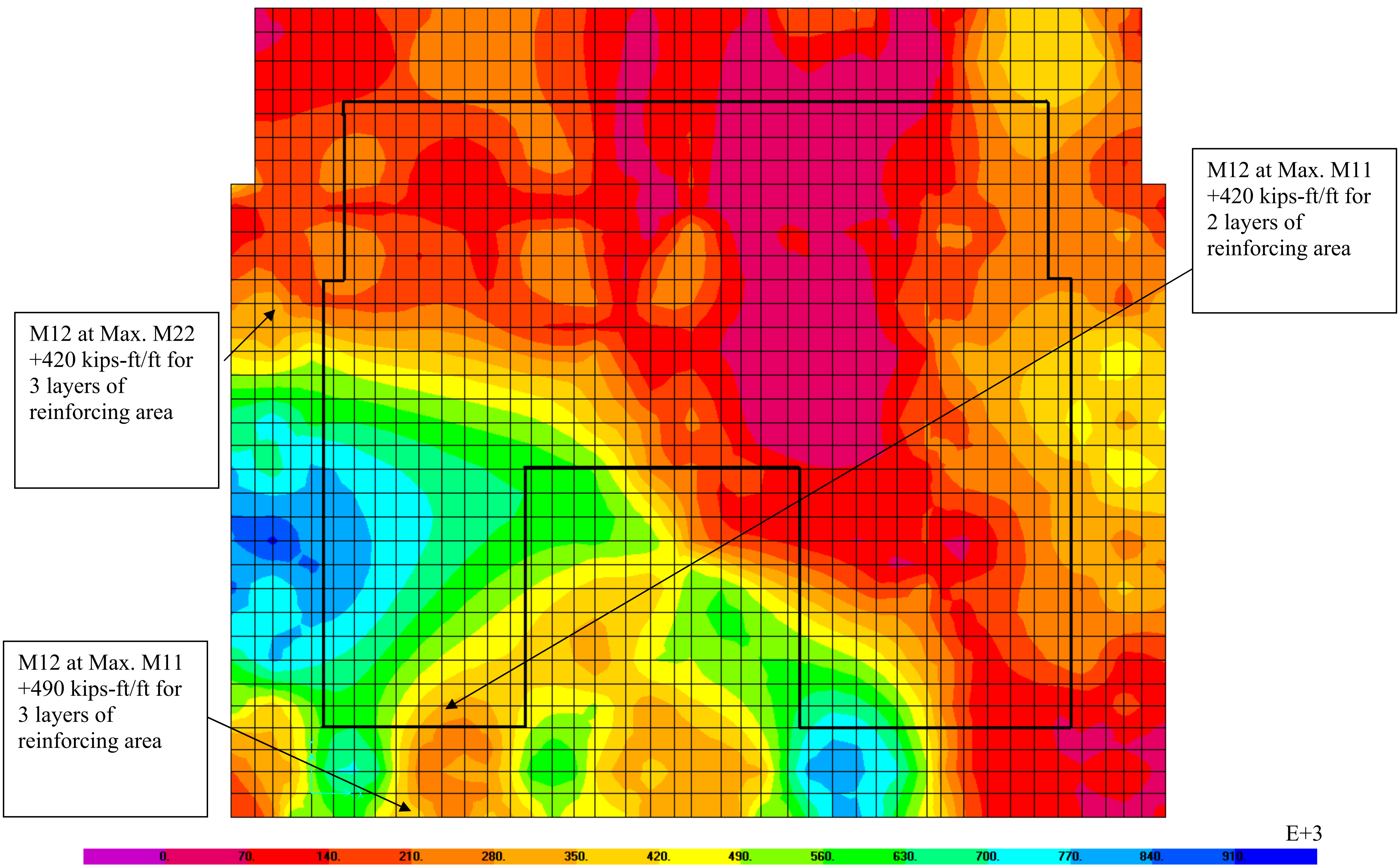


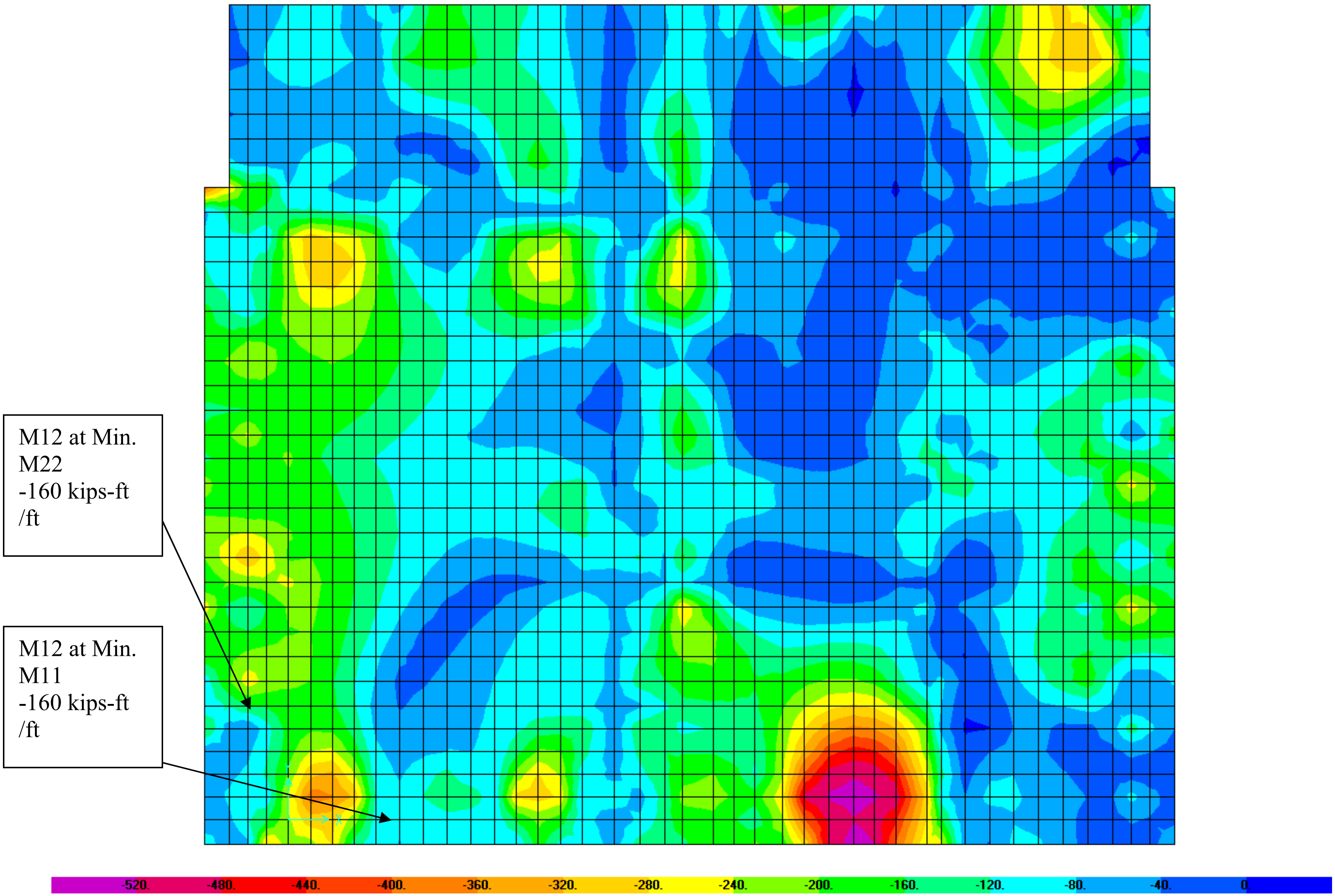
IHF Large Mat CLC7 M11 Envelope Maximum



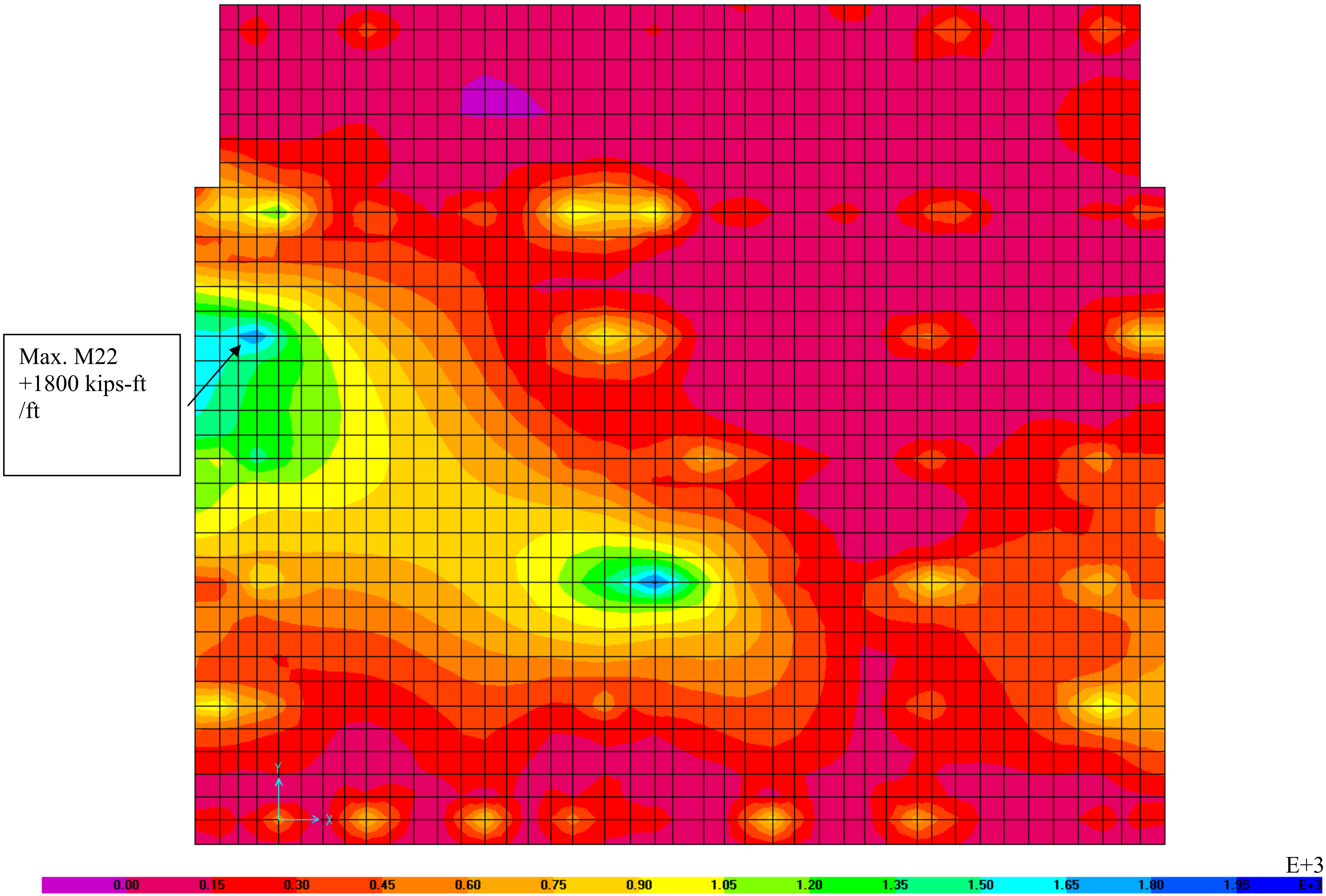
IHF Large Mat CLC7 M11 Envelope Minimum



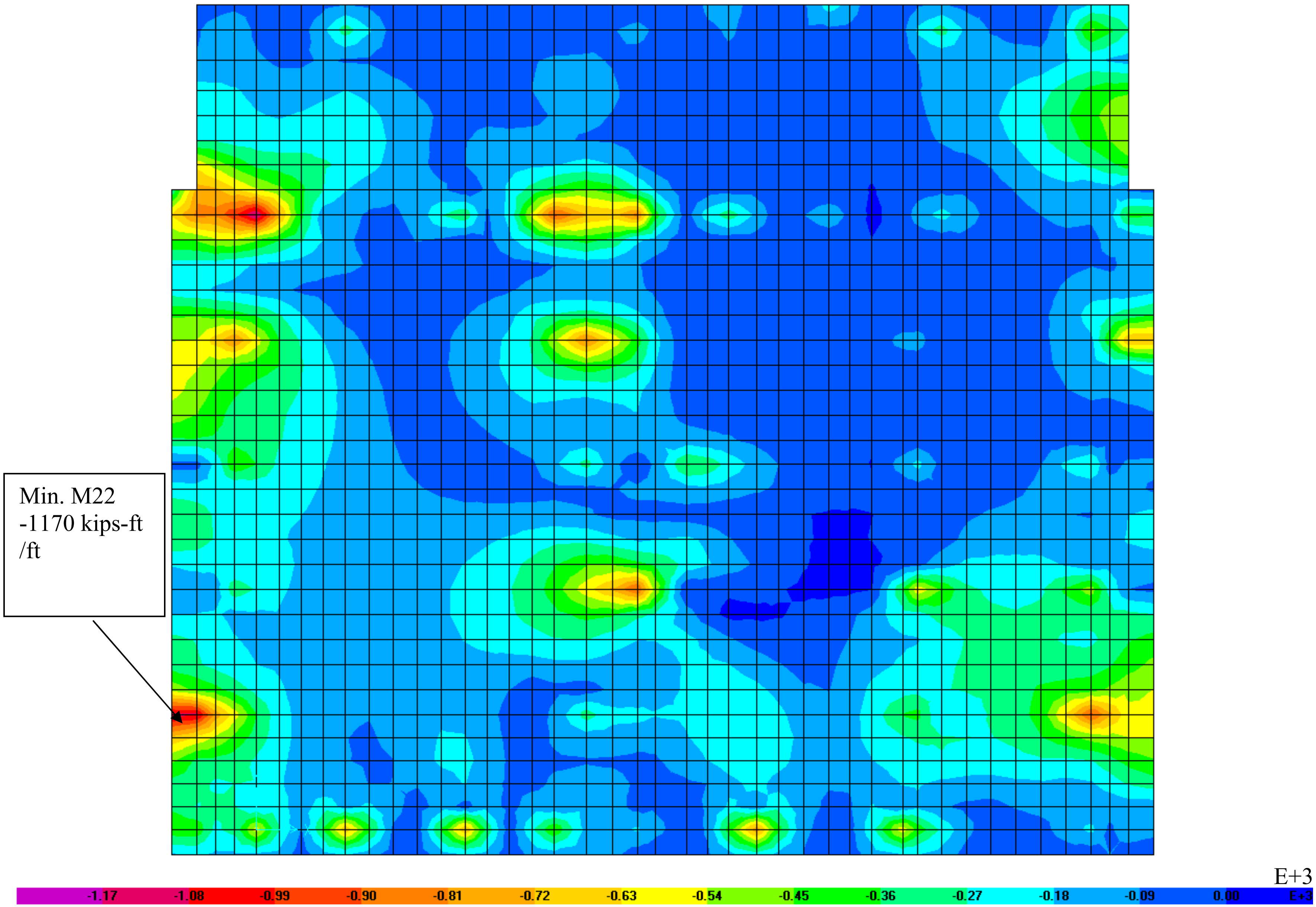
IHF Large Mat CLC7 M12 Envelope Maximum



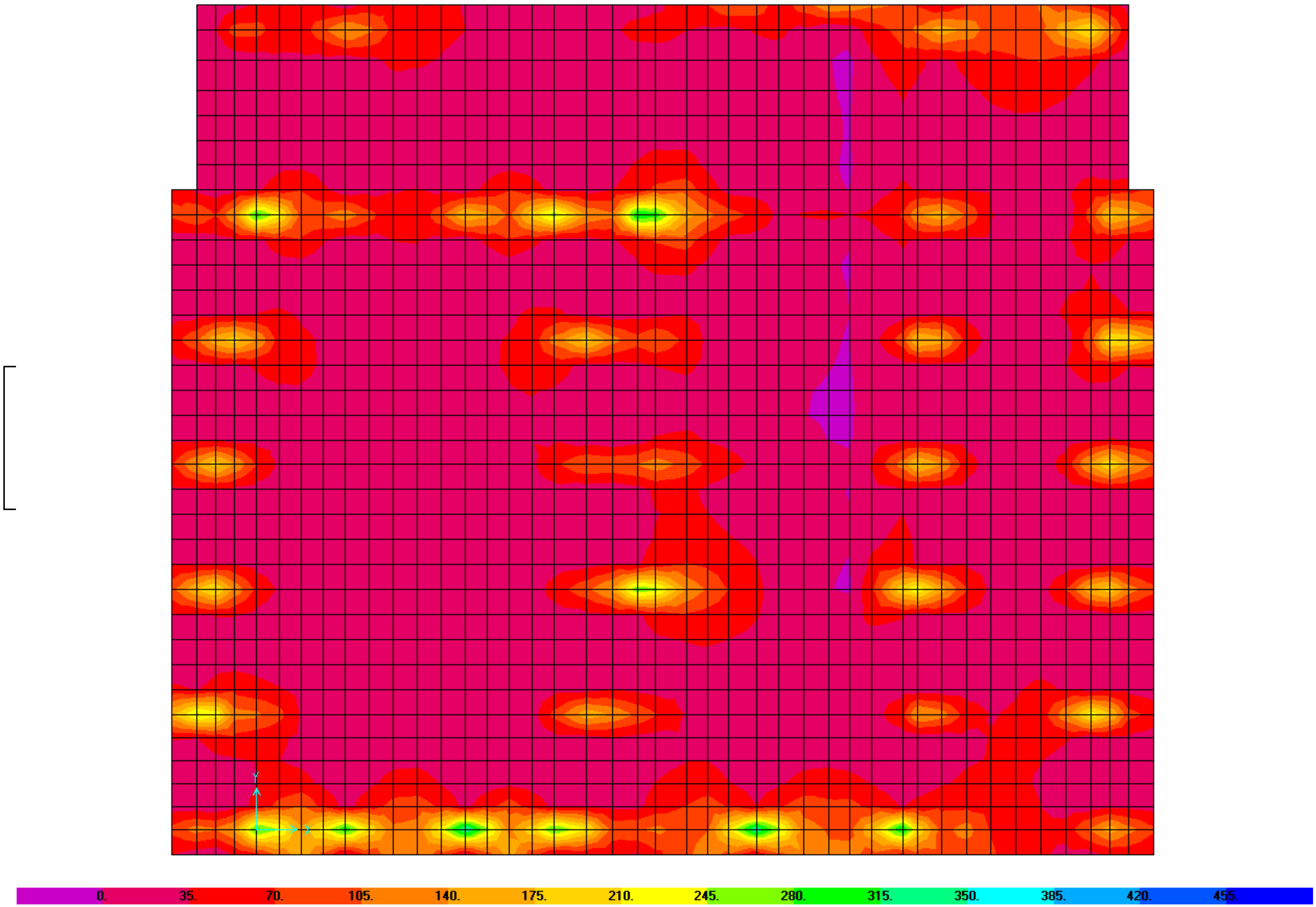
IHF Large Mat CLC7 M12 Envelope Minimum



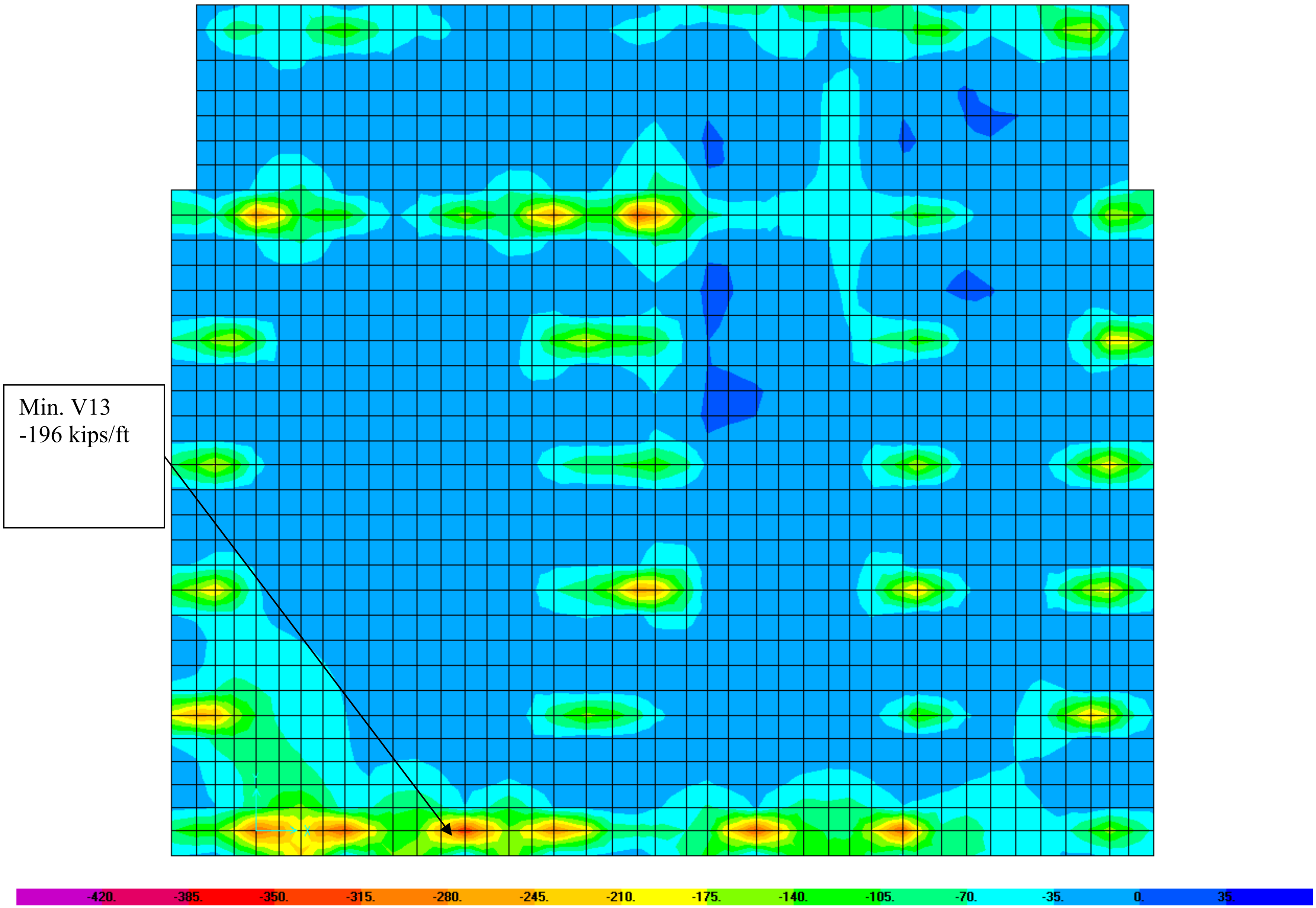
IHF Large Mat CLC7 M22 Envelope Maximum



IHF Large Mat CLC7 M22 Envelope Minimum



IHF Large Mat CLC7 V13 Envelope Maximum



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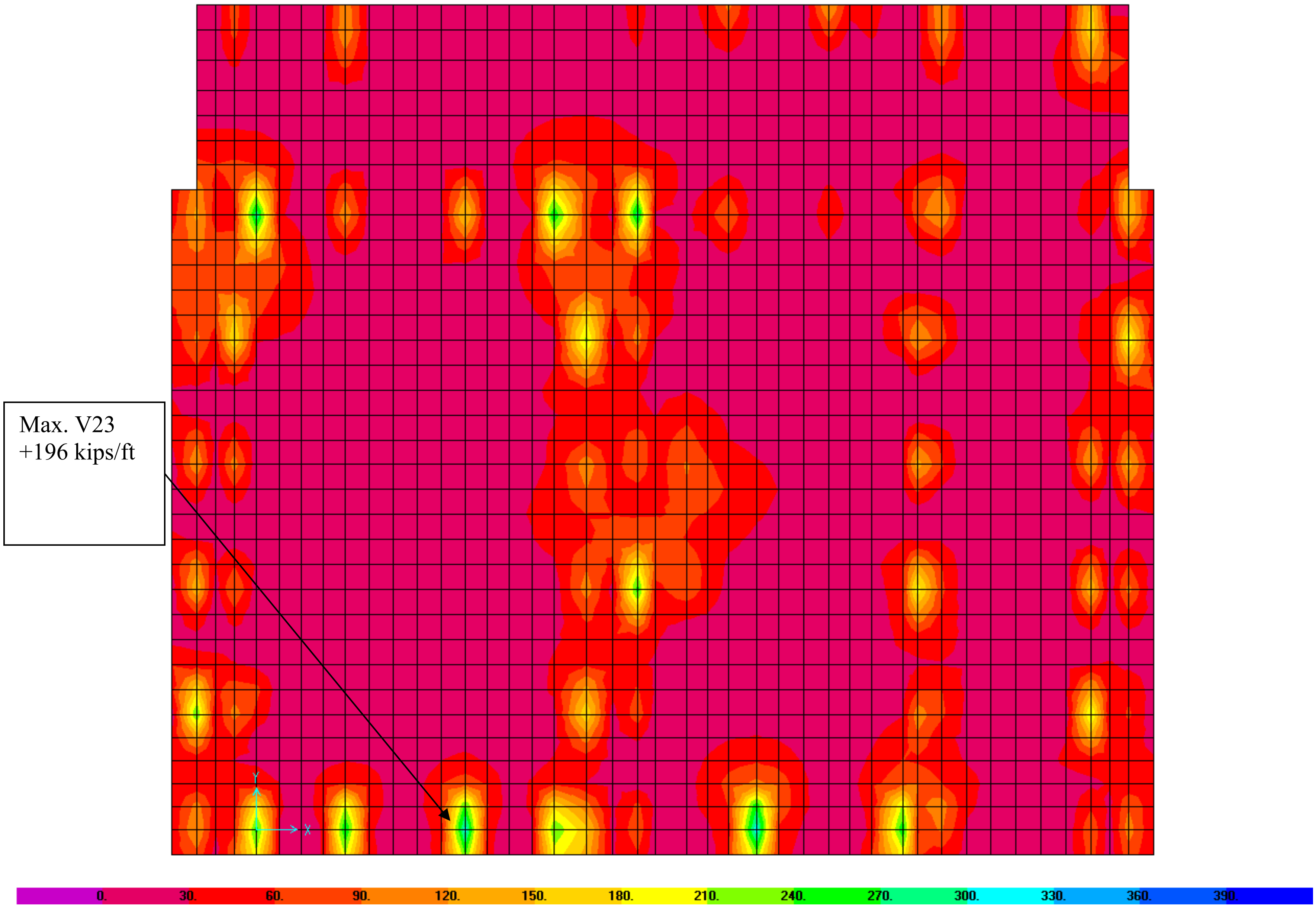

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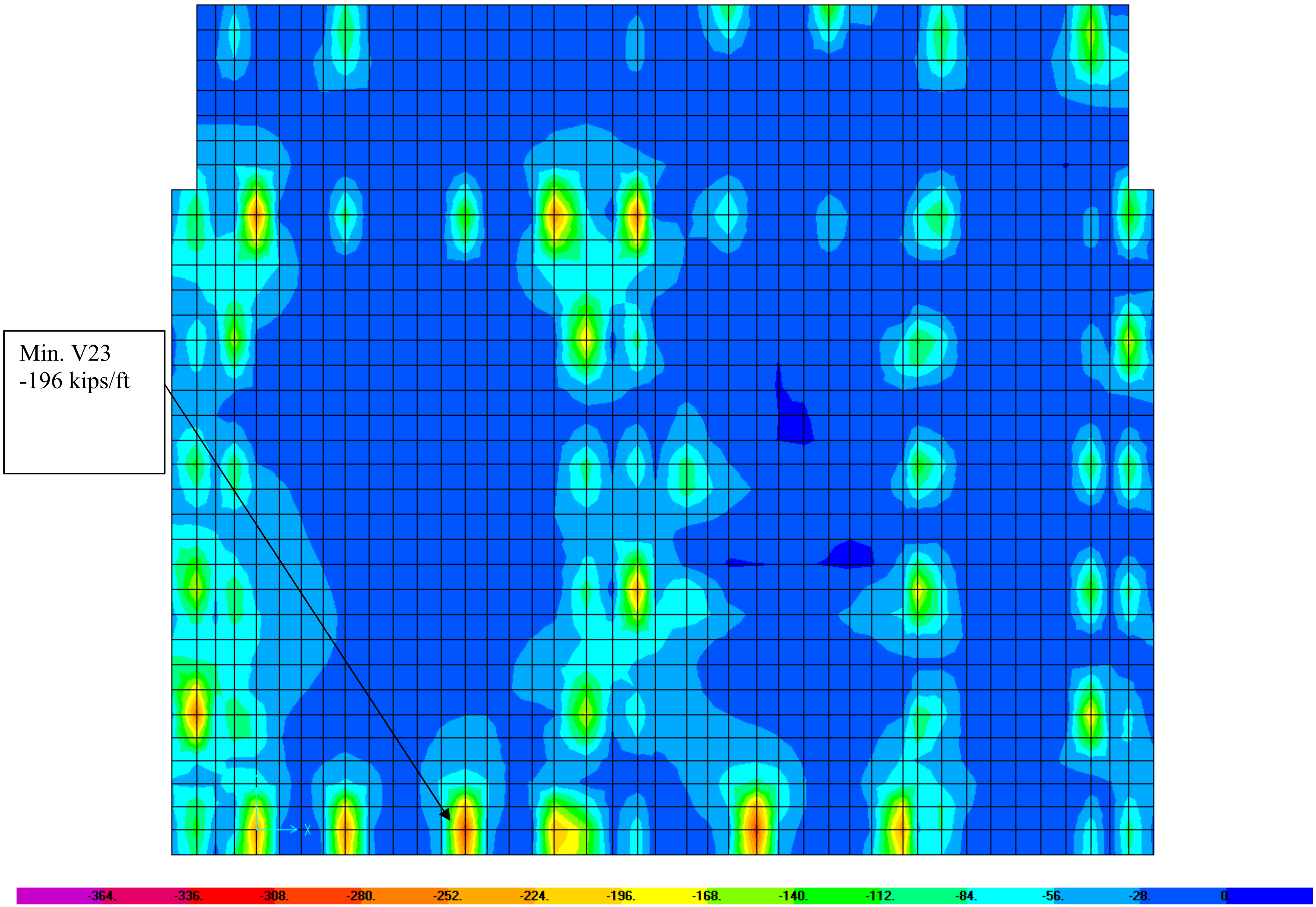


IHF Large Mat CLC7 V23 Envelope Minimum

Attachment E

SAP2000 Output - Moment and Shear Contours For Small Mat

Page

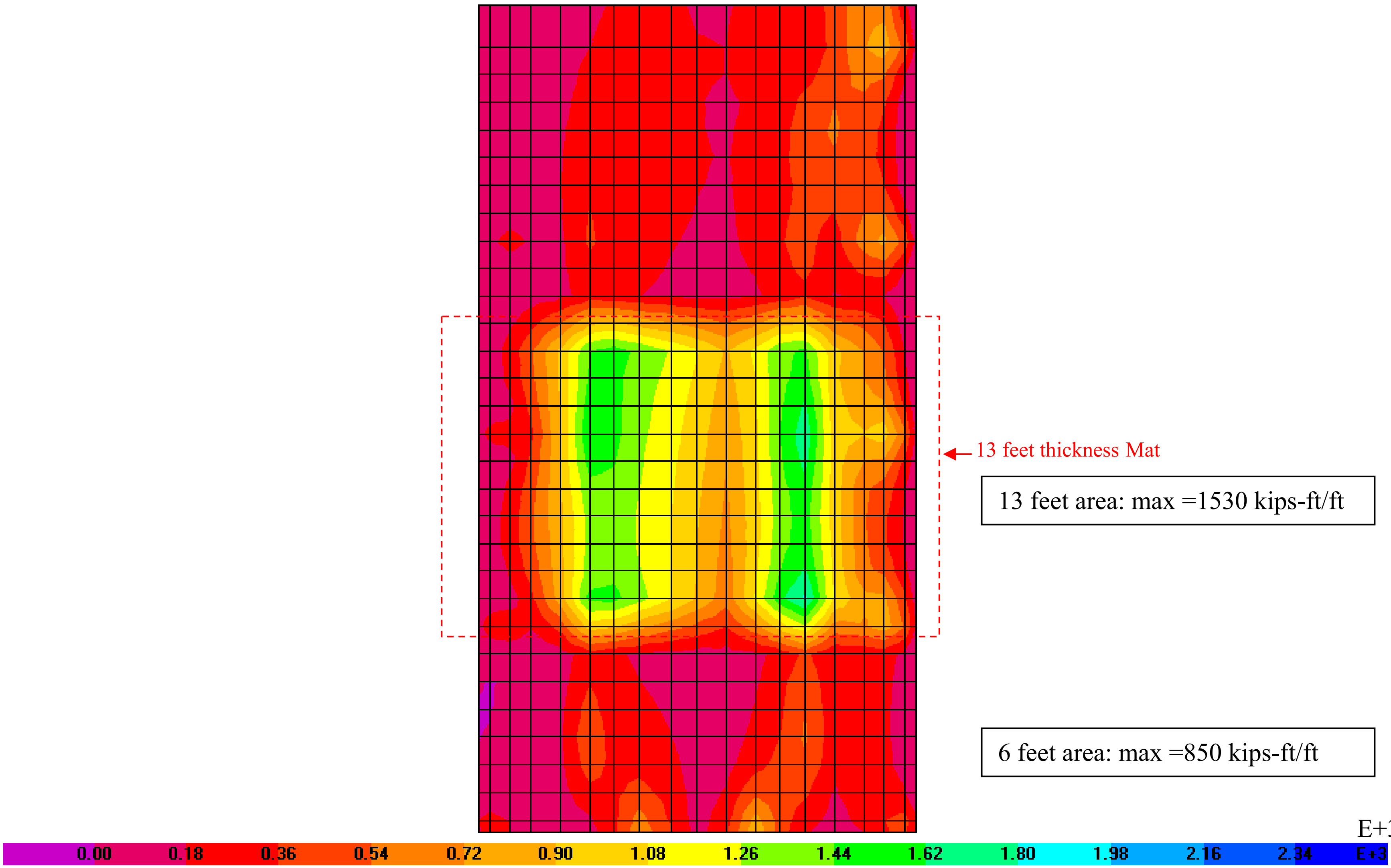


Figure E-1: M11 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 1

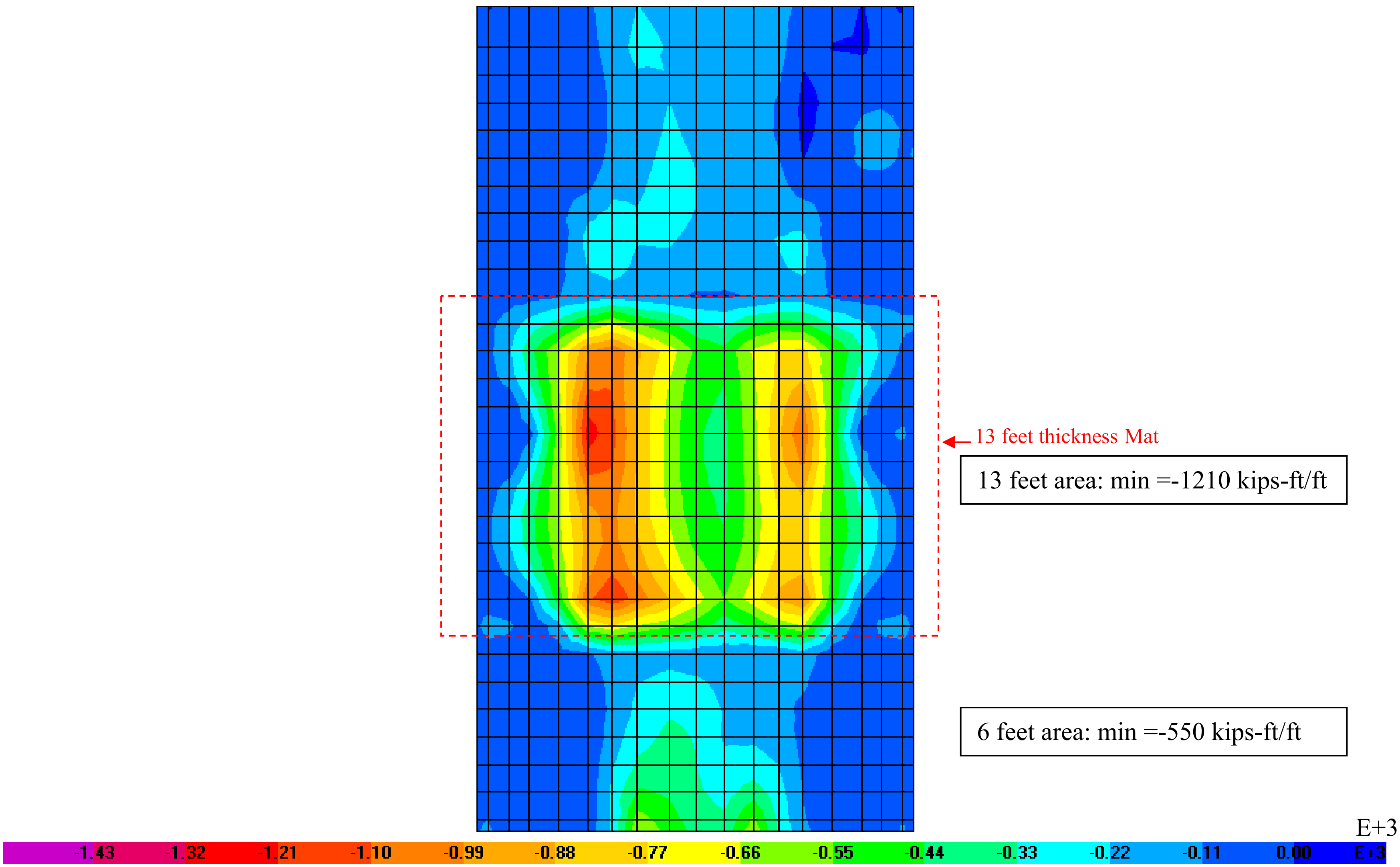


Figure E-2: M11 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 1

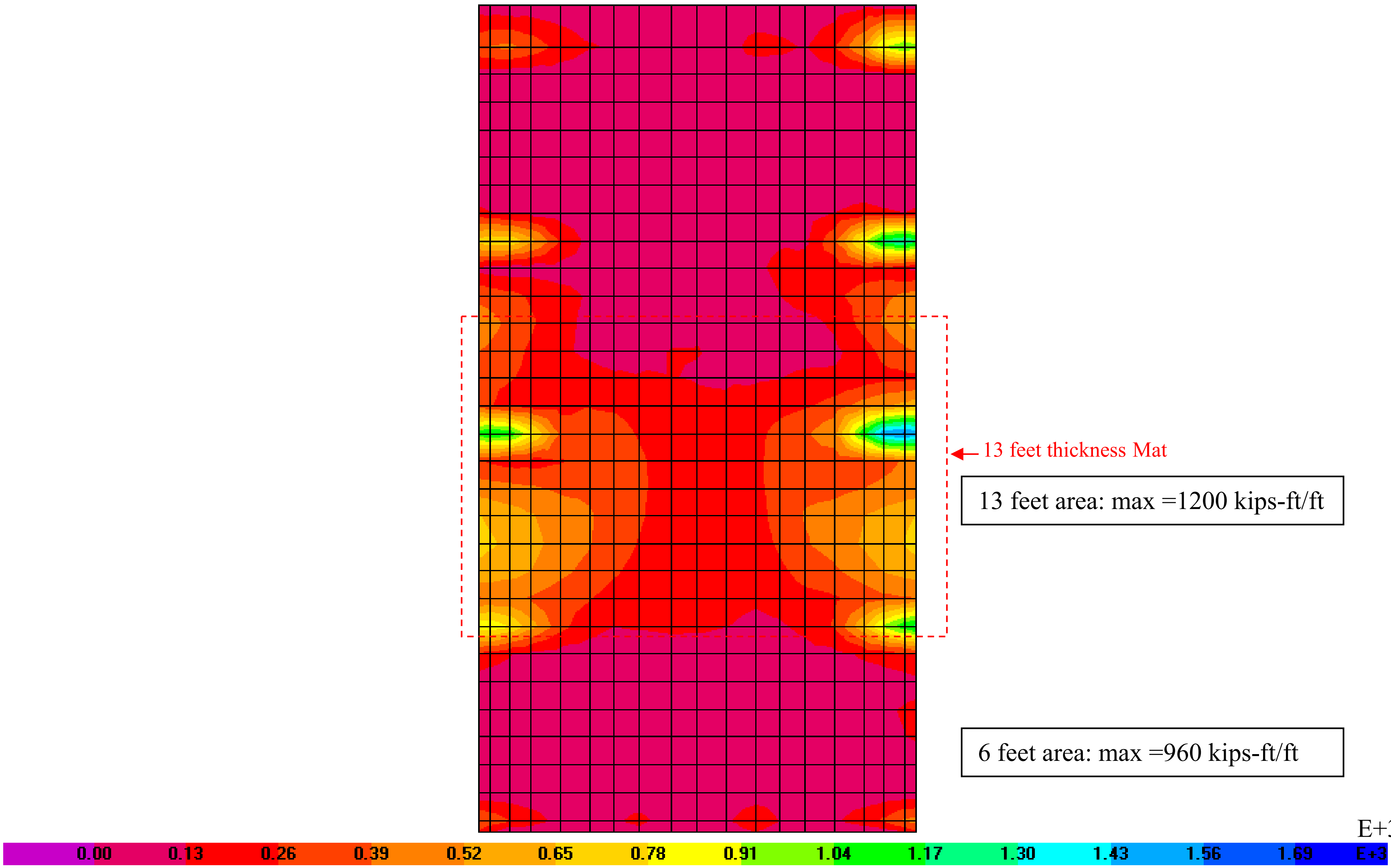


Figure E-3: M22 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 1

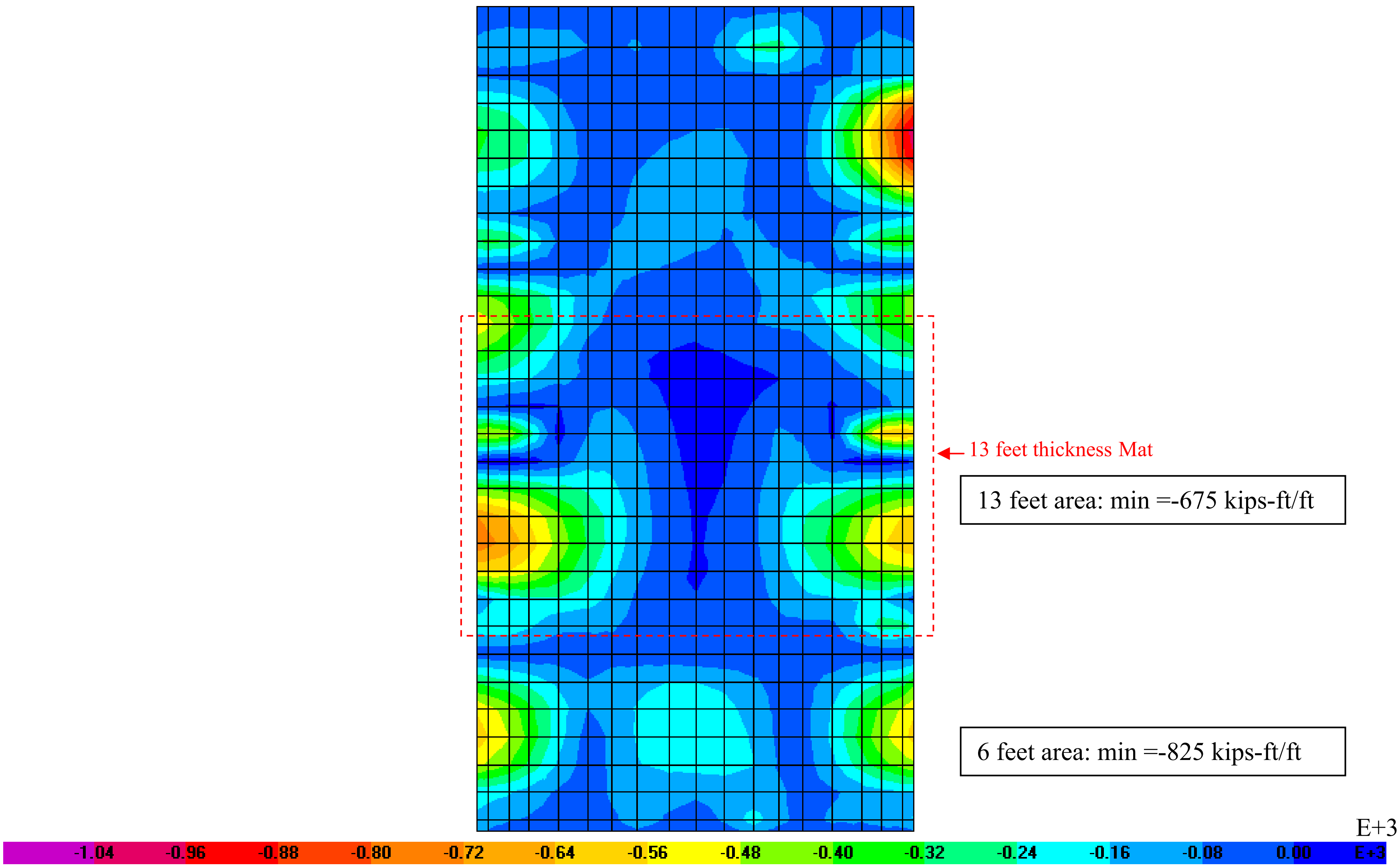


Figure E-4: M22 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 1

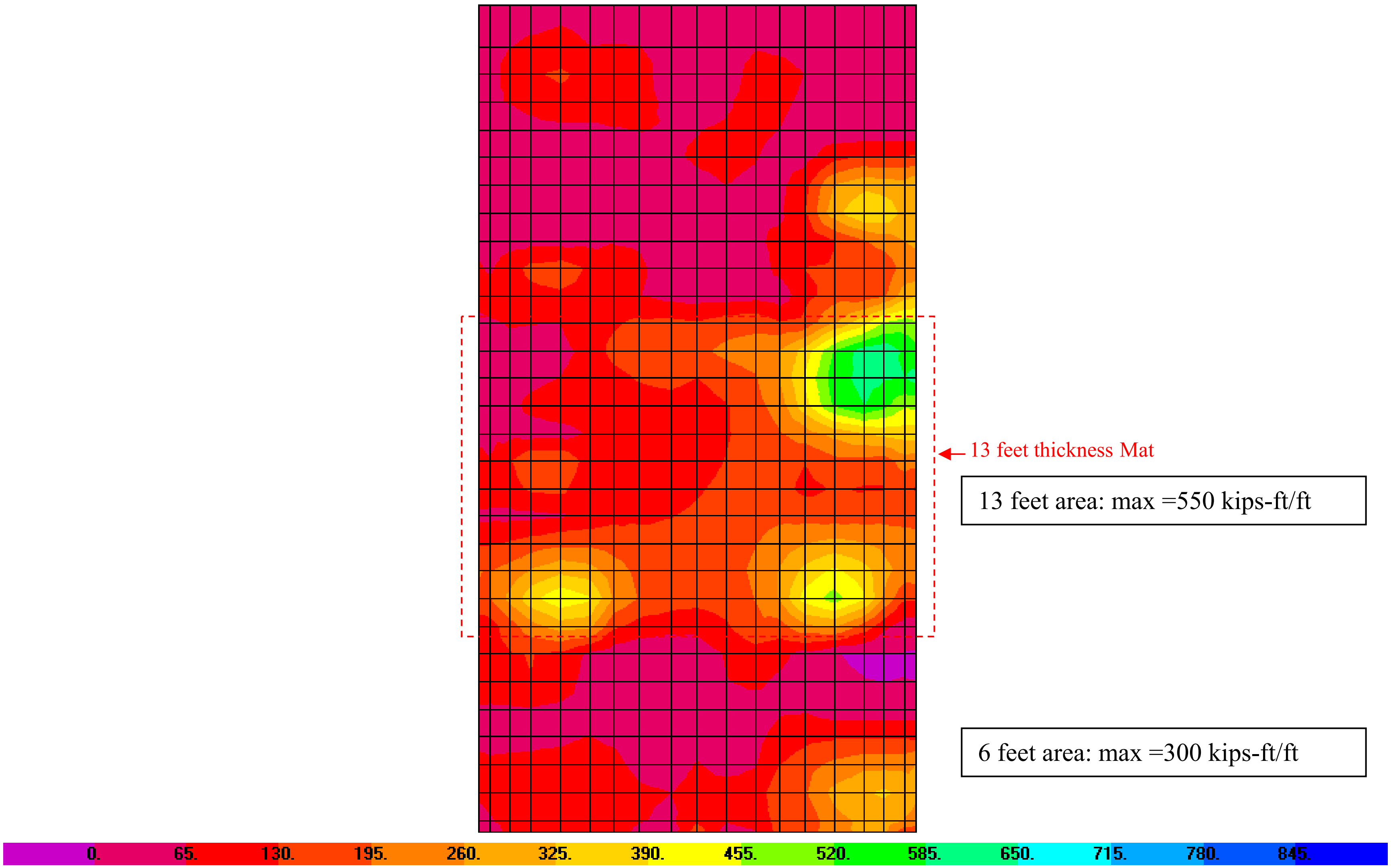


Figure E-5: M12 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 1

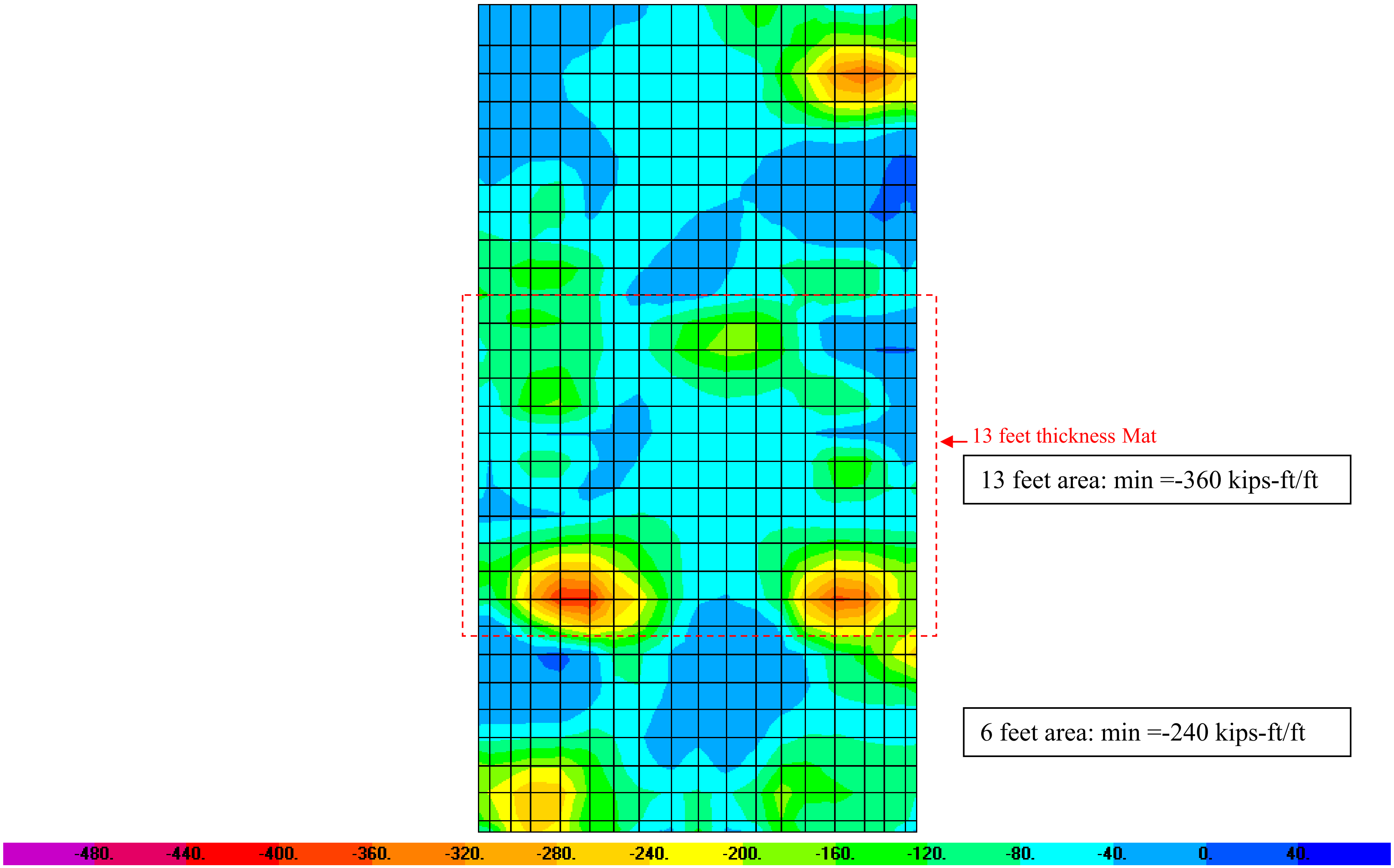


Figure E-6: M12 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 1

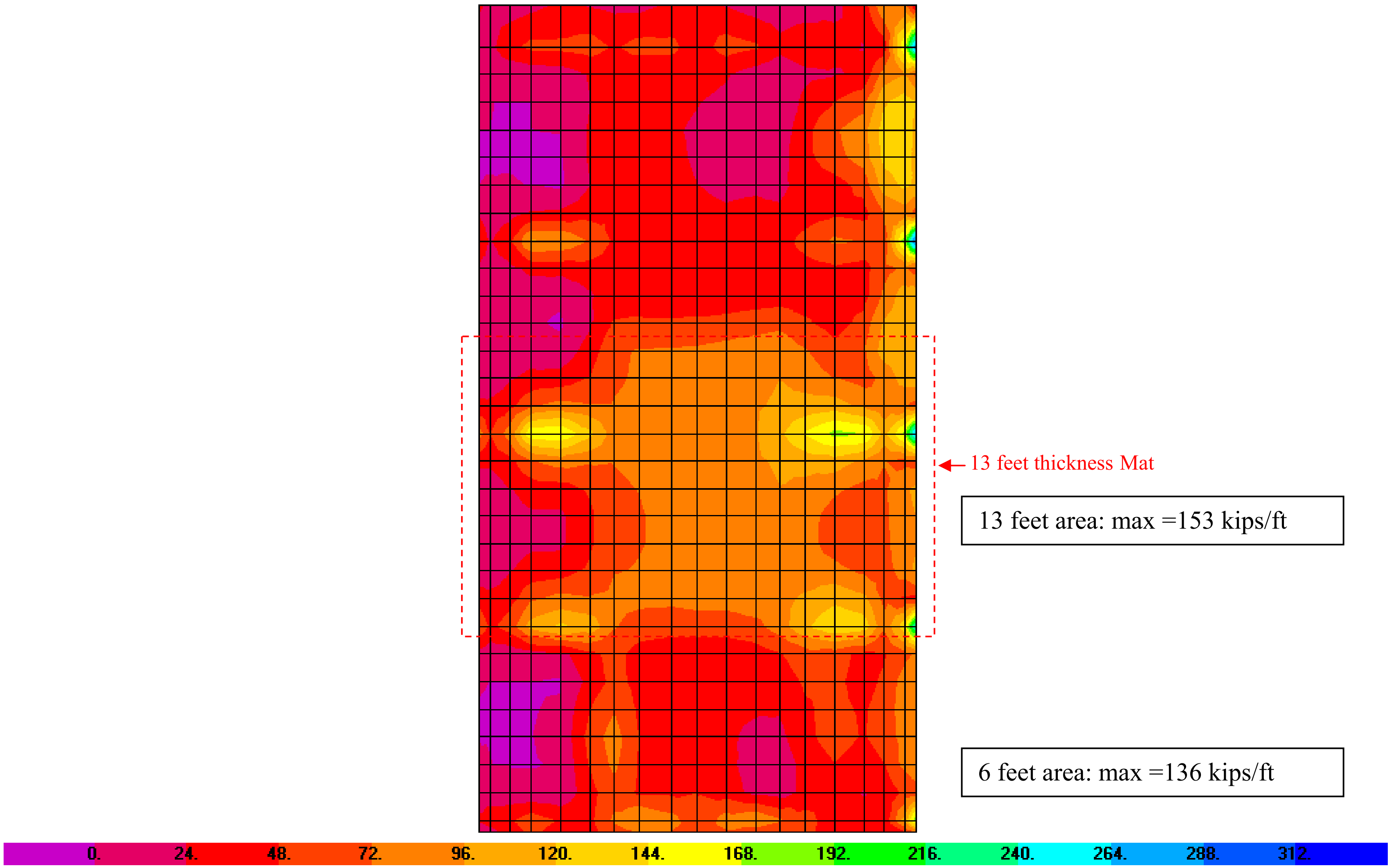


Figure E-7: V13 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 1

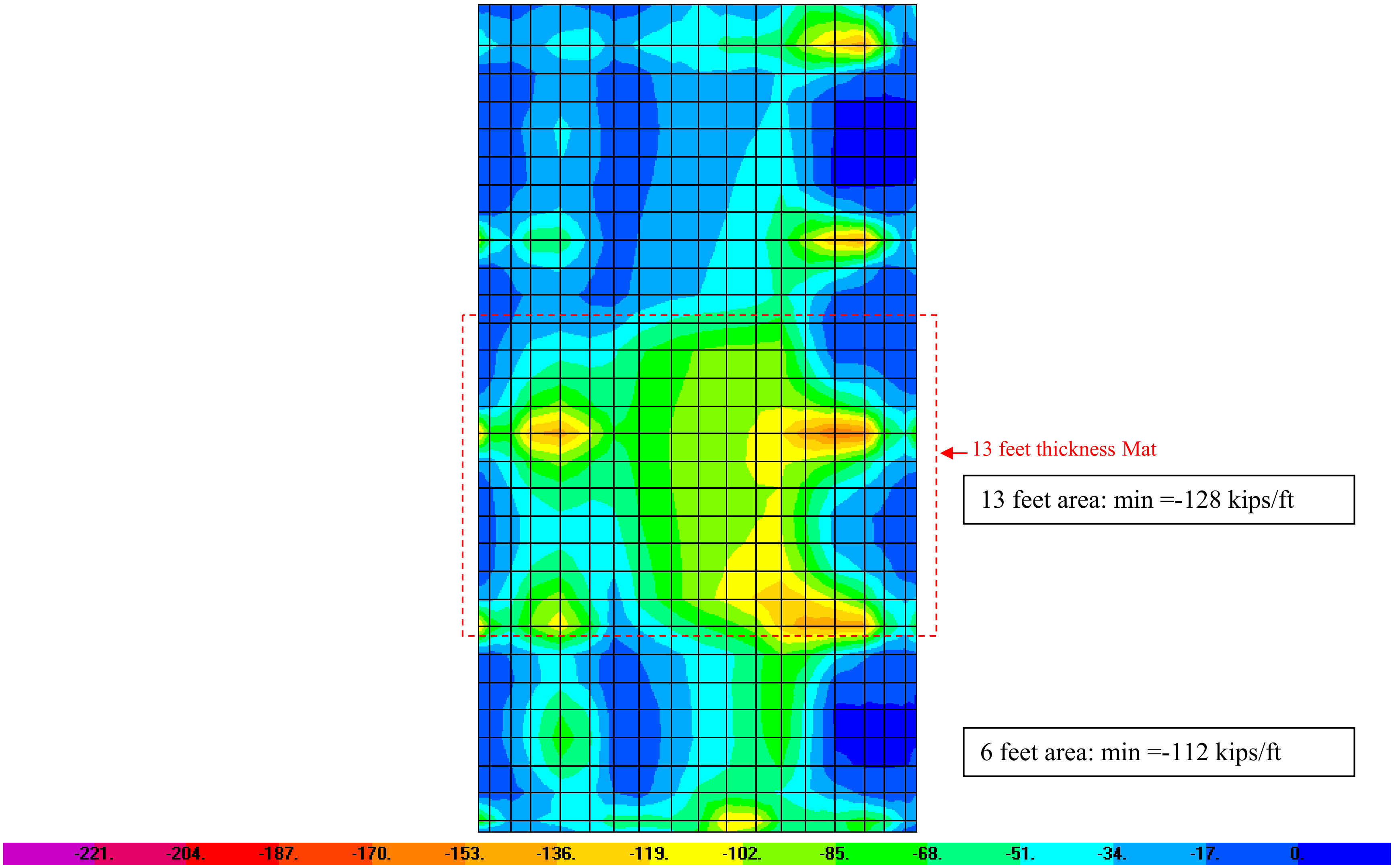


Figure E-8: V13 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 1

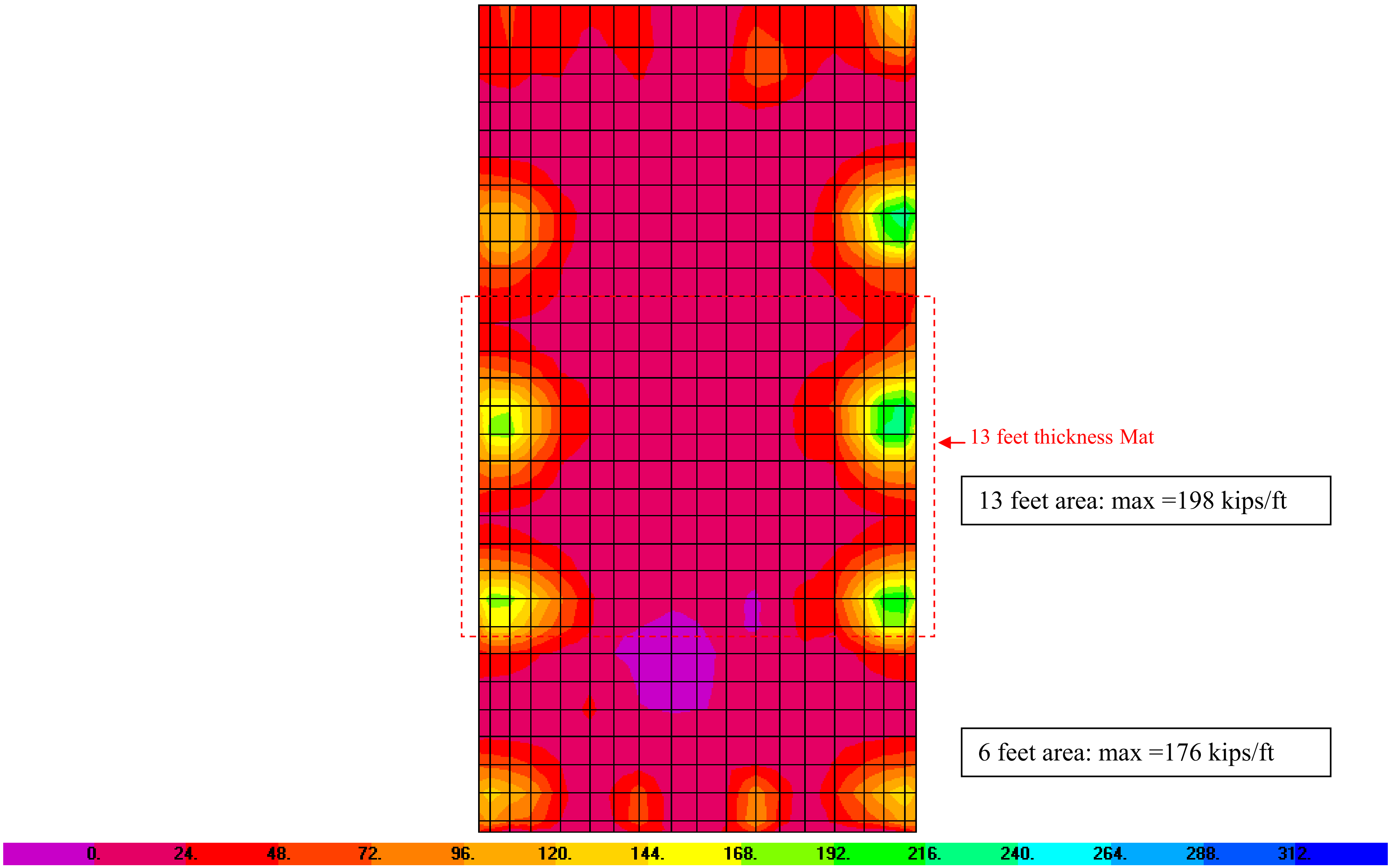


Figure E-9: V23 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 1

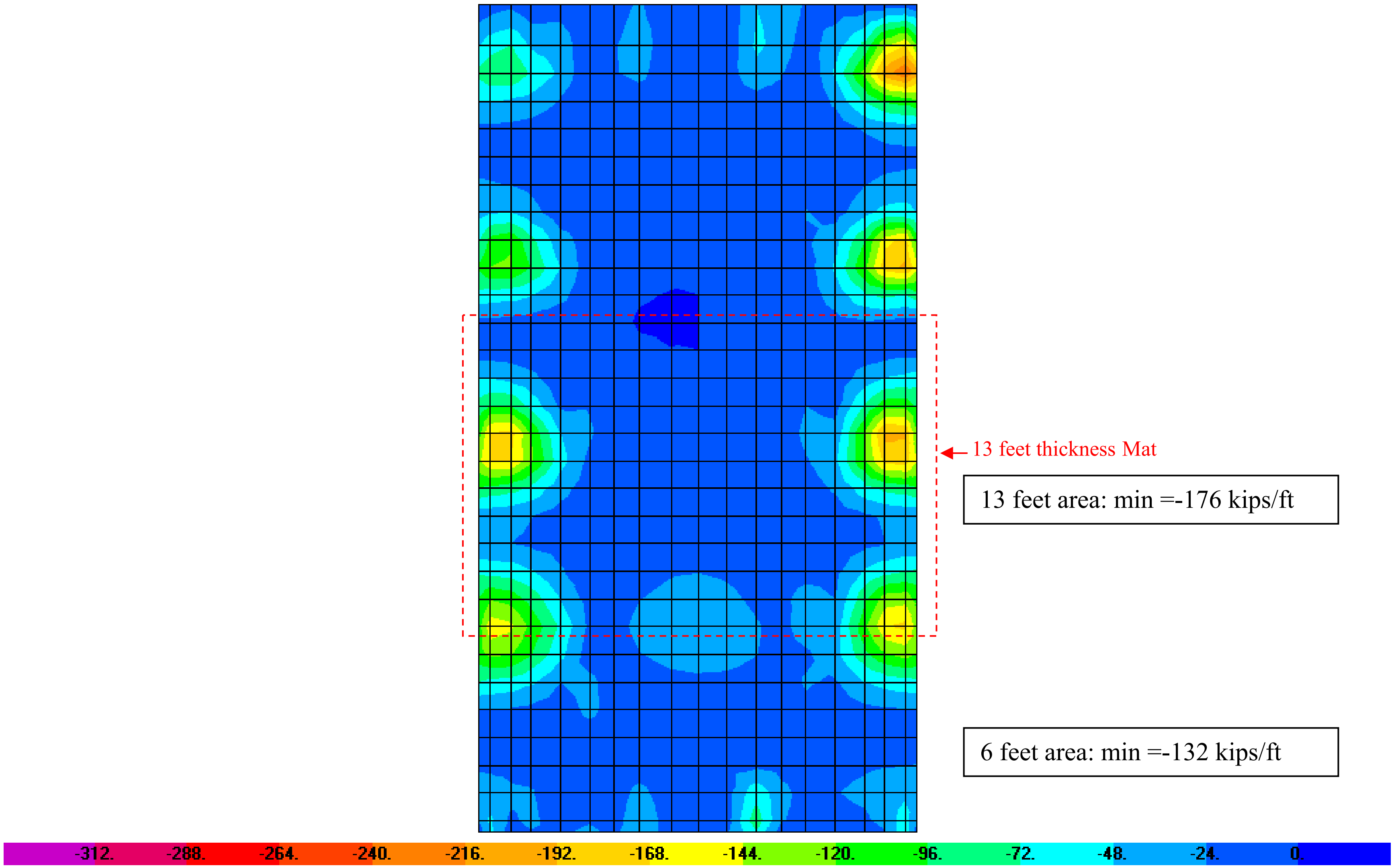


Figure E-10: V23 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 1

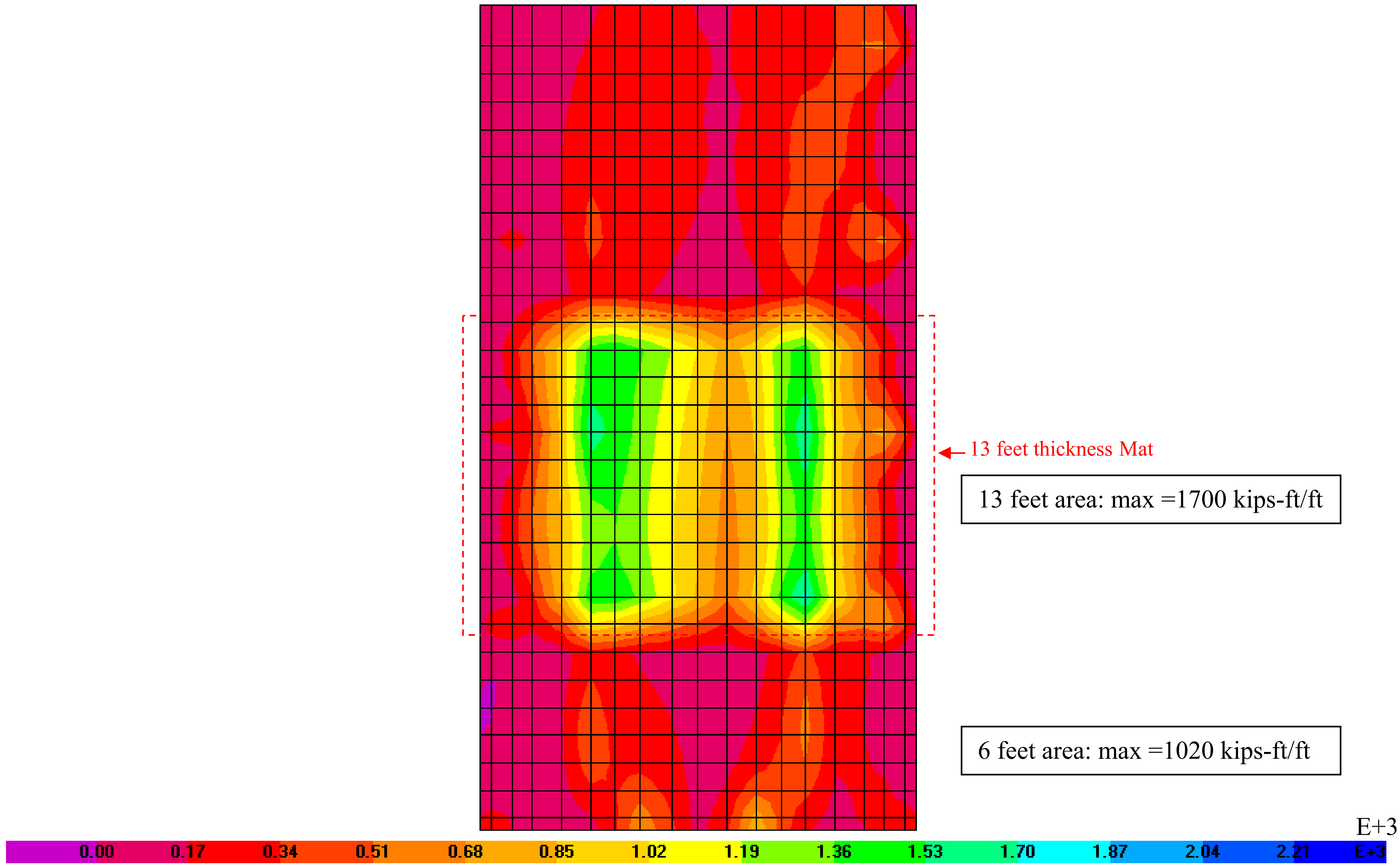


Figure E-11: M11 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 2

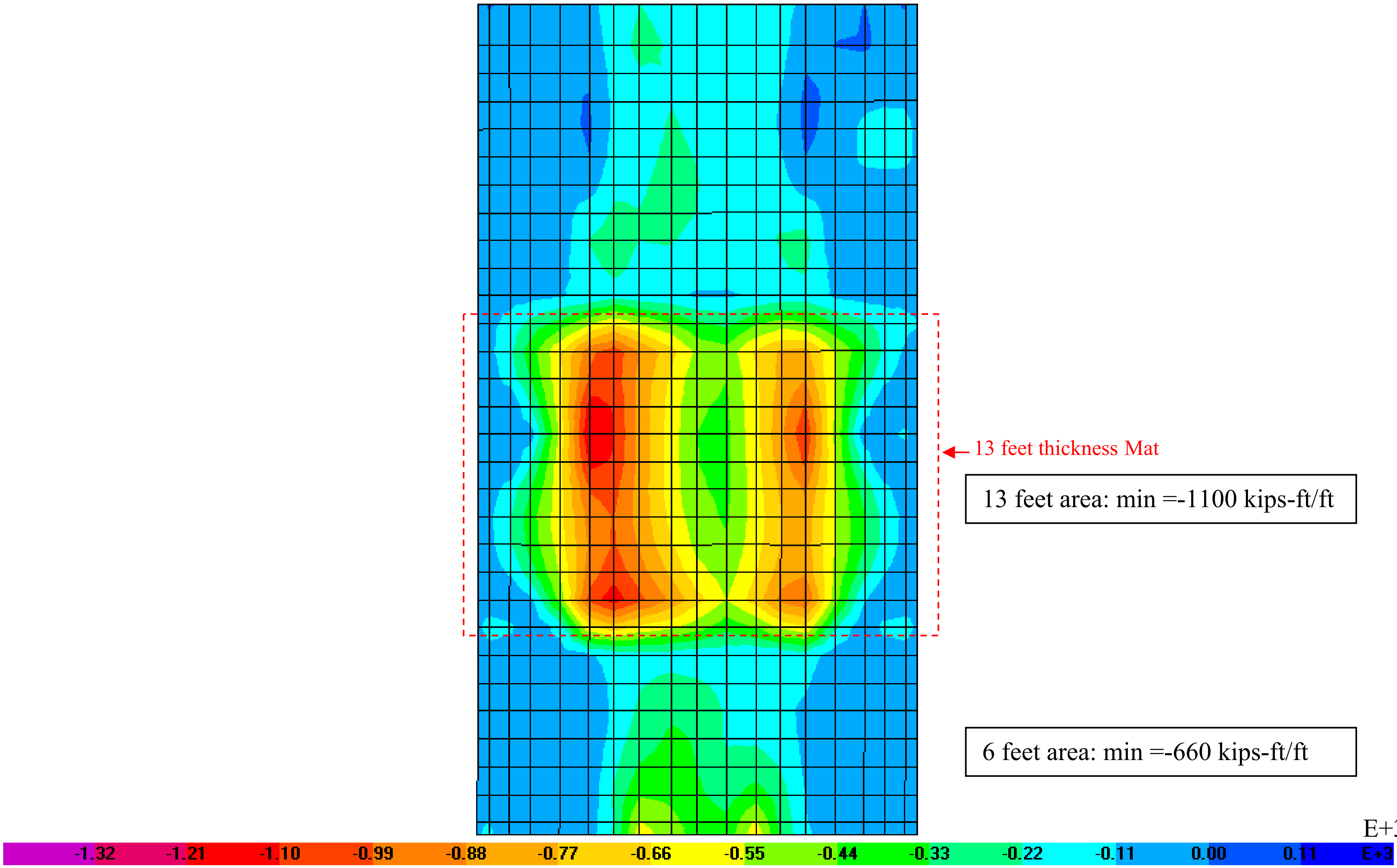


Figure E-12: M11 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 2

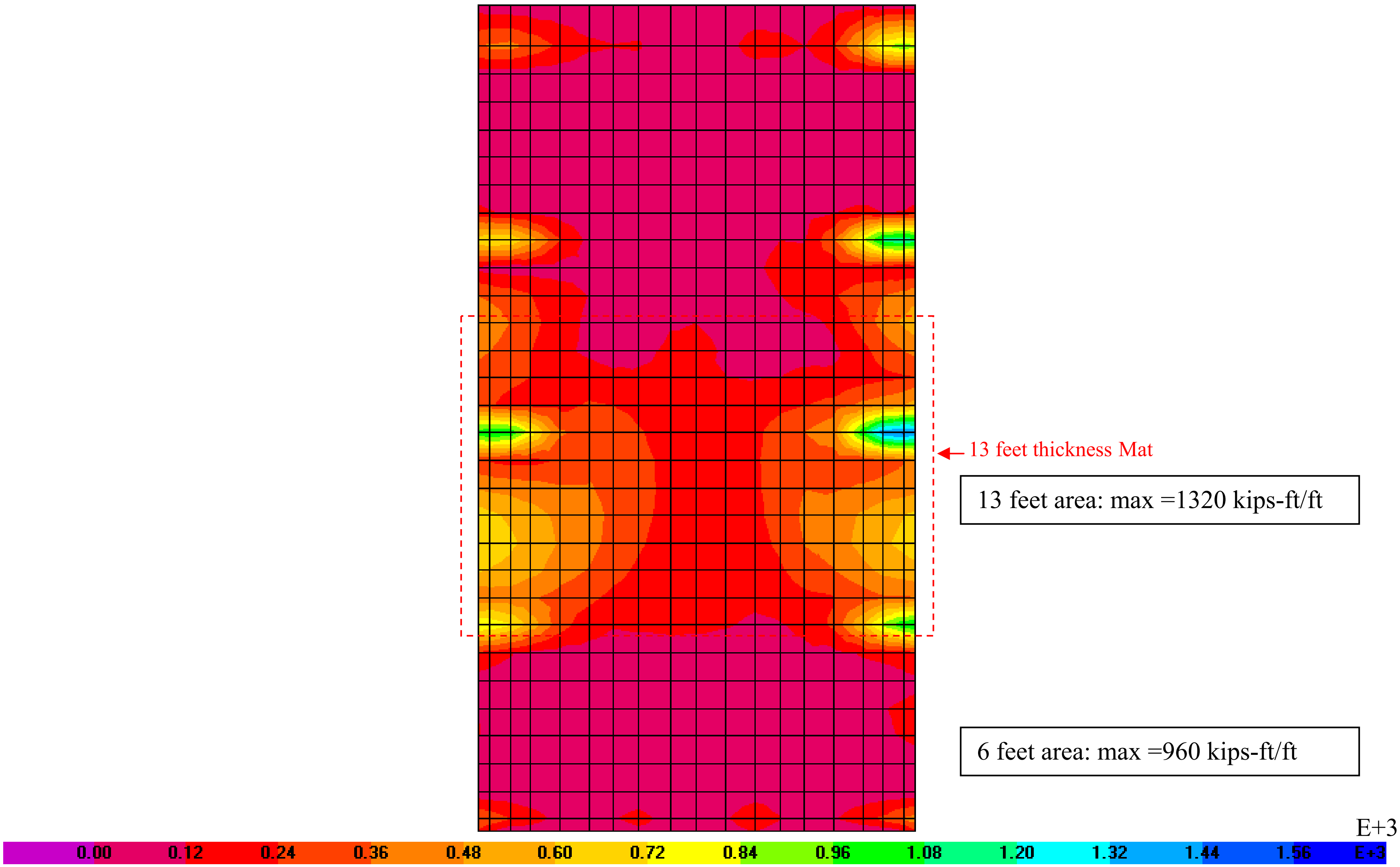


Figure E-13: M22 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 2

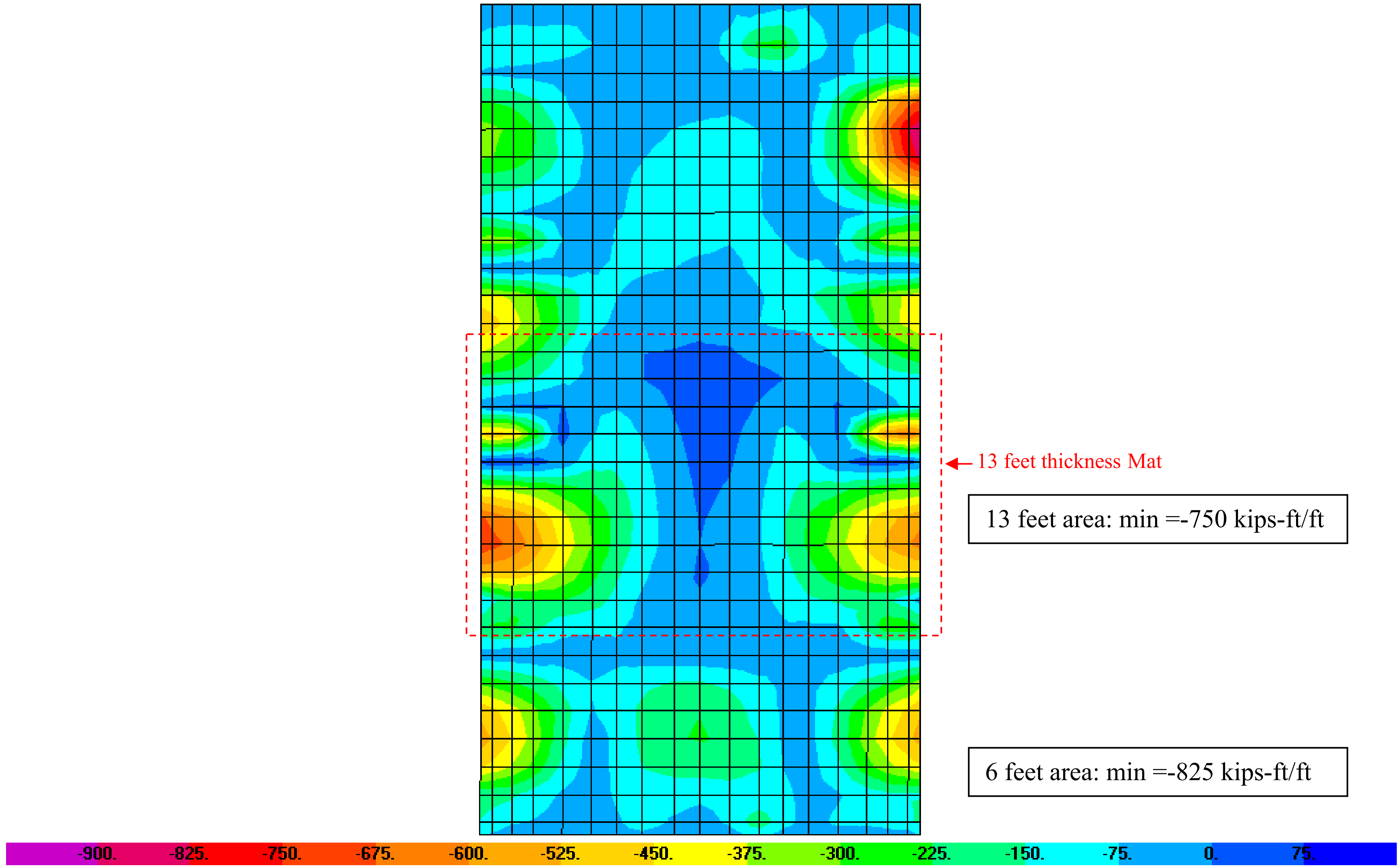


Figure E-14: M22 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 2

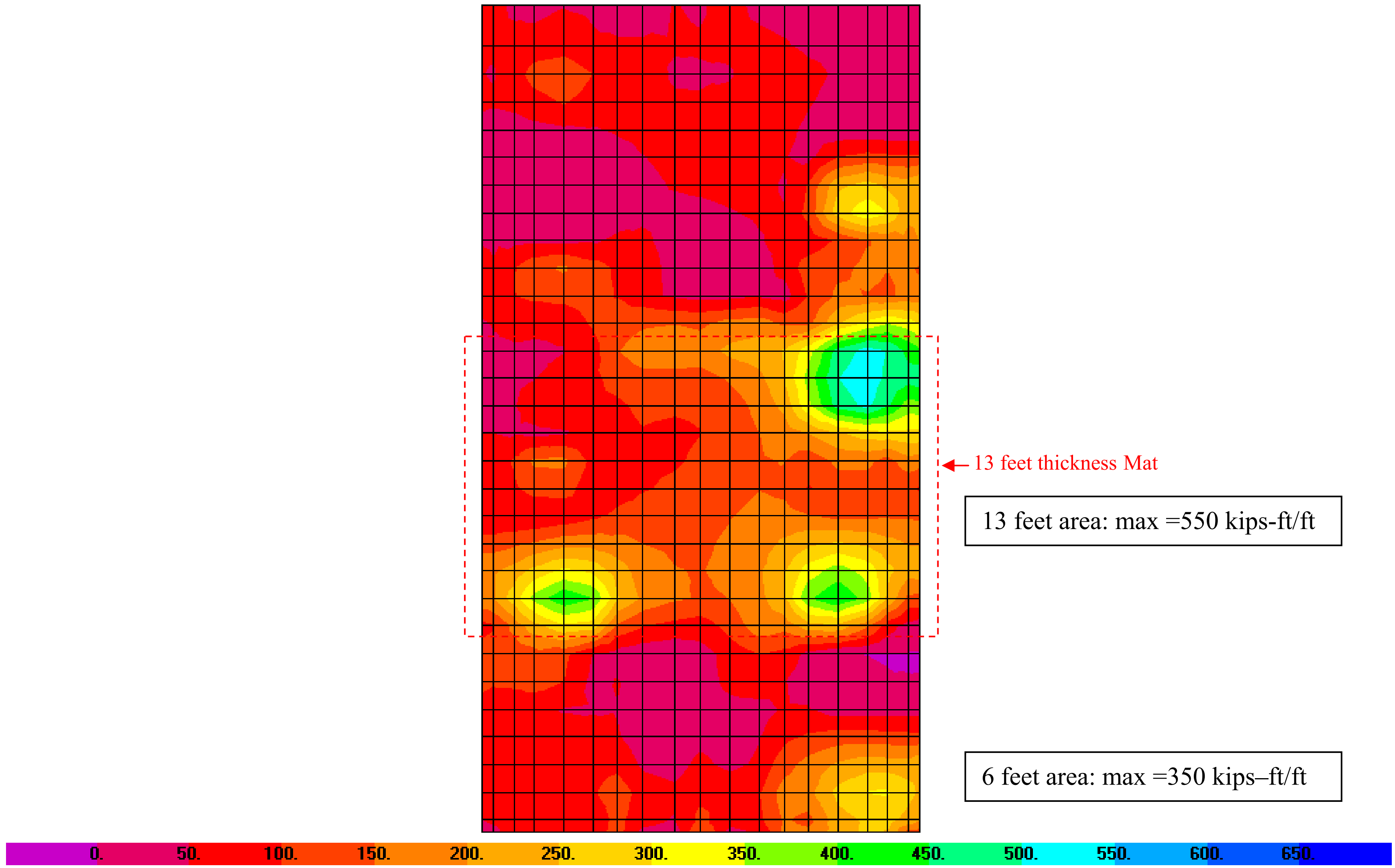


Figure E-15: M12 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 2

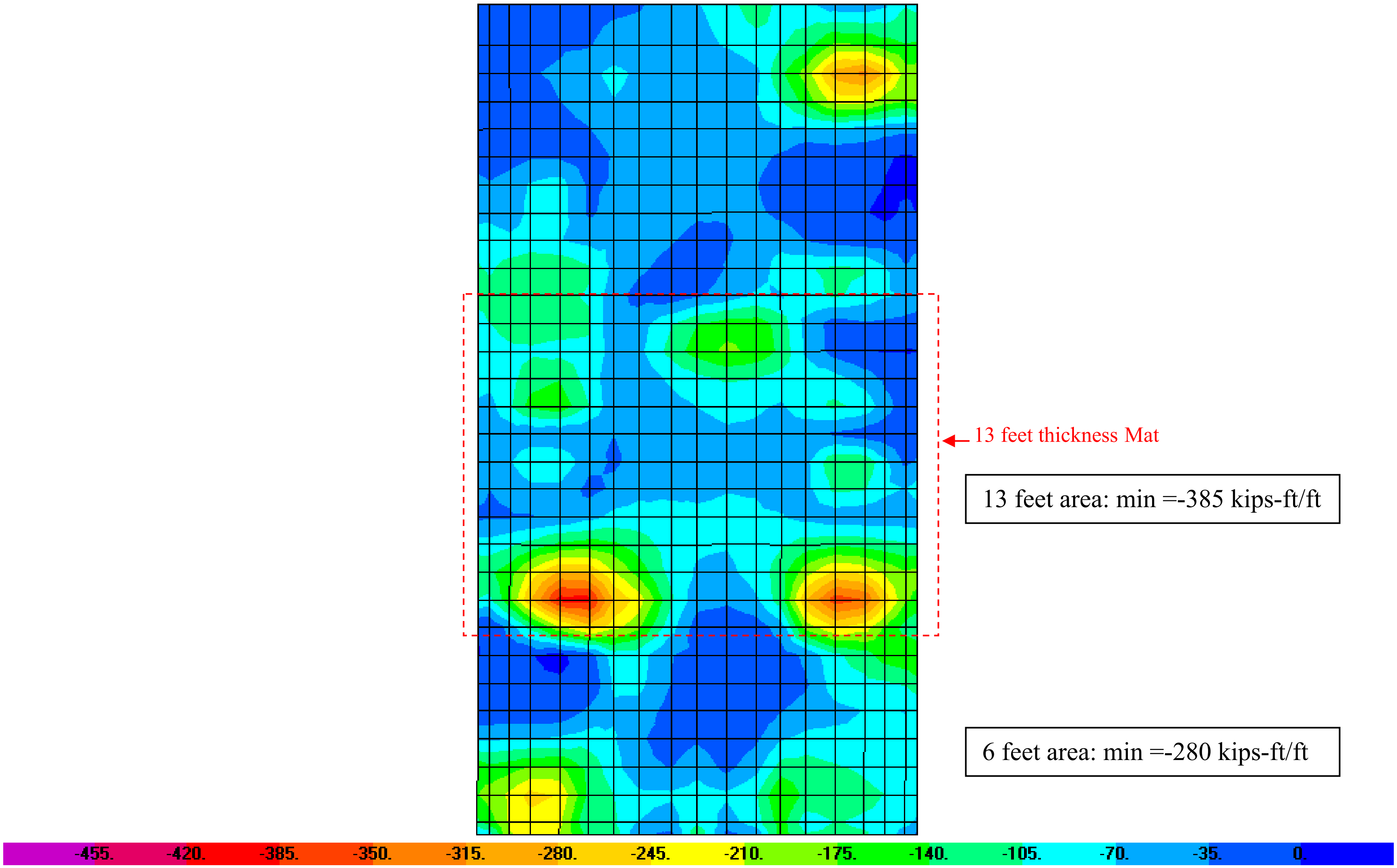


Figure E-16: M12 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 2

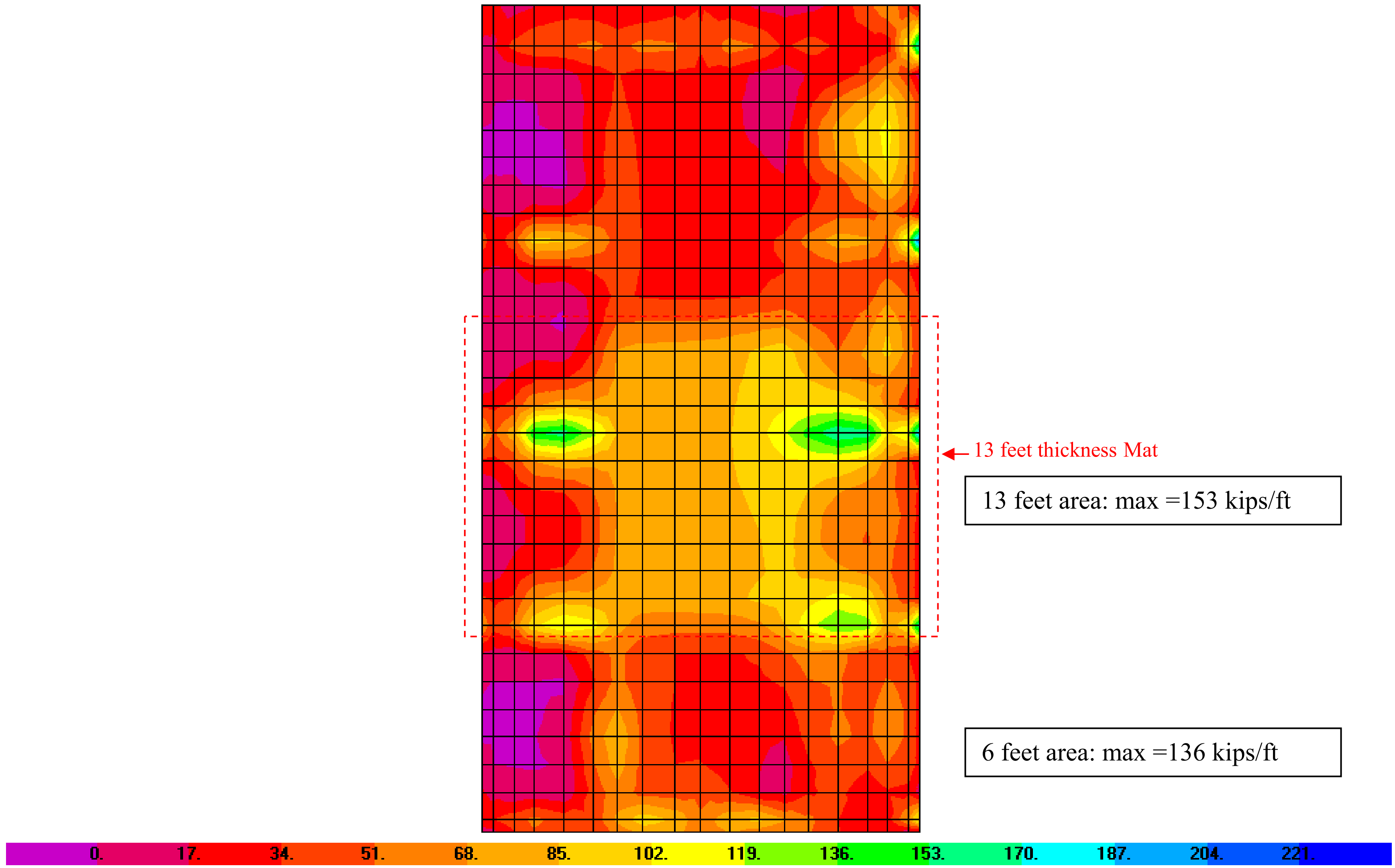


Figure E-17: V13 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 2

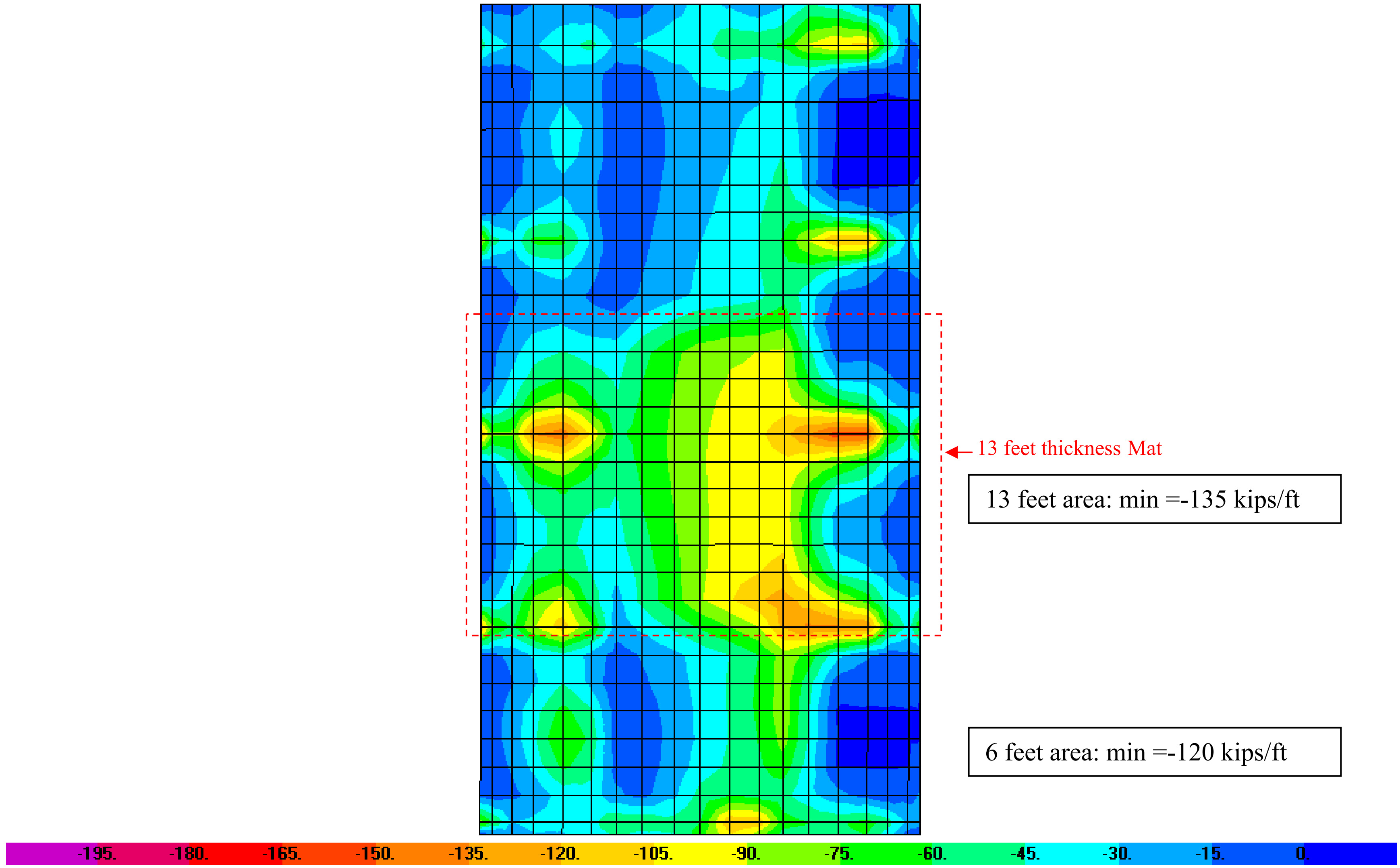


Figure E-18: V13 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 2

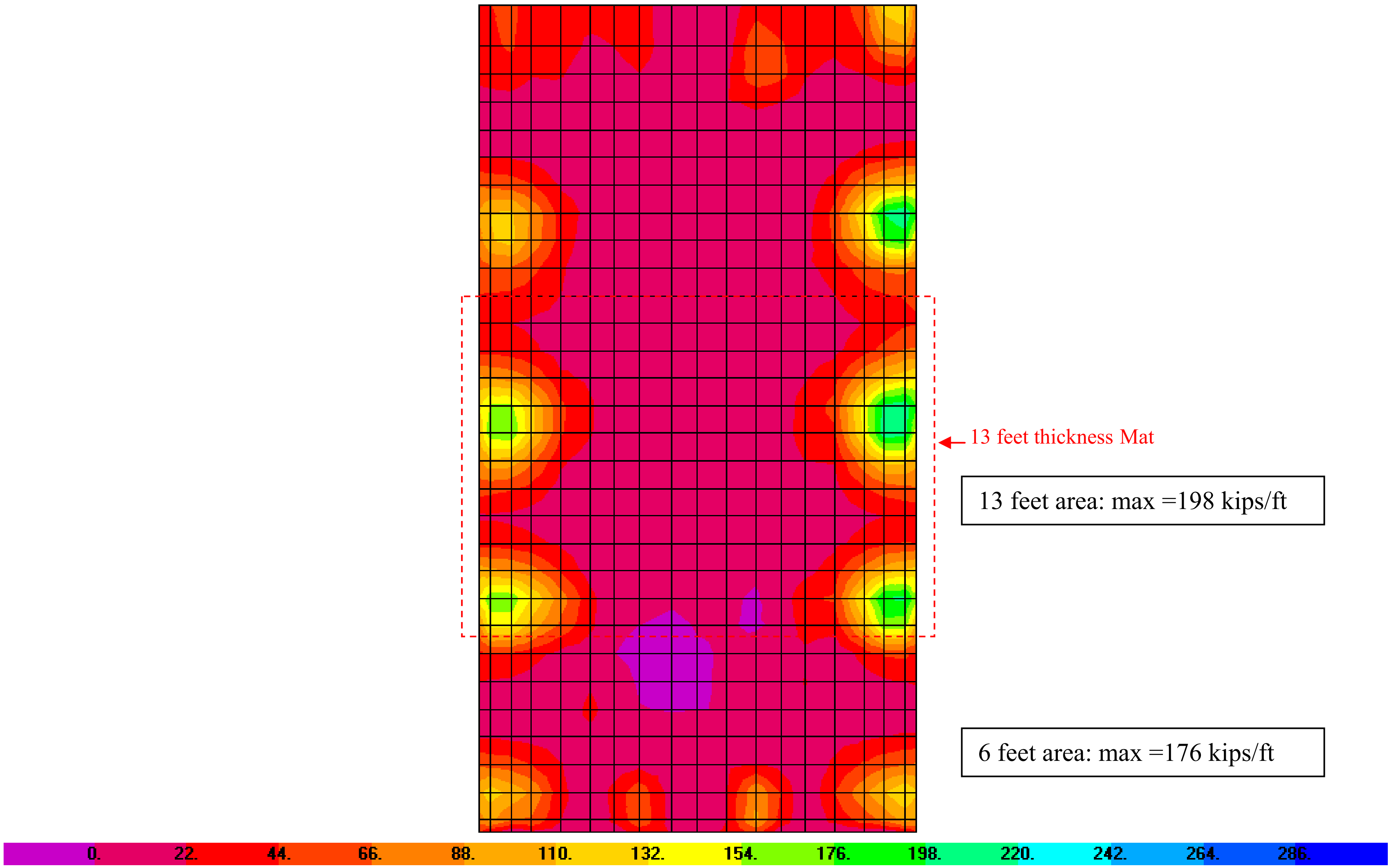


Figure E-19: V23 Maximum Enveloping Stress Contour of Small (west) Mat for Load Case 2

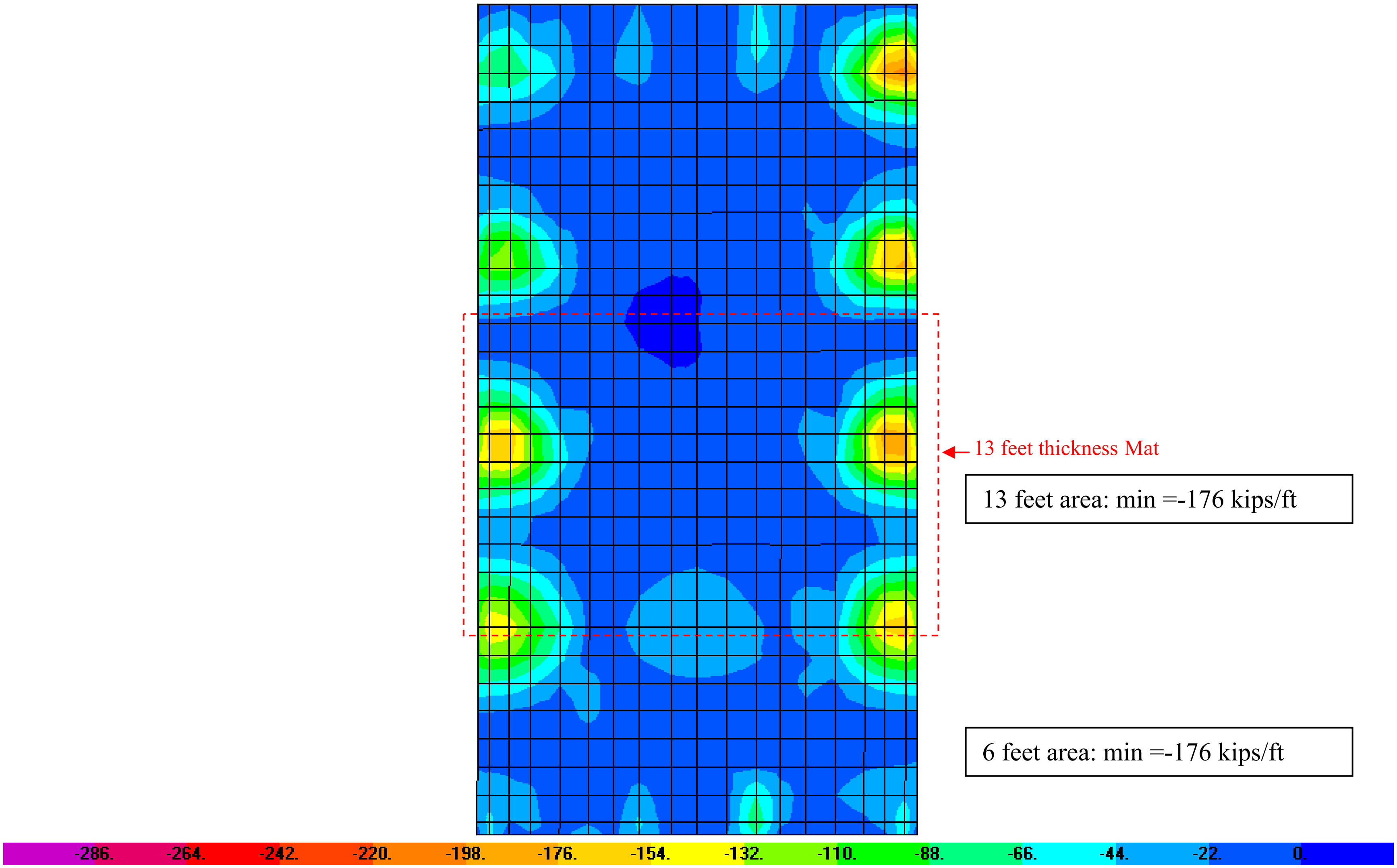


Figure E-20: V23 Minimum Enveloping Stress Contour of Small (west) Mat for Load Case 2

Attachment F

Structural Stability Evaluation - Large Mat

(Refer to the DVD 2 of 2)

Large Foundation Mat Model Explanation

Interior Concrete structure load cases:

Load Case Name	Load Description
Deadload-mat	Superimposed dead load on Mat foundation
Liveload-mat	Superimposed live load on Mat foundation
Dfloor2	Superimposed dead load on roof of interior concrete Superstructure
Lfloor2	Superimposed live load on roof of interior concrete Superstructure
SelfWt	Self weight of concrete mat foundation and Self weight of concrete superstructure
E_FX	X direction component earthquake equivalent static load of concrete superstructure
E_FY	Y direction component earthquake equivalent static load of concrete superstructure
E_FZ	Z direction component earthquake equivalent static load of concrete superstructure

Steel Structure Load Cases:

Load Case Name	Load Description
Platformdead	Superimposed dead load on platforms in steel model
Platformlive	Superimposed dead load on platforms in steel model
Roof + Cladding	Dead load due to superimposed dead load on steel roof and self-weight of cladding
Rooflive	Superimposed live load on steel roof
Cranedead	Dead load due to crane
Cranelive	Live load due to crane
Self + Rail	Self-weight of steel structure and crane rail
Snow	Load due to snow and snow drift
StaticEX	X direction component earthquake equivalent static load of Steel structure
StaticEY	Y direction component earthquake equivalent static load of Steel structure
StaticEZ	Z direction component earthquake equivalent static load of Steel structure

Attachment G

Structural Stability Evaluation - Small Mat

(Refer to the DVD 2 of 2)

Small Foundation Mat Model Load Case Explanation
Exterior Concrete load cases:

Load Case Name	Load Description
Deadload-mat	Superimposed dead load on Mat
Liveload-mat	Superimposed live load on Mat
SELF	Self weight of concrete mat foundation
W_Droof+Dcrane	Superimposed dead load on roof and dead load of the crane of Superstructure
W_Lcrane	Superimposed crane live load on Superstructure
W_Lroof	Superimposed live load on roof of Superstructure
W_SelfWt	Self weight of concrete superstructure
W_FX	X direction component earthquake equivalent static load of superstructure
W_FY	Y direction component earthquake equivalent static load of superstructure
W_FZ	Z direction component earthquake equivalent static load of superstructure

Attachment H

Combined Model for SASSI Input

(Details of the combined model are provided in DVD)

Page

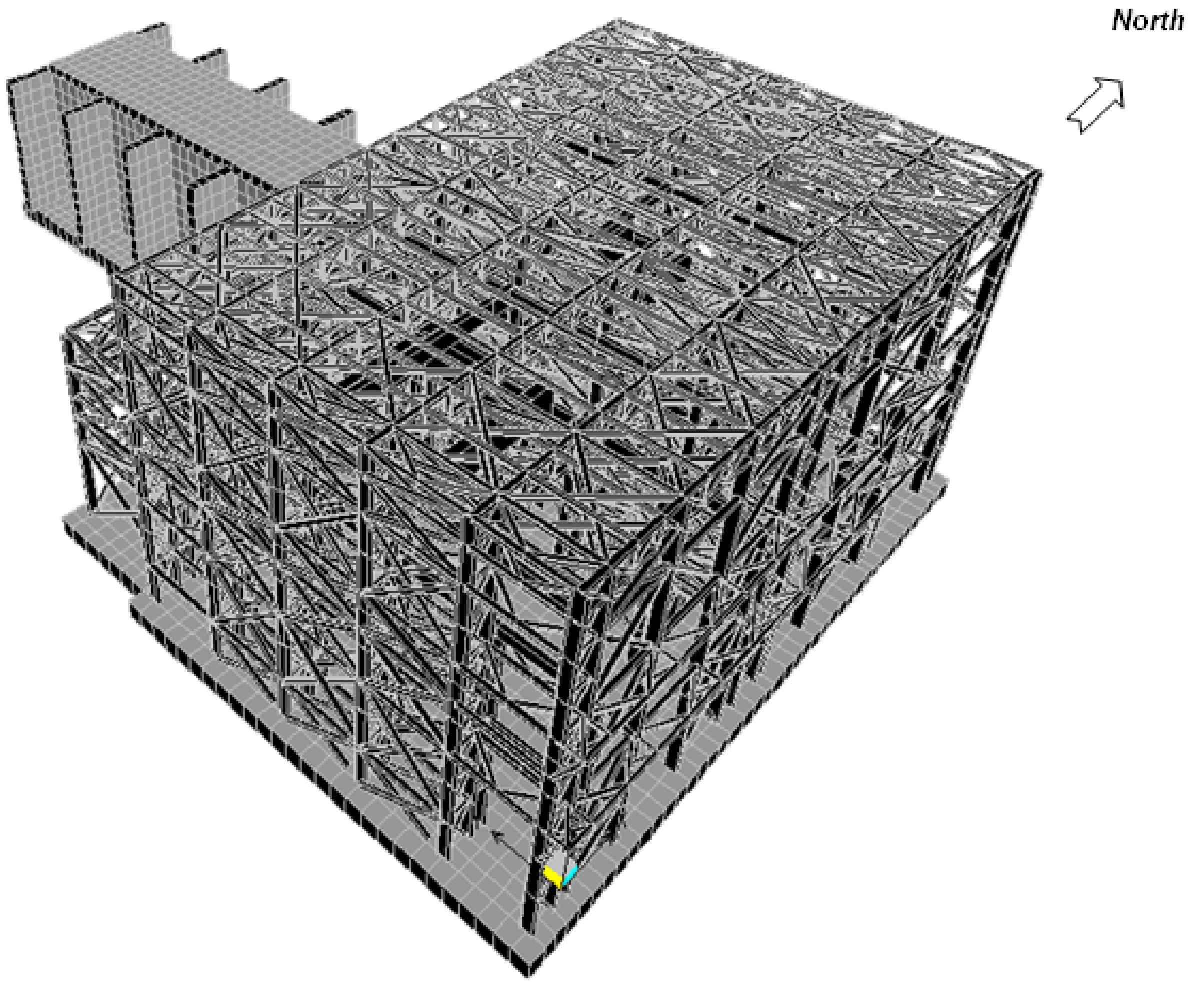


Figure H- 1: South East Isometric View of Steel, Concrete, and Mat Structures

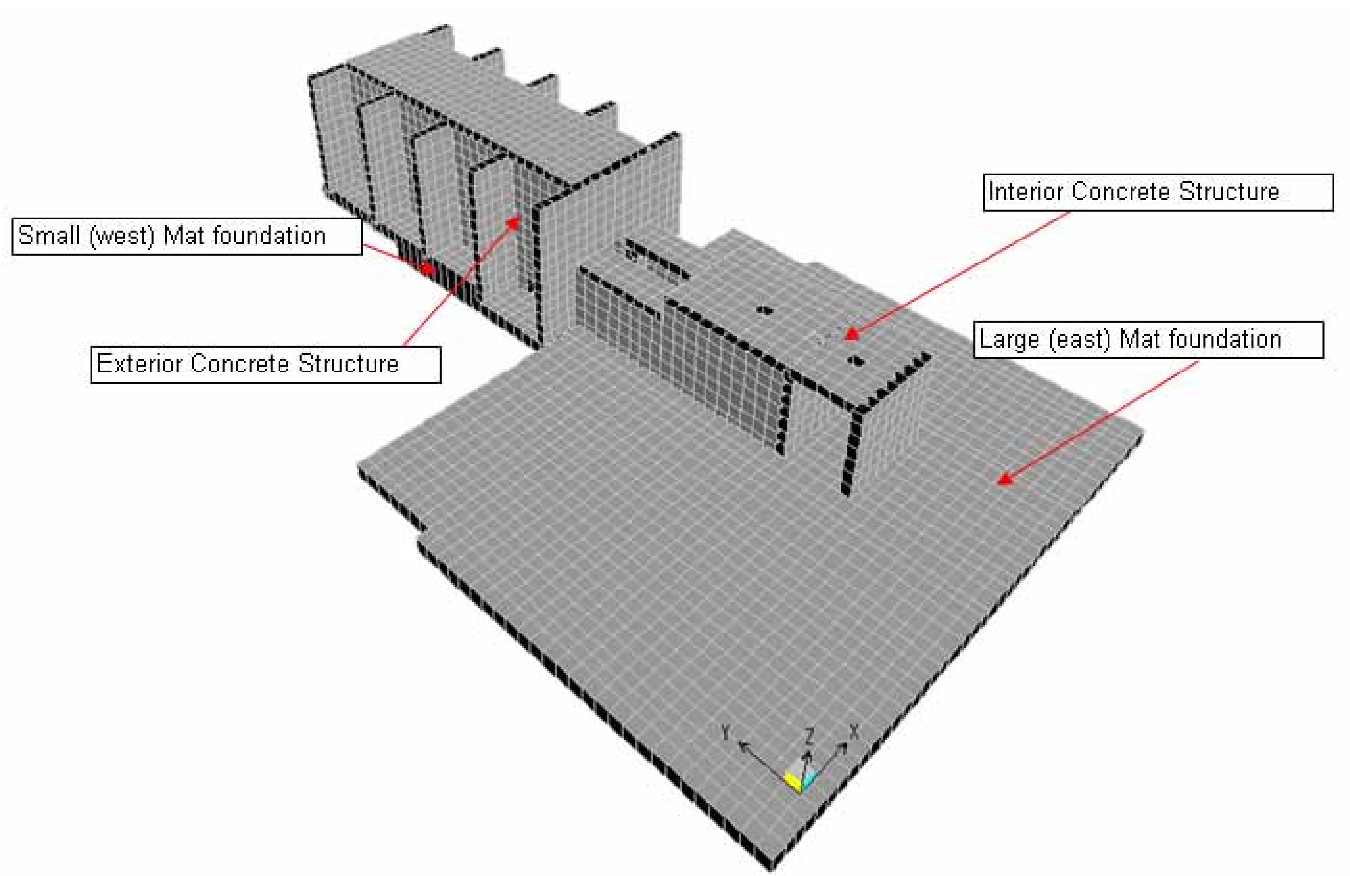


Figure H- 2: South East Isometric View of Combined Concrete and Mat Structures

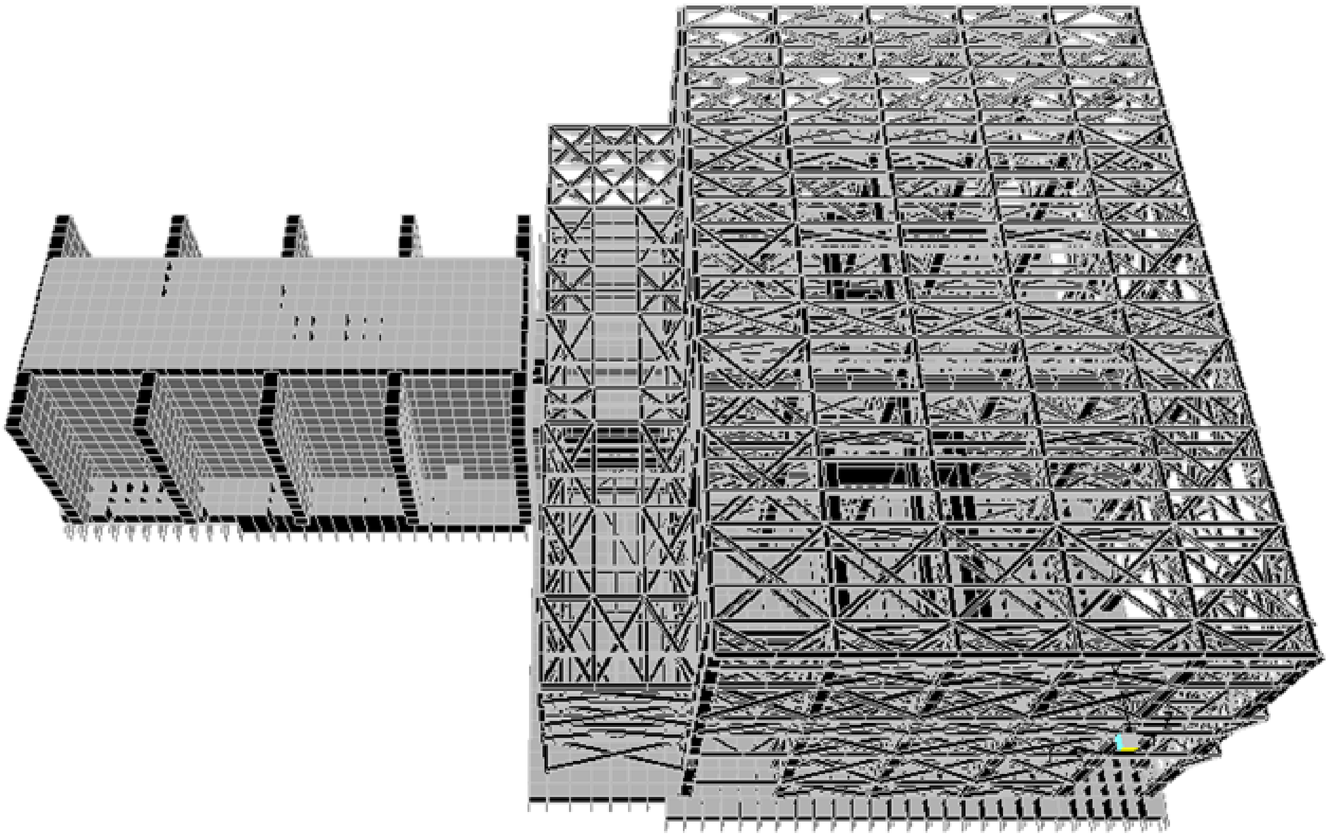


Figure H- 3: South Top Isometric View of Combined Steel, Concrete and Mat Structures

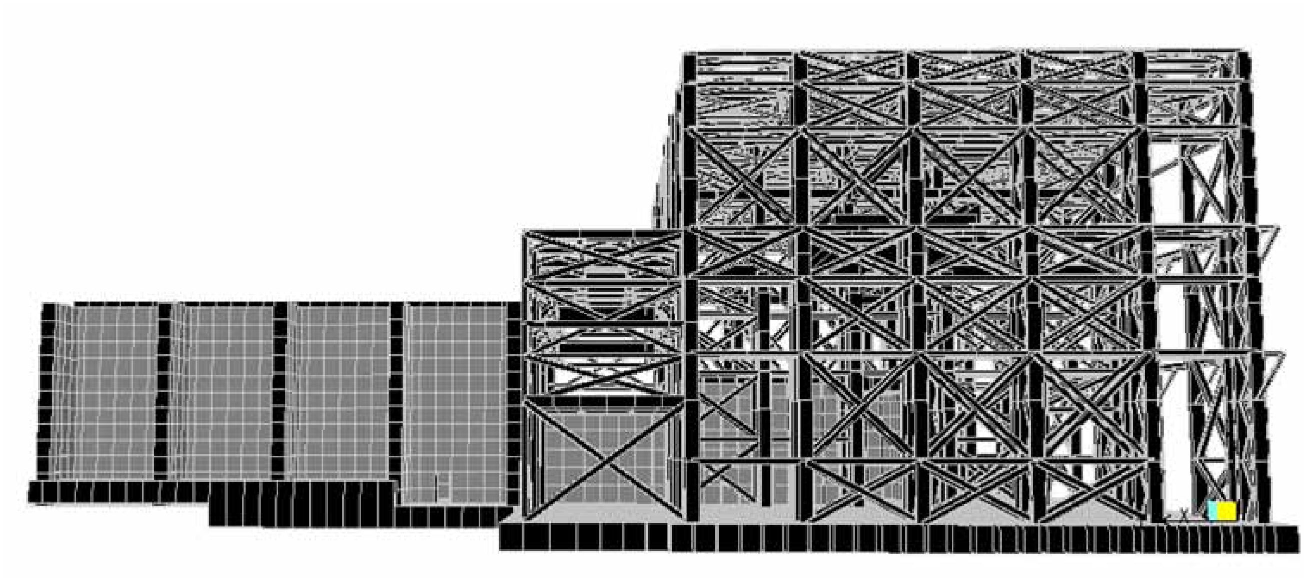


Figure H- 4: South Isometric View of Combined Steel, Concrete, and Mat Structures

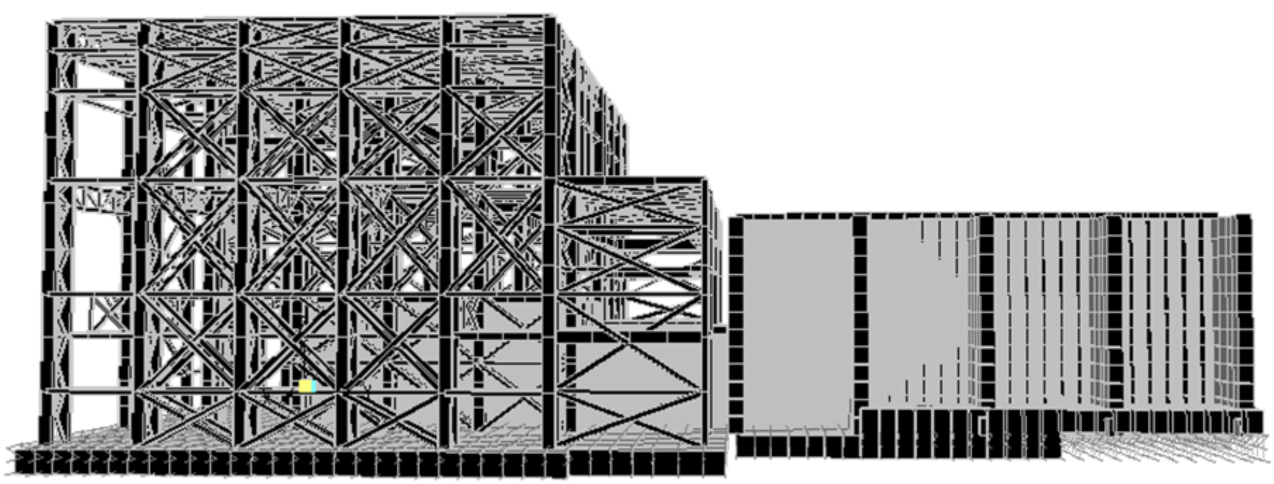


Figure H- 5: North Isometric View of Combined Steel, Concrete, and Mat Structures

Attachment I

Small Mat Vertical Link Soil Spring Properties

(Refer to the DVD 2 of 2)

Attachment J

Steel Structure Building Crane Load Cases

List of Content: **page**

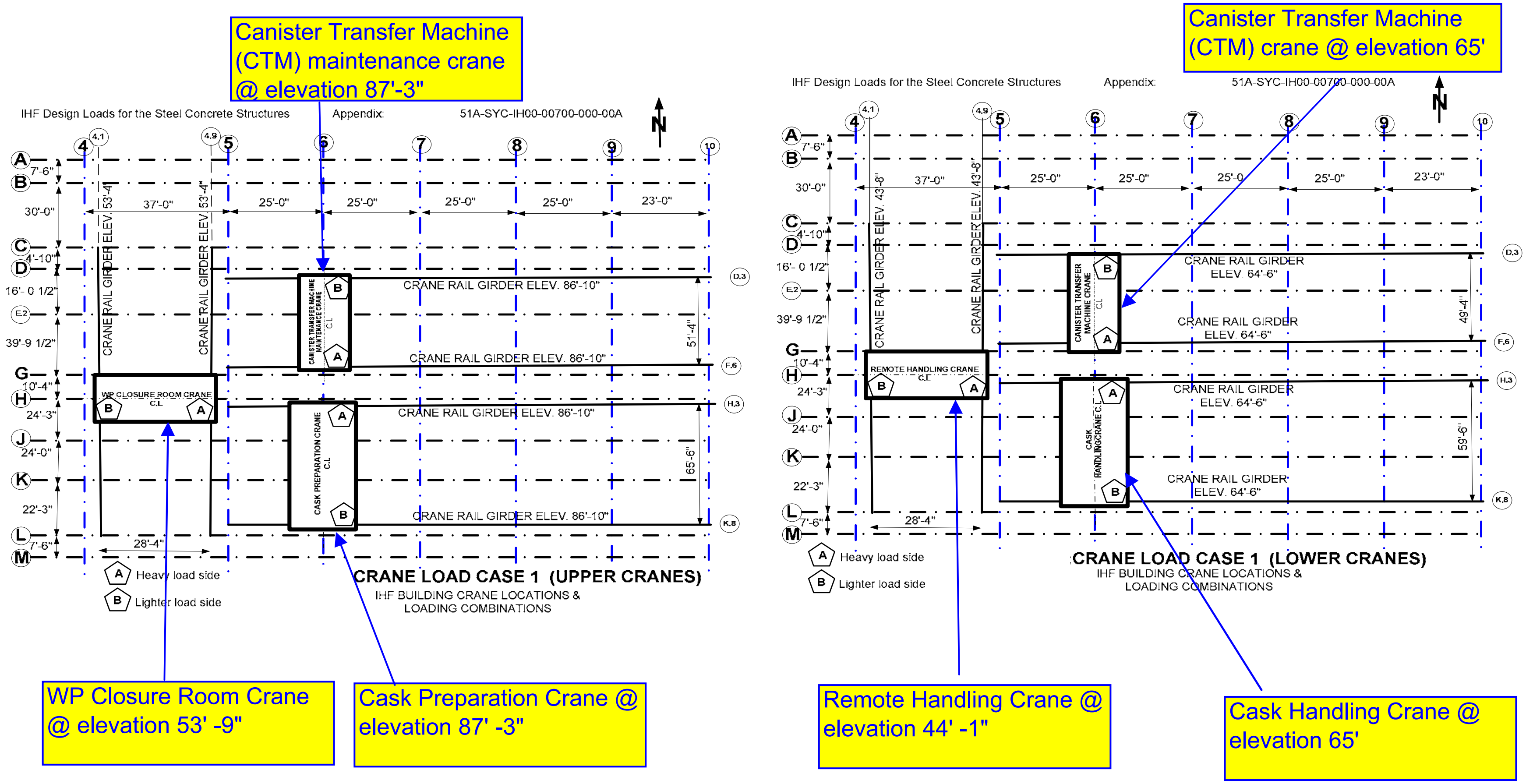


Figure J-1: Crane Load Case 1

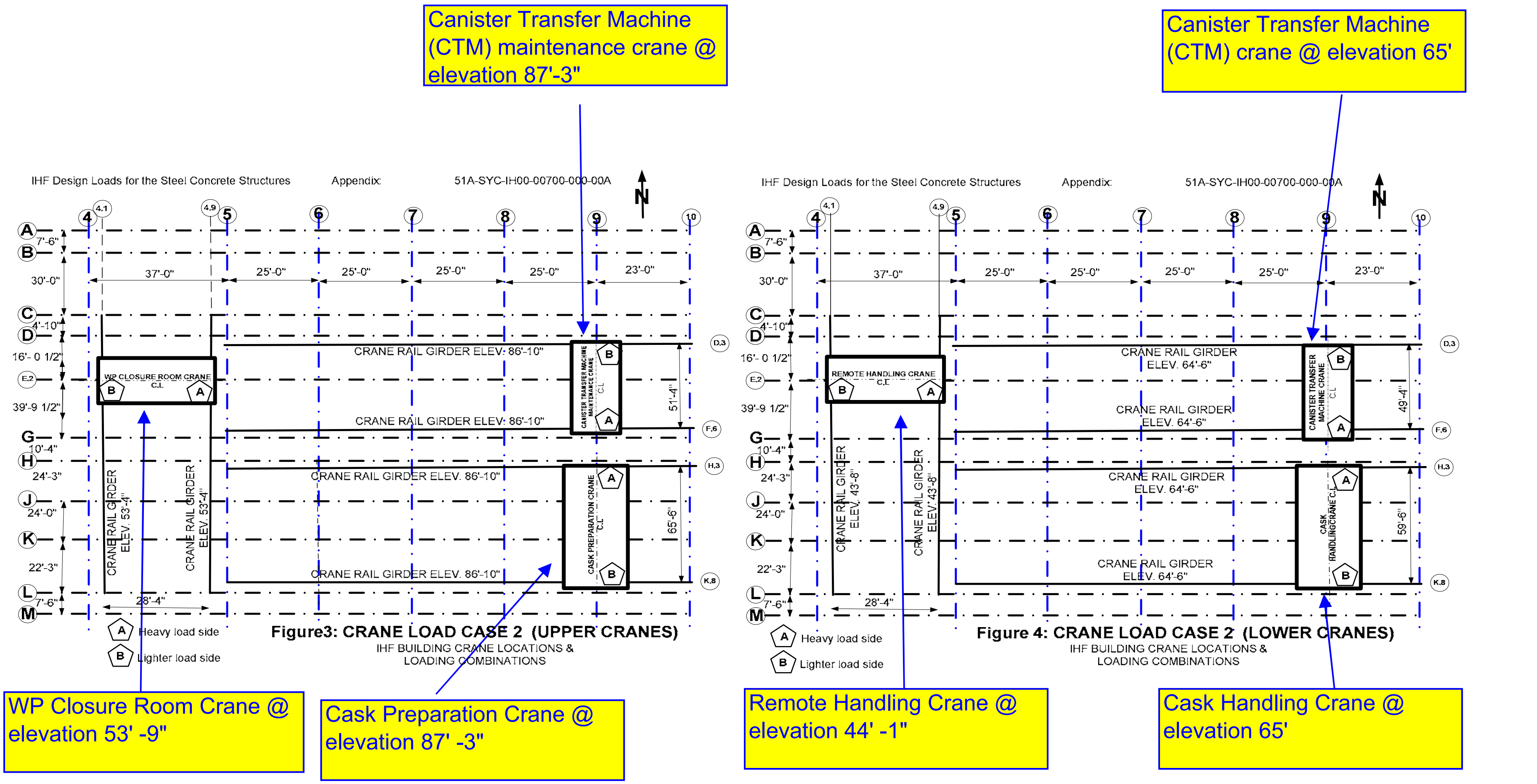


Figure J-2: Crane Load Case 2

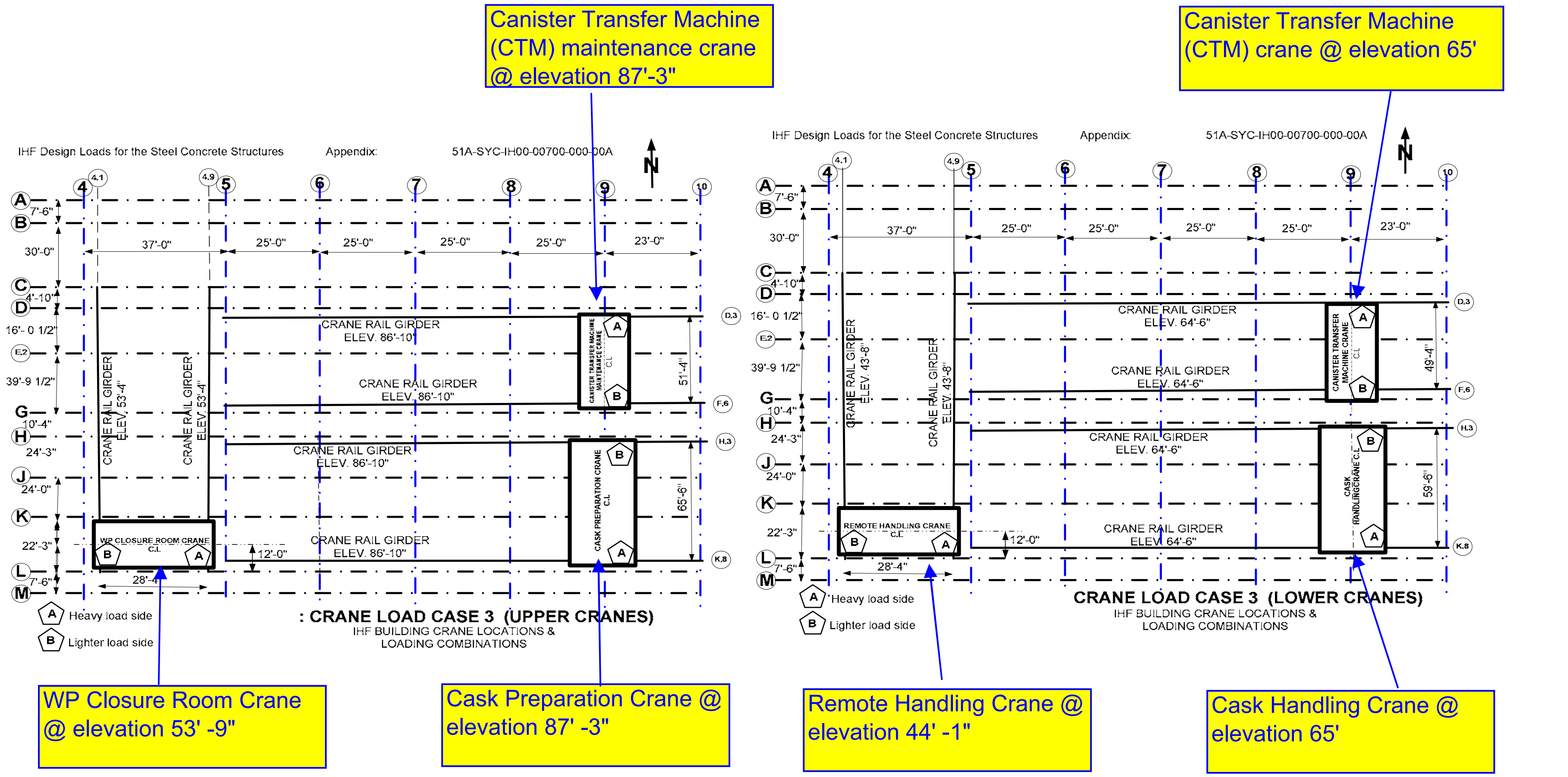


Figure J-3: Crane Load Case 3

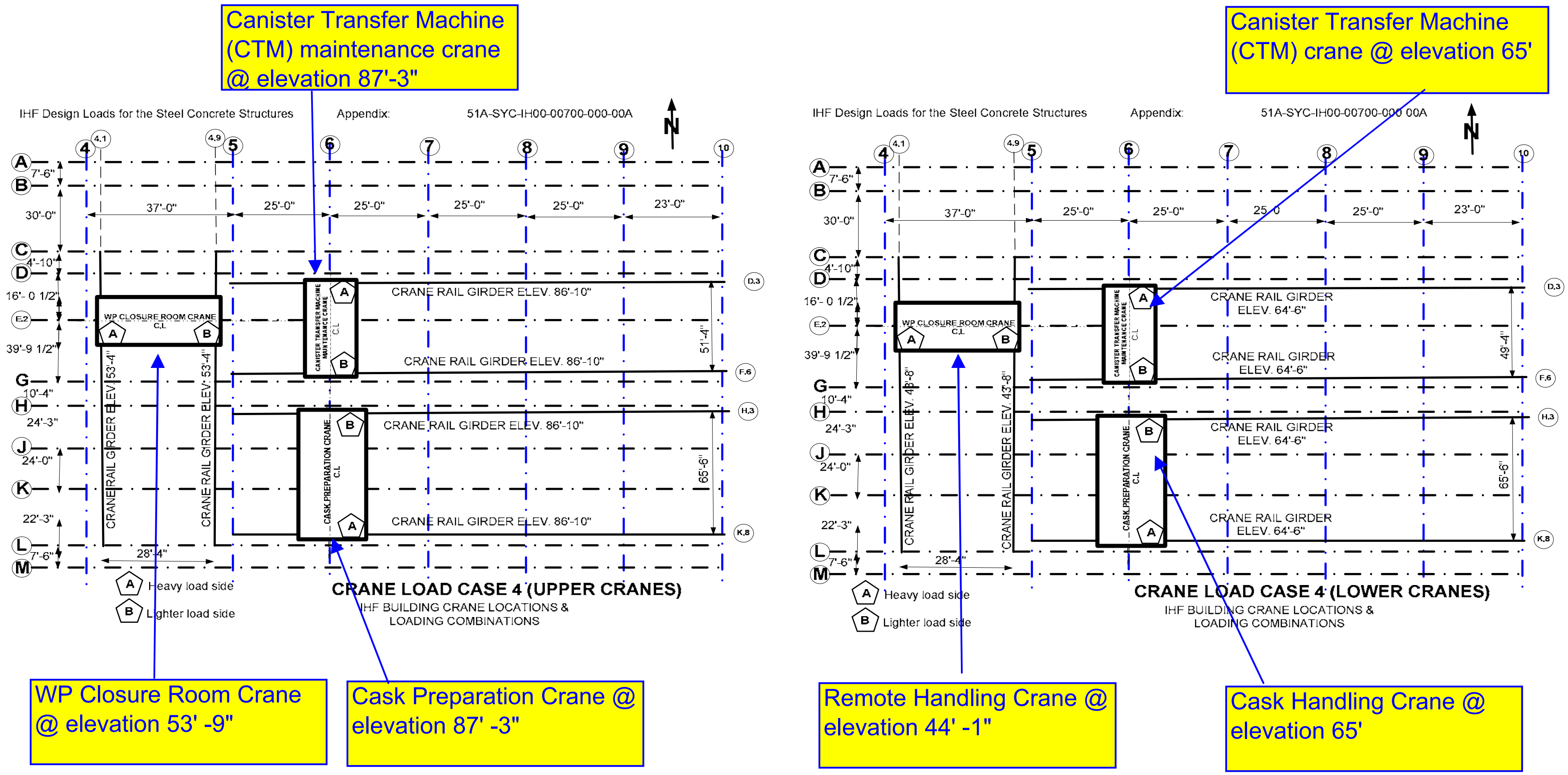


Figure J-4: Crane Load Case 4

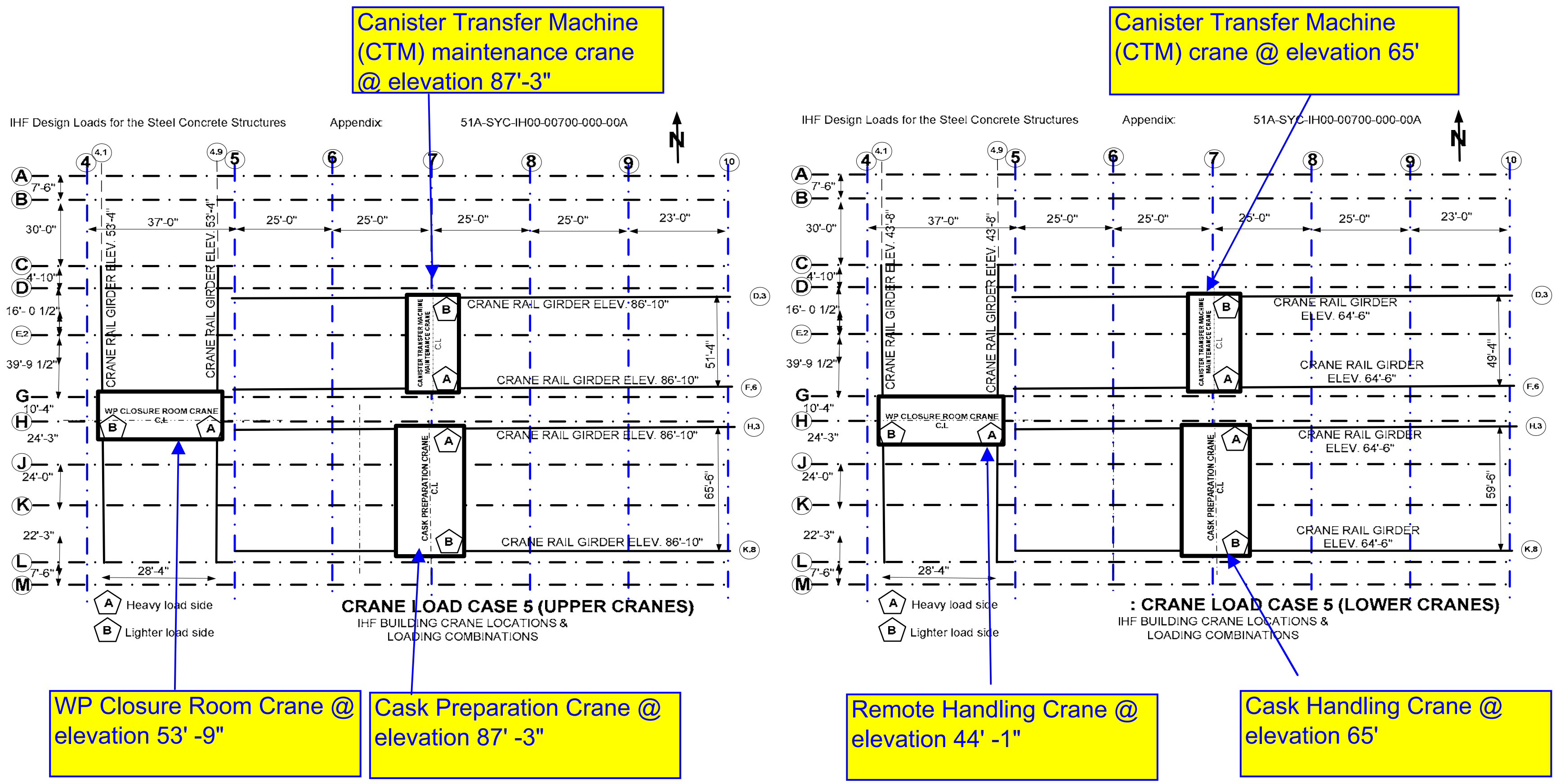


Figure J-5: Crane Load Case 5

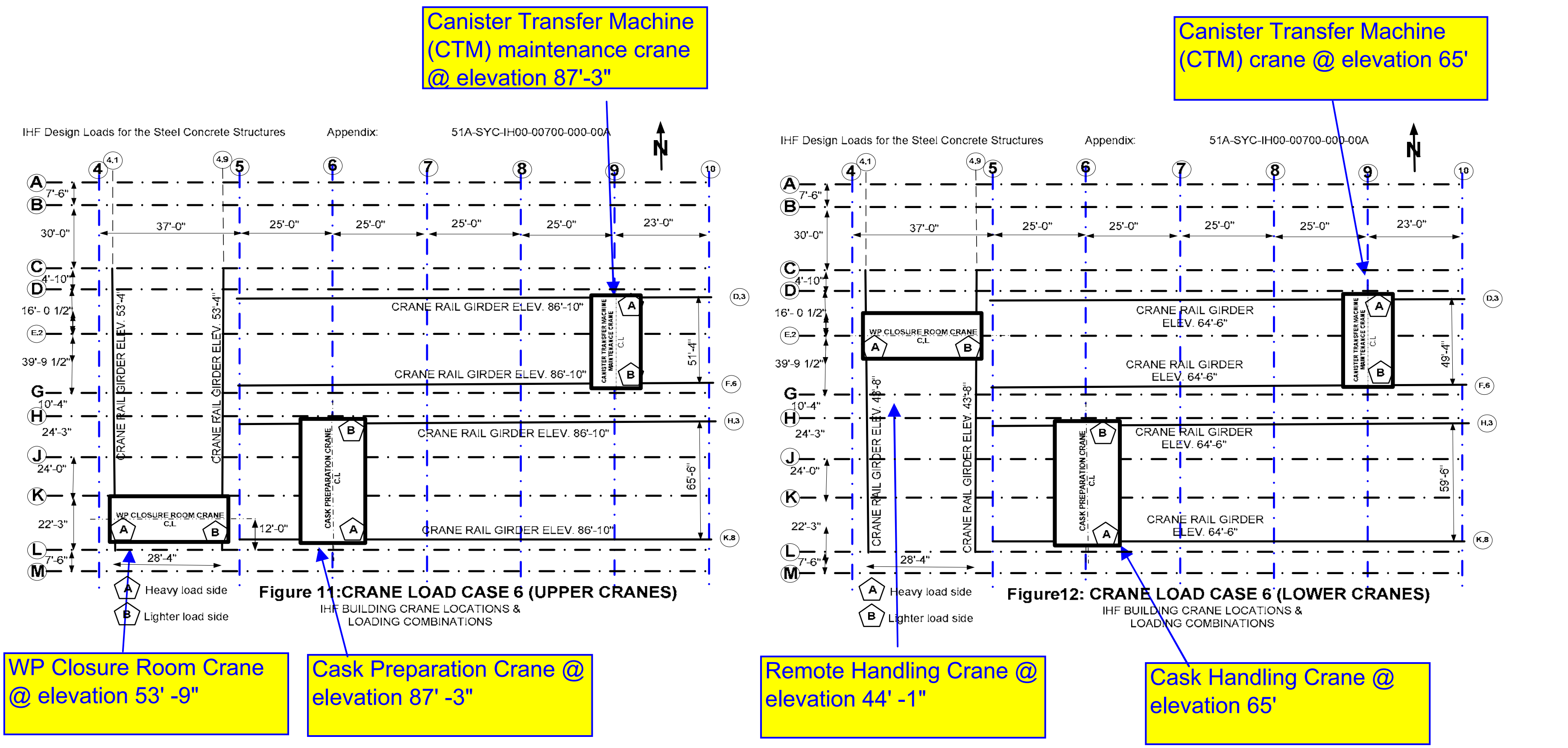


Figure J-6: Crane Load Case 6

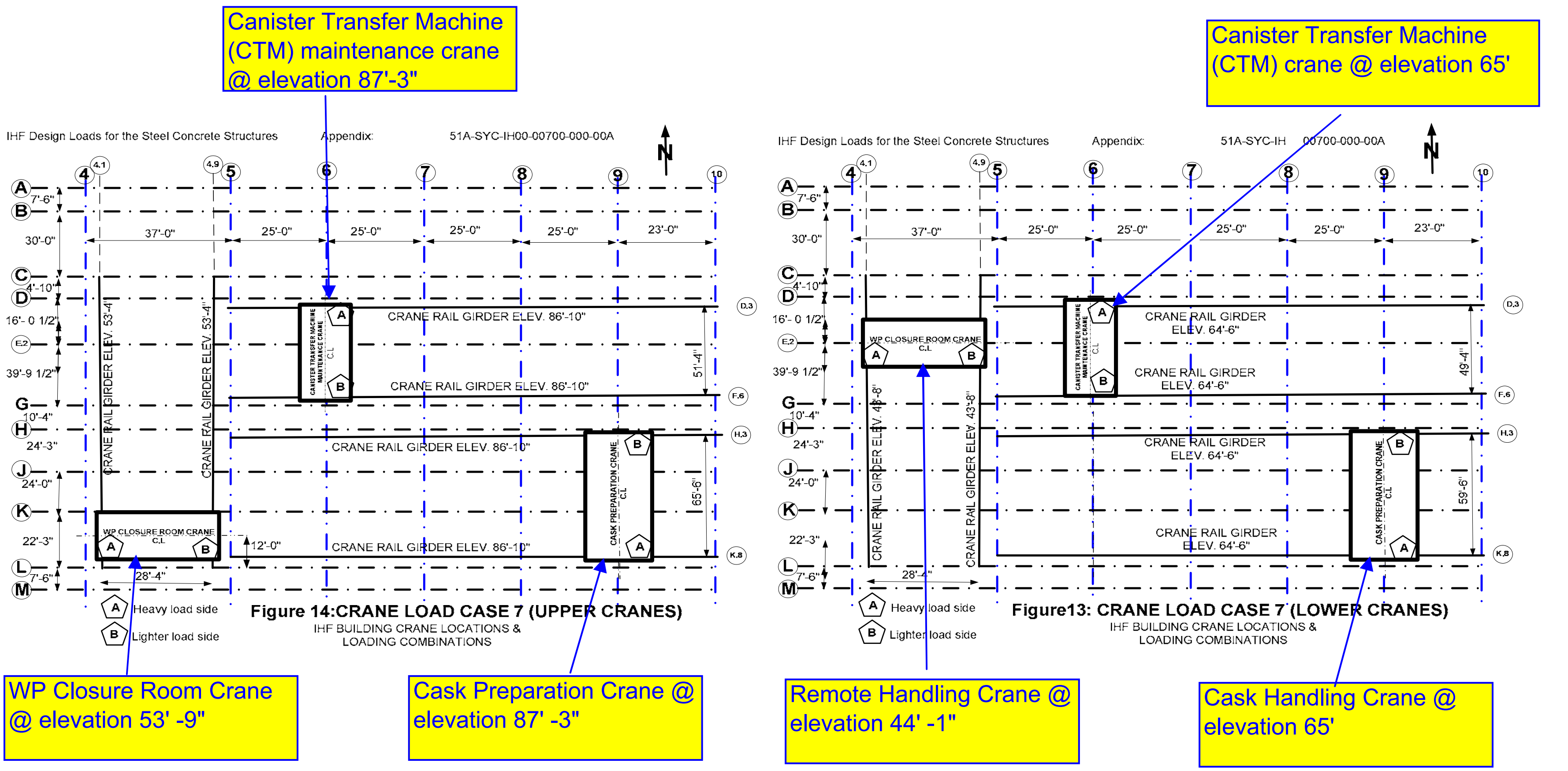


Figure J-7: Crane Load Case 7

Attachment K

Coupled vs. Uncoupled Analysis Study

Utilizing NOG-1 Boundary Conditions

(Refer to the DVD for source output data)

K.1 Introduction:

ASCE 4-98 Section 3.1.7.1 (a) (Ref. 2.2.7) states that a coupled analysis of a primary structure and secondary system shall be performed when the effects of interaction are significant based on the criteria of mass and frequency ratios. Per Section 3.1.7.1 (b) of ASCE 4-98 (Ref. 2.2.7), coupling of the primary and secondary systems is required if the total mass of the secondary system is greater than 1% of the mass of the supporting primary system. For the IHF steel structure the total weight in the vertical direction is W_{BLDG} = 14623 kips (Attachment K, DVD, Folder: Output Data). The weight of the Cask Crane including the lift load is, W_{CASK} = 1130 kips (Page O-4 of Ref. 2.2.21). The weight of the CTM Crane including the lift load is, W_{CTM} = 940 kips (Page N-5 of Ref. 2.2.21). Then,

$W_{CASK} / W_{BLDG} = 1130 / 14623 = 0.08 > 1\%$
$W_{CTM} / W_{BLDG} = 940 / 14623 = 0.06 > 1\%$

From the mass ratios, it is concluded that coupling is required.

Furthermore, this attachment also includes a study that is conducted to address frequency ratios of primary and secondary systems required by ASCE 4-98 Section 3.1.7.1 (Ref. 2.2.7) for the dynamic coupling criteria. In this study the primary structure corresponds to the IHF steel structure uncoupled building model Load Case 5, and the secondary system corresponds to the crane substructure (crane bridge girders, trolley frame and end trucks).
Two major cranes, the Cask Handling Crane and the CTM Crane, are considered in the application of the coupling criteria. In Tables K1-K4, mass and frequency ratios of these two cranes to the uncoupled IHF steel structure are shown.

Table K1: Mass Ratios

Input motion direction	M_{bldg}	M_{CTM}	M_{Cask}	M_{CTM}/M_{bldg}	M_{Cask}/M_{bldg}
	k-sec^2/ft	k-sec^2/ft	k-sec^2/ft	unitless	unitless
X	350	24.8	16.5	0.07	0.05
Y	286	24.8	16.5	0.09	0.06
Z (4.4 Hz)	3.85	29.2	35.1	7.58	9.12
Z (6.4 Hz)	13.48	29.2	35.1	2.17	2.60
Z(8.8 Hz)	18.29	29.2	35.1	1.60	1.92

Table K2: Weight Ratios

Input motion direction	W_{bldg}	W_{CTM}	W_{Cask}	W_{CTM}/W_{bldg}	W_{Cask}/W_{bldg}
	Kips	kips	kips	unitless	unitless
X	11260	800	530	0.07	0.05
Y	9212	800	530	0.09	0.06
Z (4.4 Hz)	124	940	1130	7.58	9.11
Z (6.4 Hz)	434	940	1130	2.17	2.60
Z(8.8 Hz)	589	940	1130	1.60	1.92

Table K3: Frequency Ratios

Input motion direction	f_{bldg} Hz	f_{CTM} Hz	f_{Cask} Hz	f_{CTM}/f_{bldg} unitless	f_{Cask}/f_{bldg} unitless
X	2.4	3.9	4.1	1.63	1.71
Y	3.6	4.5	6.3	1.25	1.75
Z	4.4	6.9	7.0	1.57	1.59
	6.4	6.9	7.0	1.08	1.09
	8.8	6.9	7.0	0.78	0.80

See Table K8 & K9 for crane frequencies and Table K4 for building frequencies

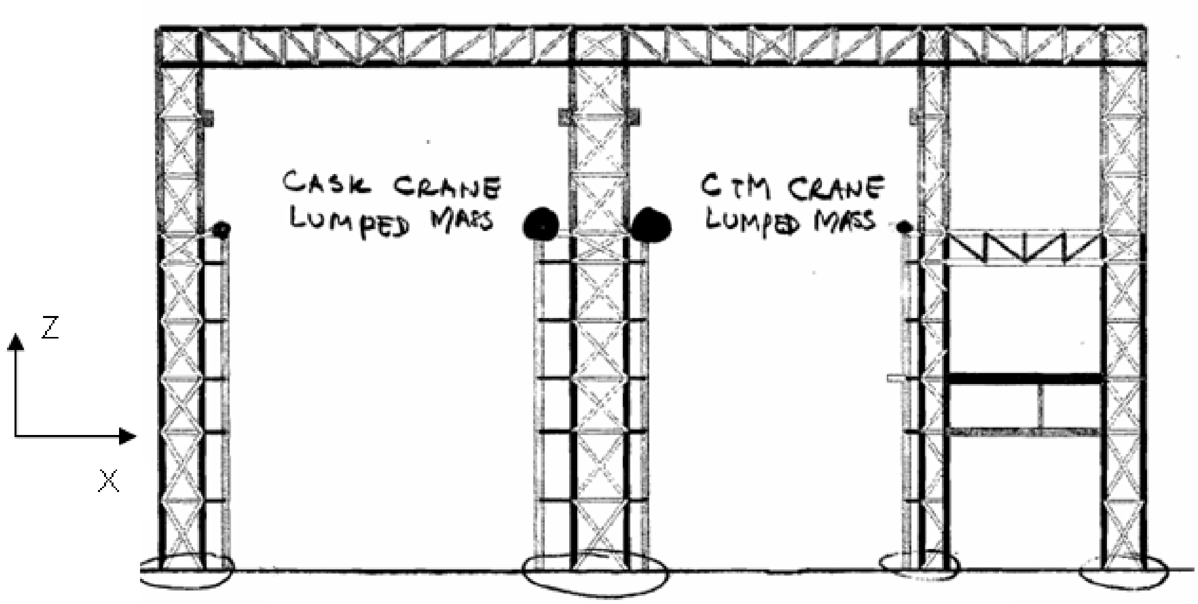
MODAL	Mode	1	0.42	2.4	0.76717	0.00053	0.00000	0.76717	0.00053	0.00000
MODAL	Mode	2	0.28	3.6	0.00102	0.63418	0.00001	0.76819	0.63470	0.00001
MODAL	Mode	3	0.25	3.9	0.00299	0.00247	0.00009	0.77118	0.63717	0.00010
MODAL	Mode	4	0.24	4.1	0.00000	0.00004	0.00209	0.77118	0.63721	0.00219
MODAL	Mode	5	0.23	4.3	0.05787	0.00004	0.00000	0.82905	0.63725	0.00219
MODAL	Mode	6	0.23	4.3	0.00018	0.00000	0.00057	0.82924	0.63725	0.00276
MODAL	Mode	7	0.23	4.4	0.00002	0.00002	0.00846	0.82926	0.63727	0.01122
MODAL	Mode	29	0.16	6.2	0.00001	0.01082	0.00060	0.83490	0.76401	0.05496
MODAL	Mode	30	0.16	6.3	0.00006	0.00820	0.00032	0.83496	0.77221	0.05528
MODAL	Mode	31	0.16	6.4	0.00001	0.00888	0.00011	0.83497	0.78109	0.05540
MODAL	Mode	32	0.16	6.4	0.00014	0.00319	0.02969	0.83511	0.78428	0.08508
MODAL	Mode	33	0.15	6.5	0.00012	0.00074	0.00008	0.83523	0.78502	0.08516
MODAL	Mode	74	0.12	8.6	0.00000	0.00029	0.00179	0.92155	0.81154	0.18348
MODAL	Mode	75	0.11	8.8	0.00000	0.00008	0.00000	0.92155	0.81162	0.18349
MODAL	Mode	76	0.11	8.8	0.00000	0.00000	0.04029	0.92155	0.81162	0.22378
MODAL	Mode	77	0.11	9.0	0.00067	0.01812	0.00001	0.92222	0.82974	0.22379
MODAL	Mode	78	0.11	9.0	0.00700	0.00064	0.00396	0.92922	0.83038	0.22774

Source: Attachment K, DVD, Folder: Output Data

Based on the evaluation of these ratios with respect to Figure 3.1-2 of ASCE 4-98 (Ref. 2.2.7) it is concluded that coupling of IHF steel structure and the crane subsystem is required.

K.2 Uncoupled and Coupled Analysis:

The following pages summarize the uncoupled and coupled analysis through figures and tables.



FIXED BASE UNCOUPLED MODEL LC 5

Figure K 1

Table K5 - Response of Structural Elements (High D/C Ratios)			
Element #	**Member Size**	**Type**	**D/C Ratio**
2009	W36X328	Column	0.67
2006	W36X328	Column	0.56
1718	W8X58	Brace	0.77
1717	W8X58	Brace	0.74
2223	W14X109	Beam	0.43
2258	W12X65	Beam	0.45

Source: Attachment K, DVD, Folder: Output Data

Table K6 - Frequencies and Mass Participation				
Mode #	**Freq. (Hz)**	**X**	**Y**	**Z**
1	2.4	0.77	0.0005	7.0E-11
2	3.6	0.00102	0.63	1.16E-05
7	4.4	1.9E-05	2.0E-05	0.008
32	6.4	1.4E-04	3.0E-04	0.03

Source: Table K4

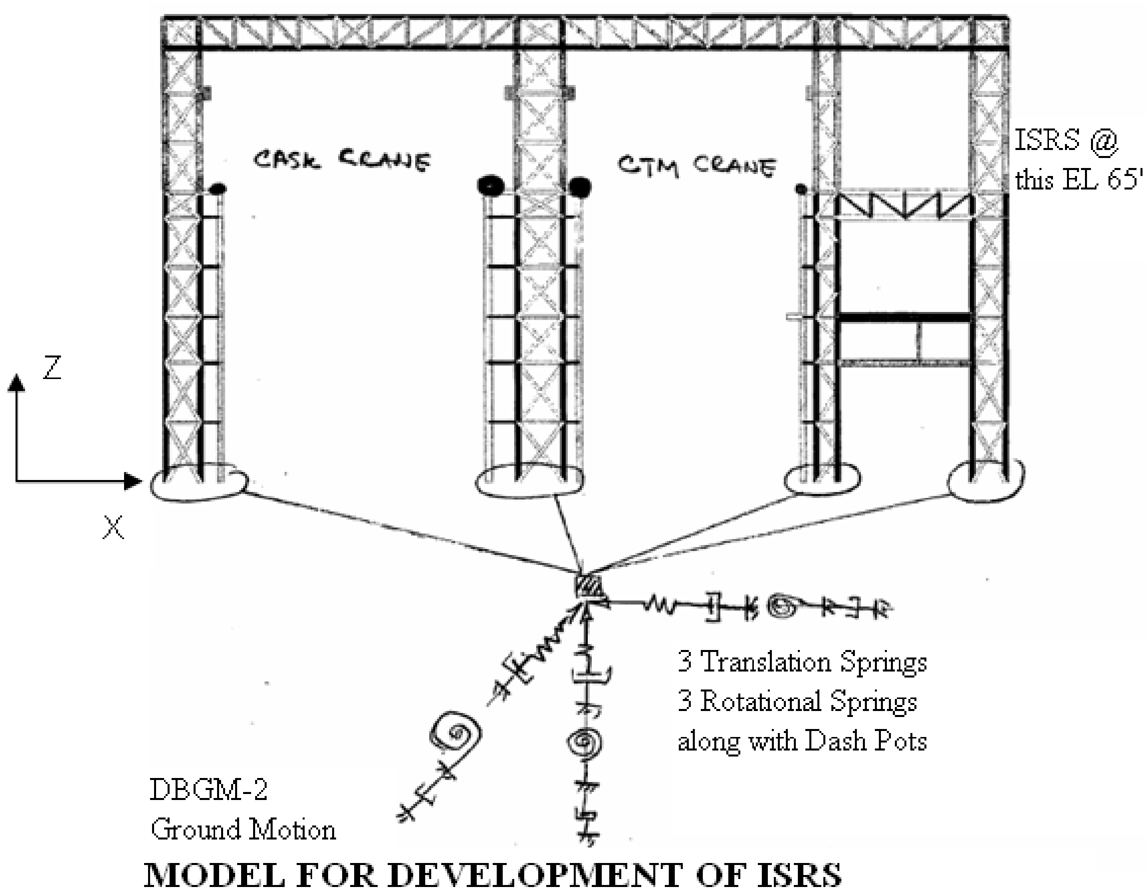


Figure K 2

Table K7 - Frequencies and Mass Participation				
Mode #	**Freq. (Hz)**	**X**	**Y**	**Z**
1	2.5	0.2	1.1E-03	1.67E-08
2	3.8	4.2E-04	0.2	6.75E-06
9	5.7	6.7E-06	9.0E-04	0.015
16	11.7	1.3E-03	5.9E-02	0.24

Source: Attachment K, DVD, Folder: Output Data

The ISRS in Figure K3 through Figure K8 are based on a Tier-1 DBGM-2 uncoupled model from the *Initial Handling Facility (IHF) Tier-1 In-Structure Response Spectra* calculation (Ref. 2.2.22). The response spectra curves are for EL 65'-0" with 4% and 7% damping; the elevation corresponds to the location of the two heavy cranes located in the Initial Handling Facility (IHF). The ISRS curves are provided for the two heavy cranes: Cask Handling crane and Canister Transfer Machine (CTM) crane, in the North-South (X), East-West (Y), and vertical direction. The spectral accelerations attained are in the range of 6 – 9 g's for 7% damping and 9 – 13 g's for 4% damping.

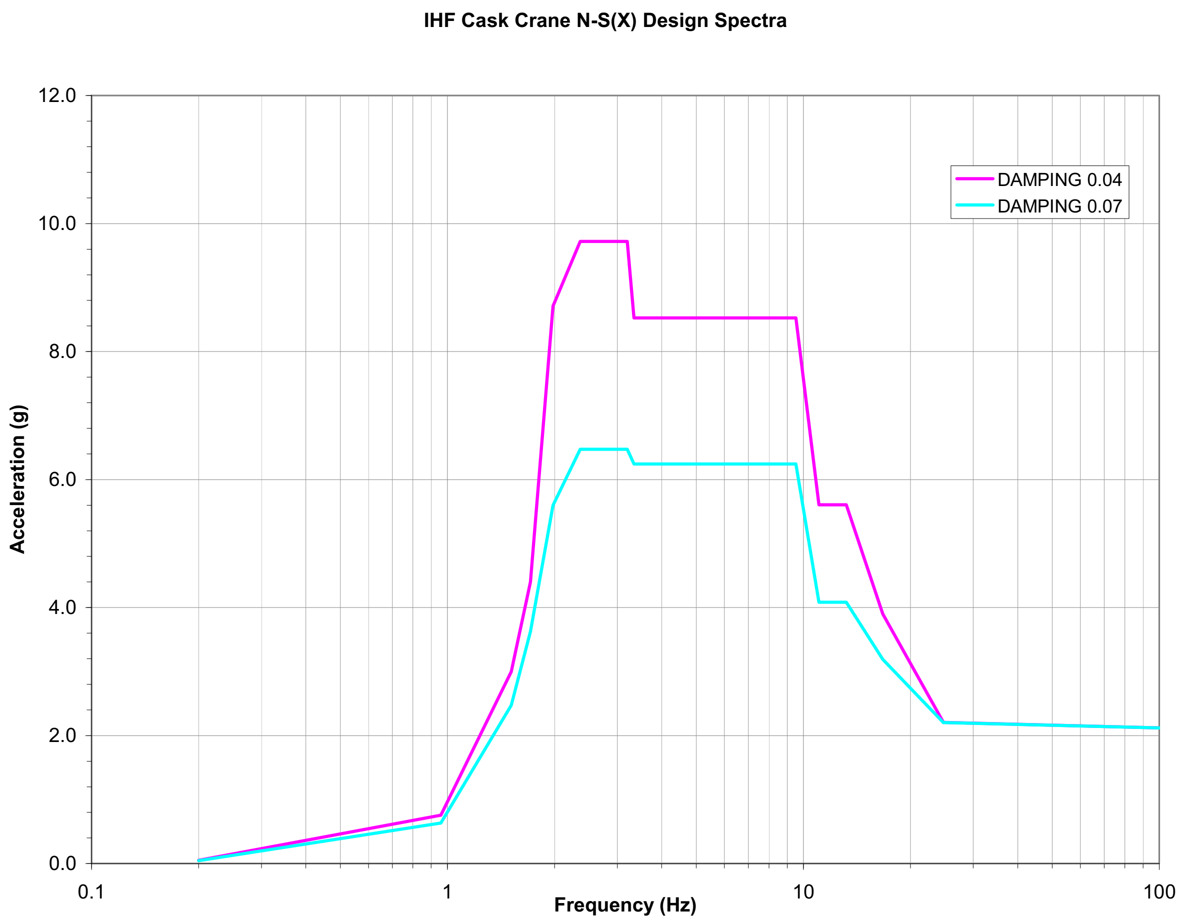


Figure K 3

Source: Ref. 2.2.22, Figure 7.1.1 without dips between peaks

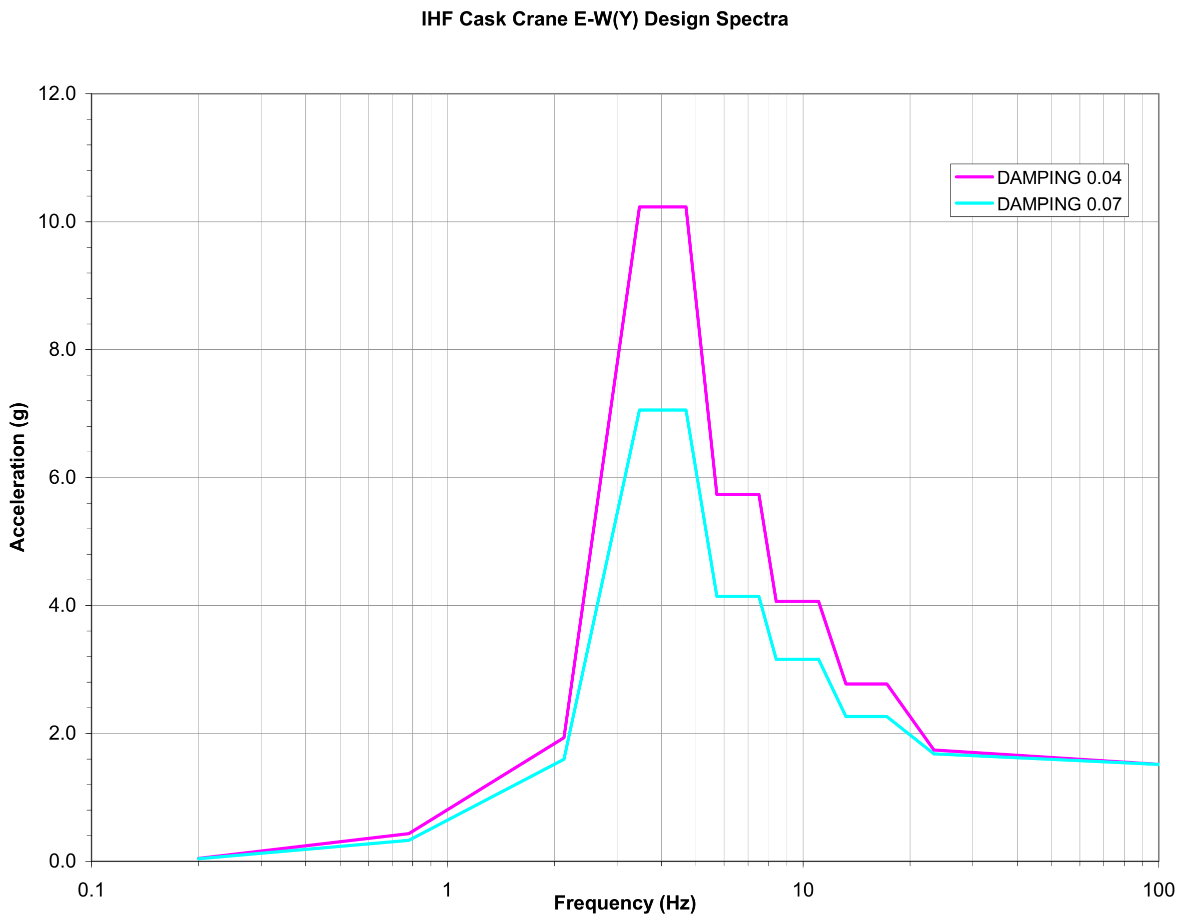


Figure K 4
Source: Ref. 2.2.22, Figure 7.1.2

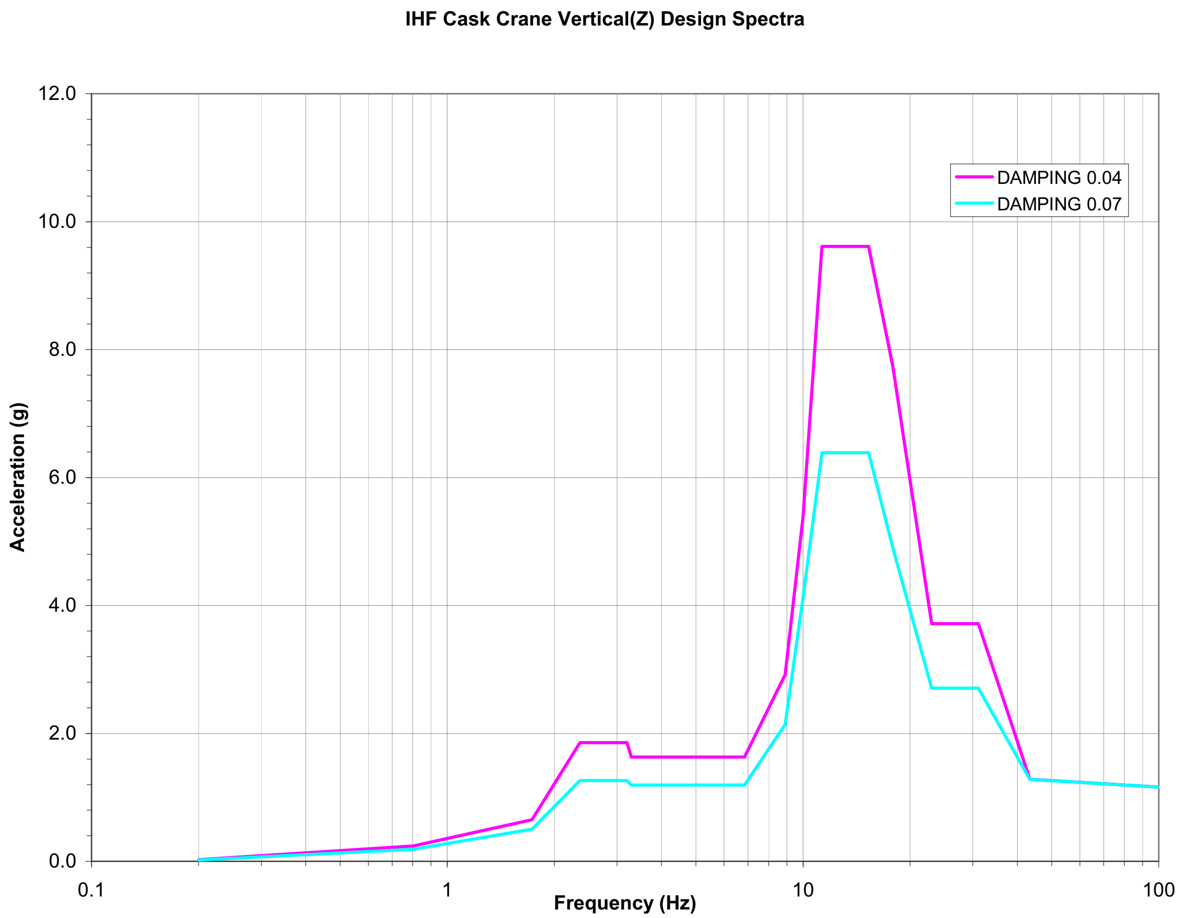


Figure K 5

Source: Ref. 2.2.22, Figure 7.1.3

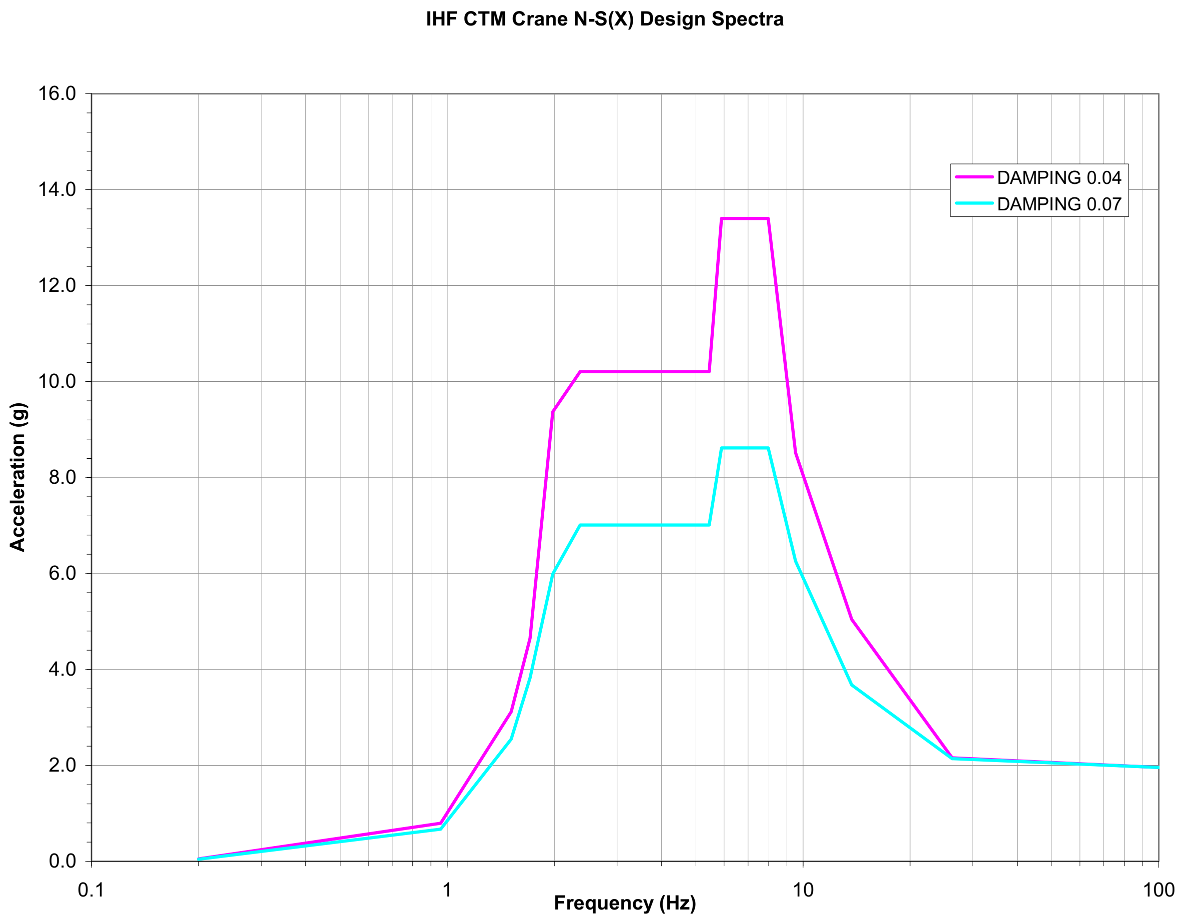


Figure K 6

Source: Ref. 2.2.22, Figure 7.1.4 without dips between peaks

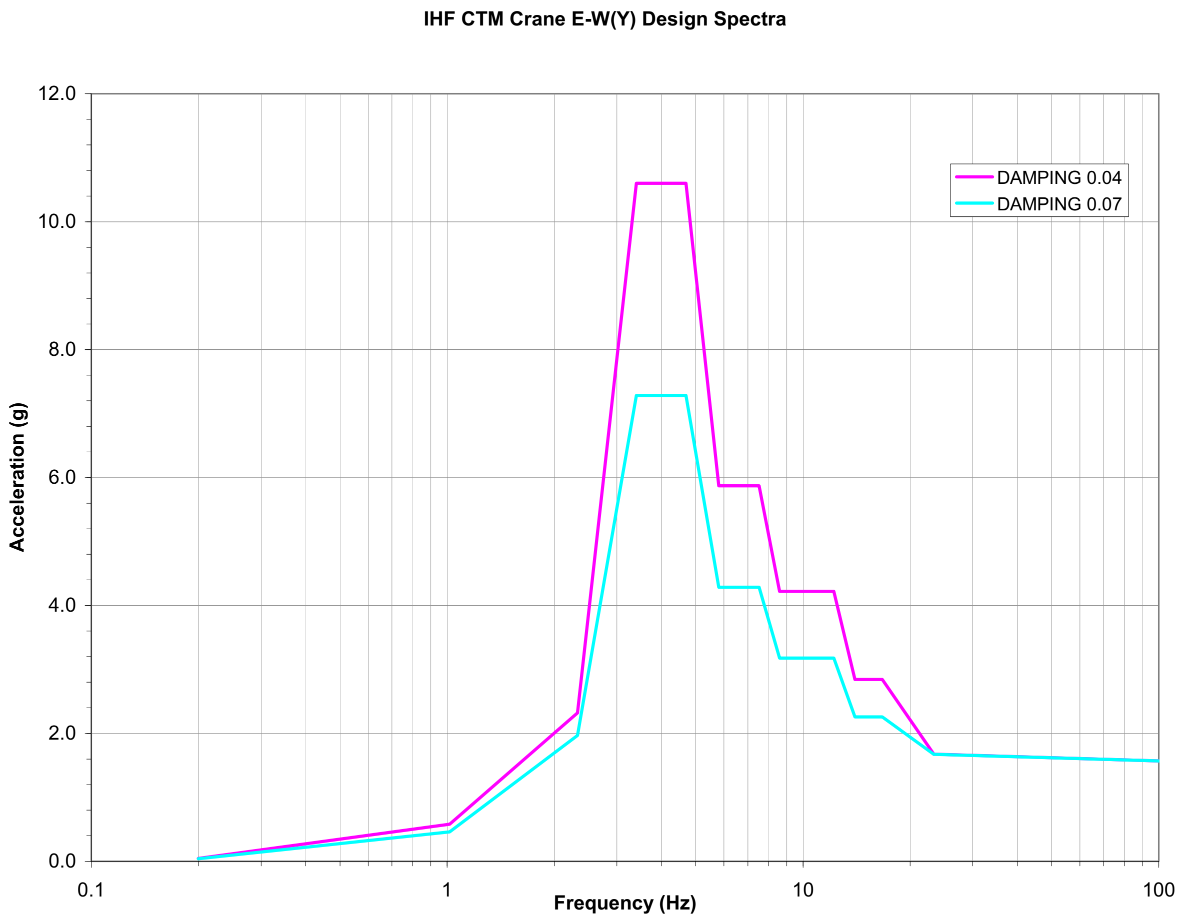


Figure K 7

Source: Ref. 2.2.22, Figure 7.1.5

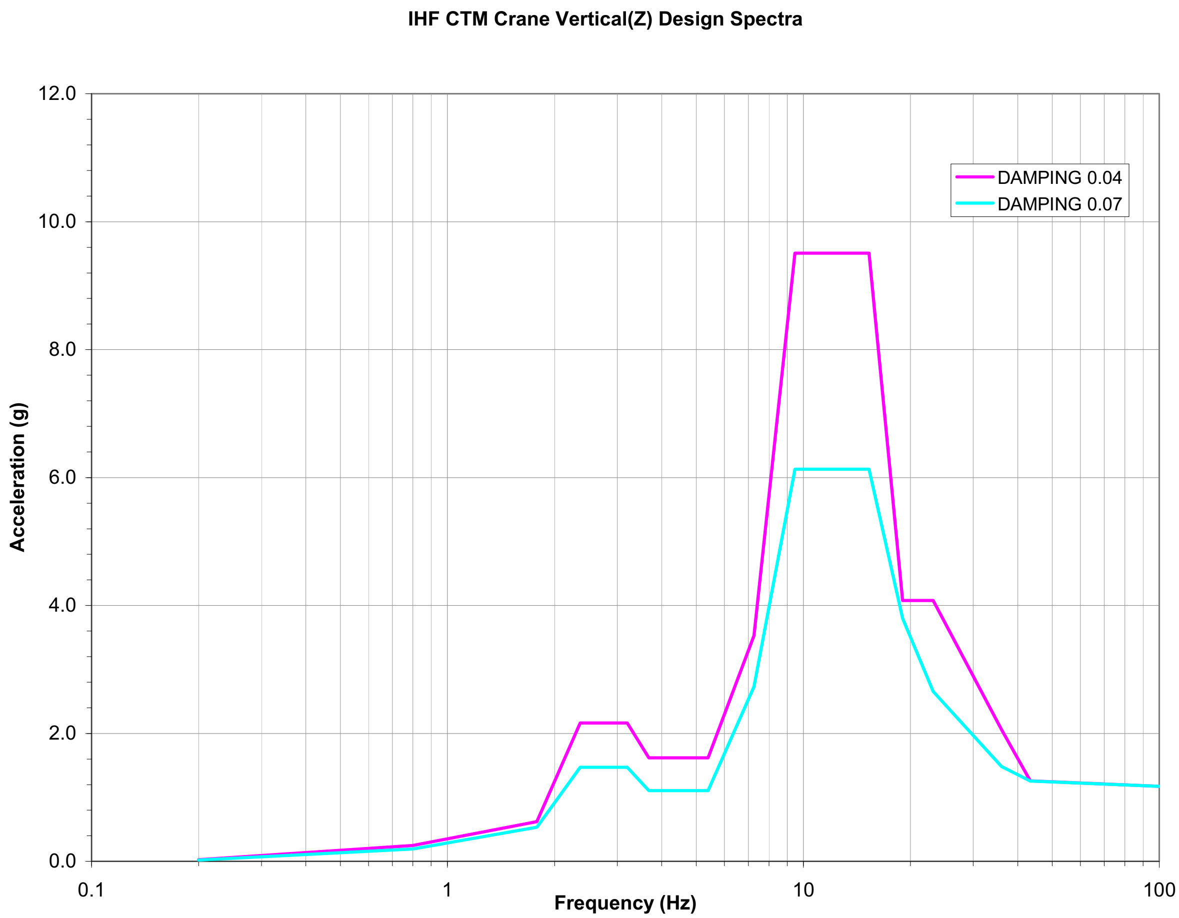


Figure K 8

Source: Ref. 2.2.22, Figure 7.1.6

Uncoupled Typical Crane Model

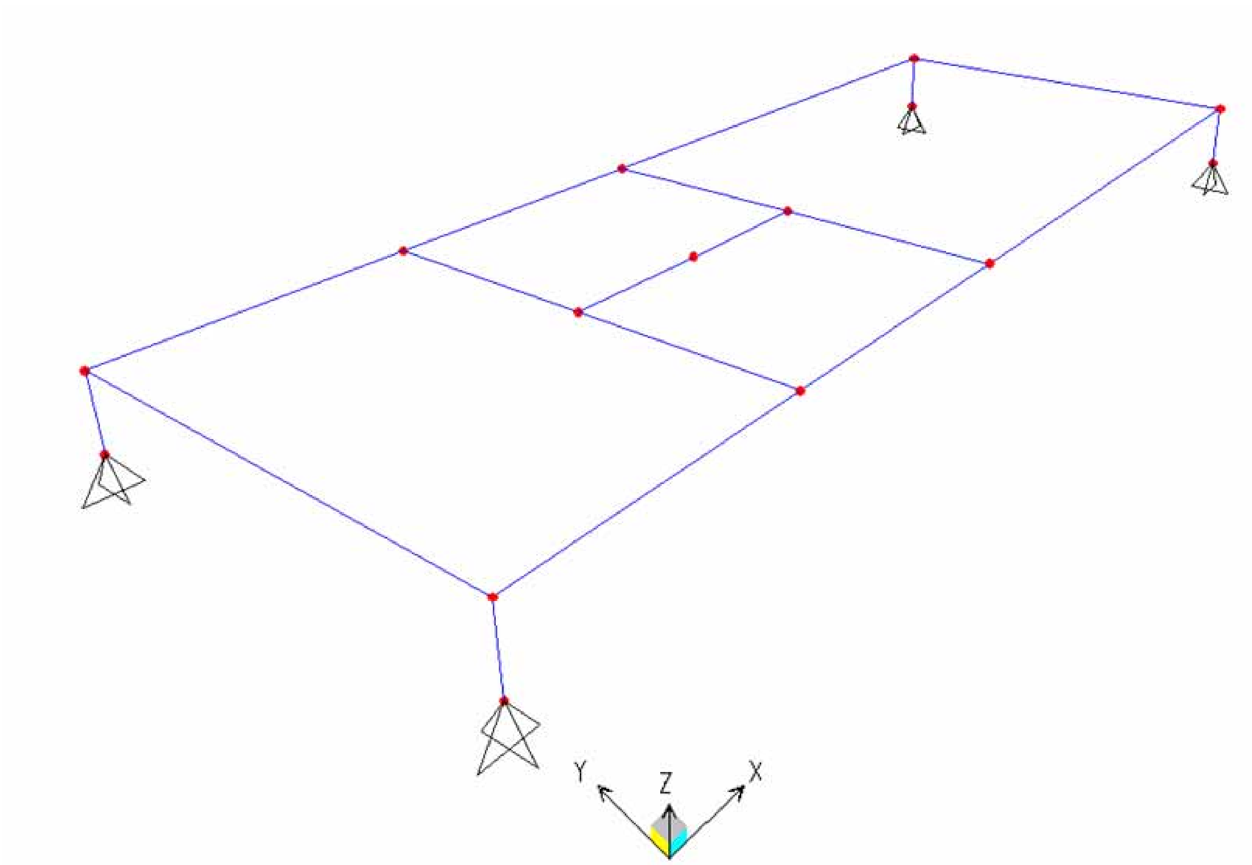


Figure K 9

Table K8 - Uncoupled Cask Handling Crane NOG-1 Modal Analysis Results											
Case	**StepType**	**StepNum**	**Period**	**Frequency**	**UX**	**UY**	**UZ**	**SumUX**	**SumUY**	**SumUZ**
			Sec	Hz	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	0.24	4.1	0.94063	0.00000	0.00038	0.94063	0.00000	0.00038
MODAL	Mode	2	0.16	6.3	0.00000	0.82464	0.00000	0.94063	0.82464	0.00038
MODAL	Mode	3	0.14	7.0	0.00028	0.00000	0.76058	0.94091	0.82464	0.76096
MODAL	Mode	4	0.09	11.4	0.00000	0.00000	0.00286	0.94091	0.82464	0.76383
MODAL	Mode	5	0.08	12.4	0.00000	0.00394	0.00000	0.94091	0.82858	0.76383
MODAL	Mode	6	0.07	14.5	0.05887	0.00000	0.00000	0.99978	0.82858	0.76383
MODAL	Mode	7	0.06	16.6	0.00000	0.01514	0.00000	0.99978	0.84371	0.76383
MODAL	Mode	8	0.04	23.8	0.00000	0.08221	0.00000	0.99978	0.92593	0.76383
MODAL	Mode	9	0.04	25.7	0.00003	0.00000	0.02086	0.99980	0.92593	0.78469
MODAL	Mode	10	0.03	28.9	0.00000	0.07203	0.00000	0.99980	0.99795	0.78469
MODAL	Mode	11	0.03	31.0	0.00000	0.00002	0.00000	0.99980	0.99798	0.78469
MODAL	Mode	12	0.03	34.2	0.00015	0.00000	0.06412	0.99995	0.99798	0.84880
MODAL	Mode	13	0.02	65.9	0.00000	0.00000	0.00000	0.99995	0.99798	0.84880
MODAL	Mode	14	0.01	83.2	0.00000	0.00000	0.00000	0.99995	0.99798	0.84880
MODAL	Mode	15	0.01	88.1	0.00003	0.00000	0.00001	0.99998	0.99798	0.84881
MODAL	Mode	16	0.01	88.3	0.00000	0.00000	0.00000	0.99998	0.99798	0.84881
MODAL	Mode	17	0.01	88.8	0.00001	0.00000	0.00000	0.99999	0.99798	0.84881
MODAL	Mode	18	0.01	92.4	0.00000	0.00137	0.00000	0.99999	0.99934	0.84881

Source: Attachment K, DVD, Folder: Output Data\Cask Crane NOG-1

Table K9 - Uncoupled CTM Crane NOG-1 Modal Analysis Results										
Case	**StepType**	**StepNum**	**Period**	**Frequency**	**UX**	**UY**	**UZ**	**SumUX**	**SumUY**	**SumUZ**
			Sec	Hz	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	0.26	3.9	0.99191	0.00000	0.00111	0.99191	0.00000	0.00111
MODAL	Mode	2	0.22	4.5	0.00000	0.94208	0.00000	0.99191	0.94208	0.00111
MODAL	Mode	3	0.15	6.9	0.00112	0.00000	0.89874	0.99303	0.94208	0.89985
MODAL	Mode	4	0.07	14.4	0.00000	0.00000	0.00152	0.99303	0.94208	0.90137
MODAL	Mode	5	0.06	17.1	0.00000	0.00068	0.00000	0.99303	0.94276	0.90137
MODAL	Mode	6	0.05	19.4	0.00000	0.00063	0.00000	0.99303	0.94339	0.90137
MODAL	Mode	7	0.05	22.1	0.00000	0.01122	0.00000	0.99303	0.95461	0.90137
MODAL	Mode	8	0.04	23.6	0.00337	0.00000	0.00798	0.99641	0.95461	0.90935
MODAL	Mode	9	0.04	23.9	0.00356	0.00000	0.01027	0.99996	0.95461	0.91962
MODAL	Mode	10	0.03	34.9	0.00000	0.00001	0.00000	0.99996	0.95462	0.91962
MODAL	Mode	11	0.02	41.3	0.00000	0.03681	0.00000	0.99996	0.99143	0.91962
MODAL	Mode	12	0.02	48.7	0.00002	0.00000	0.01645	0.99998	0.99143	0.93607
MODAL	Mode	13	0.02	55.9	0.00000	0.00000	0.00000	0.99998	0.99143	0.93607
MODAL	Mode	14	0.02	61.1	0.00000	0.00798	0.00000	0.99998	0.99941	0.93607
MODAL	Mode	15	0.01	74.6	0.00000	0.00000	0.00000	0.99998	0.99941	0.93607

Source: Attachment K, DVD, Folder: Output Data\CTM Crane NOG-1

Interface Reactions from Uncoupled Cask Handling Crane Using NOG-1 Boundary Conditions

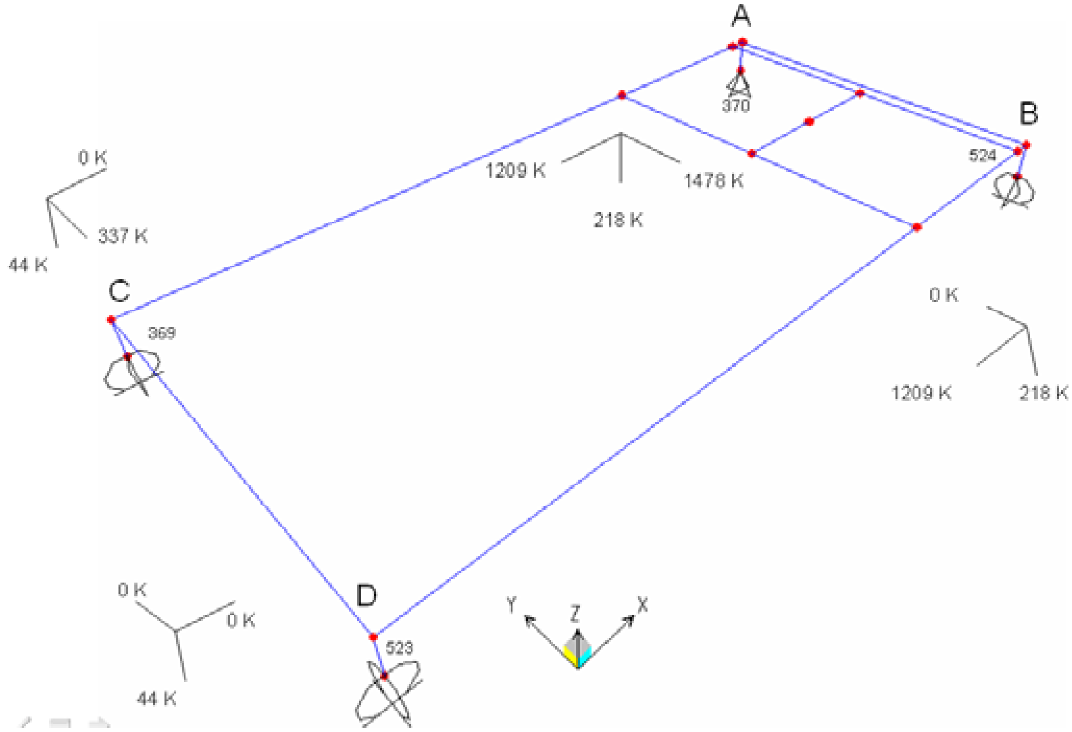


Figure K 10

Source: Attachment K, DVD, Folder: Output Data\Cask Crane NOG-1

Interface Reactions from Uncoupled CTM Crane Using NOG-1 Boundary Conditions

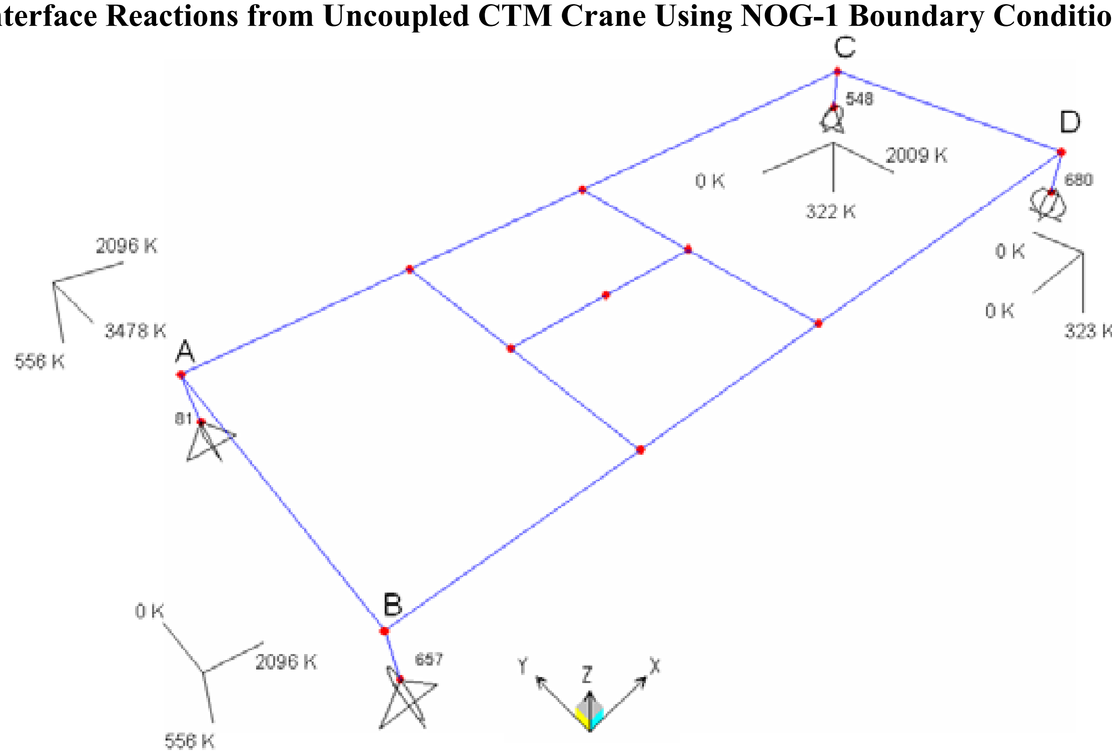


Figure K 11

Source: Attachment K, DVD, Folder: Output Data\CTM Crane NOG-1

Table K10 - Uncoupled CTM Crane Stresses NOG-1					
Frame	**Section**	**Type**	**ffa**	**ffbMajor**	**ffbMinor**
			Kip/in2	Kip/in2	Kip/in2
4310	CTM1	Beam	5.85	10.45	61.90
4314	CTM2	Beam	10.99	25.19	83.04
4397	CTM2	Beam	8.77	18.81	83.25

Table K11 - Uncoupled Cask Handling Crane Stresses NOG-1					
Frame	**Section**	**Type**	**ffa**	**ffbMajor**	**ffbMinor**
			Kip/in2	Kip/in2	Kip/in2
4401	CASK1	Beam	0.64	3.88	18.43
4497	CASK2	Beam	3.51	12.02	70.86
4507	CASK2	Beam	4.65	7.05	31.92

Source of Table K10 & K11: Attachment K, DVD, Folder: Output Data, "corresponding crane folder"

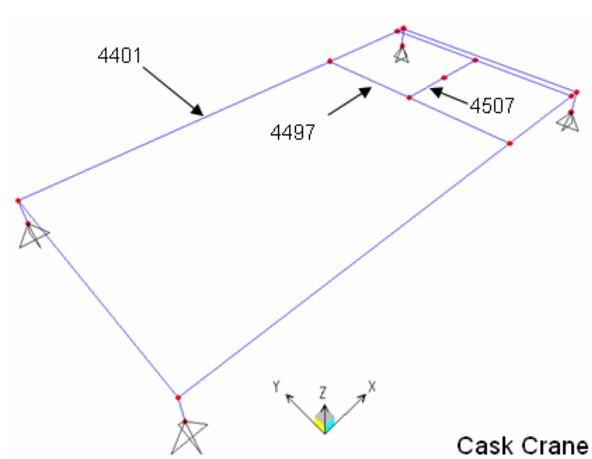


Figure K 12

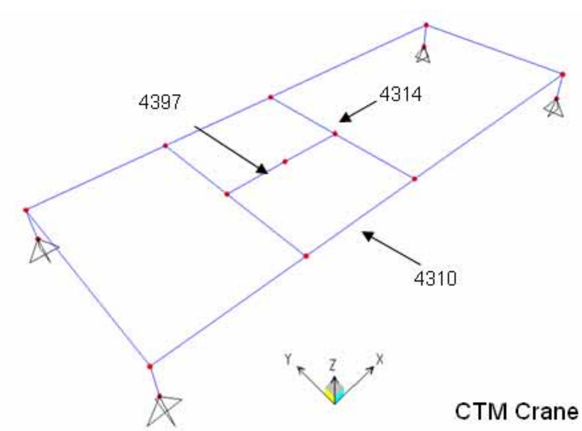


Figure K 13

Coupled Building Model at EL 65'-0"

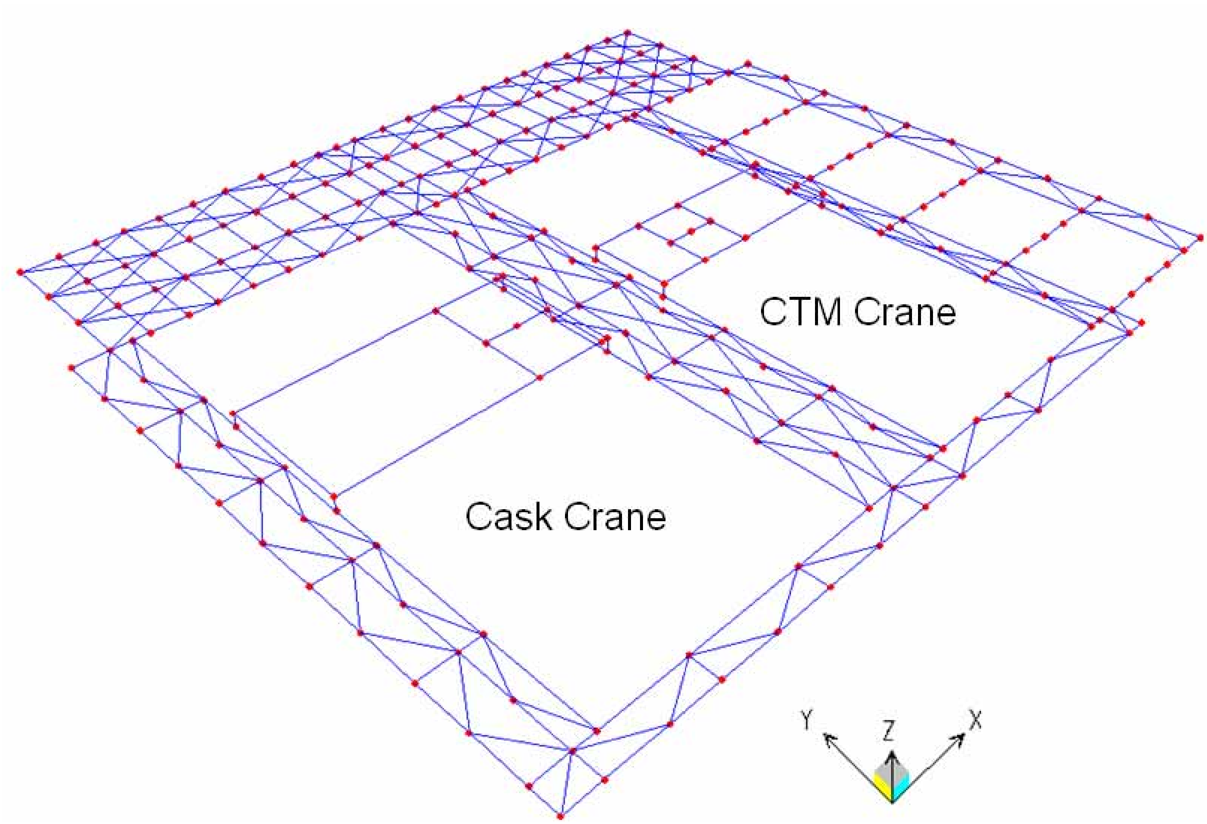


Figure K 14

Fixed Base Coupled Building Model LC 5 Using NOG-1 Boundary Conditions

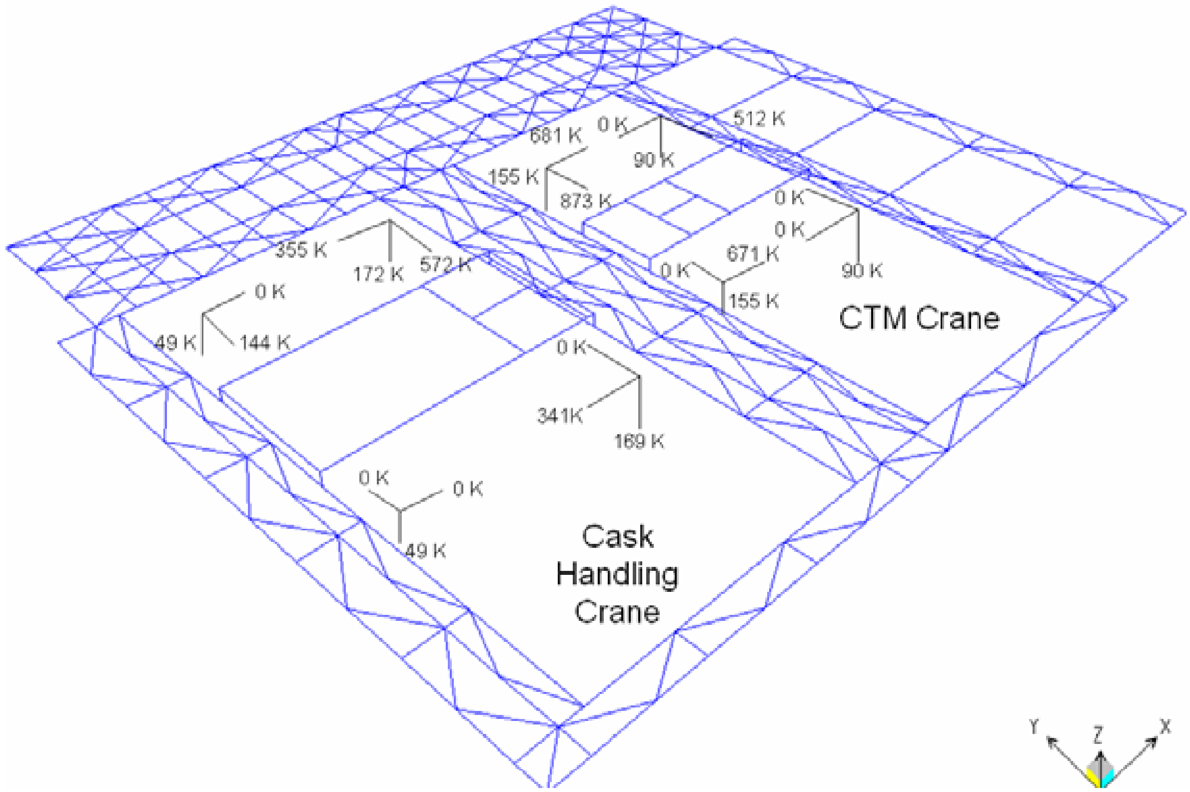


Figure K 15

Source: Attachment K, DVD, Folder: Output Data\LC 5 Coupled Model NOG-1

The stresses in the crane components of the Canister Transfer Machine (CTM) crane and the Cask Handling crane are within manageable levels, as noted in Table K12 and K13. Axial and bending stresses are tabulated in Table K12 and Table K13 for the crane components in the steel portion of the Initial Handling Facility at EL 65'-0". The stress values are attained for typical crane section in the two cranes for Crane Load Case 5 with NOG-1 boundary conditions. Figure K16 illustrates an isometric view of the two cranes, which are modeled in SAP2000, as well as the location of the stresses tabulated in Table K12 and Table K13.

The stresses are based on a response spectrum analysis using results from a DBGM-2 modal analysis and 7% structural damping. The analysis combines the modal responses by the 10% method and combines the North-South (X), East-West (Y), and Vertical (Z) direction spectral cases by the square root of sum of the squares (SRSS) method.

Table K12 - CTM Crane Load Case 5

Frame	Section	Type	ffa	ffbMajor	ffbMinor
			Kip/in2	Kip/in2	Kip/in2
4310	CTM1	Beam	1.75	2.84	15.54
4314	CTM2	Beam	2.71	6.61	26.35
4397	CTM2	Beam	2.78	5.76	20.93

Table K13 - Cask Handling Crane Load Case 5

Frame	Section	Type	ffa	ffbMajor	ffbMinor
			Kip/in2	Kip/in2	Kip/in2
4401	CASK1	Beam	0.29	1.87	6.90
4497	CASK2	Beam	1.25	9.17	18.98
4507	CASK2	Beam	1.25	8.22	12.12

Source: Attachment K, DVD, Folder: Output Data\LC 5 Coupled Model NOG-1

Isometric View at EL 65'-0"

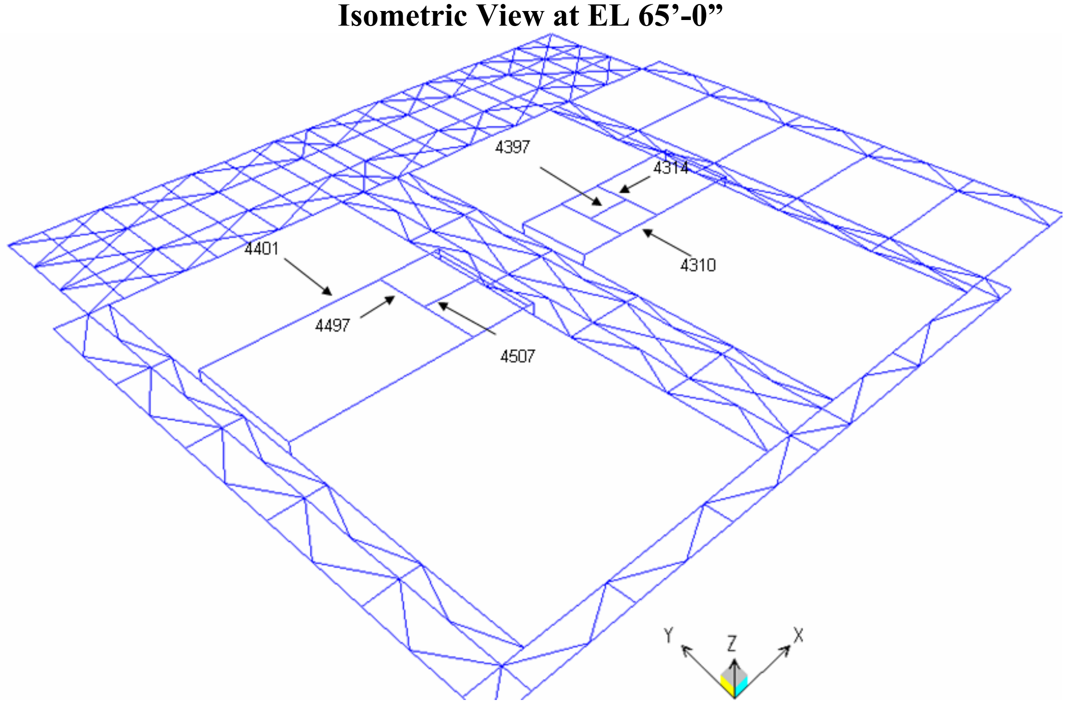


Figure K 16

K.3 Summary and Conclusion:

Table K14: Comparison of CTM Crane Uncoupled and Coupled Interface Reactions

Node	Force	NOG-1	
		Uncoupled	Coupled
		[kips]	[kips]
A	**Fx**	2096	681
A	**Fy**	3478	873
A	**Fz**	556	155
B	**Fx**	2096	671
B	**Fy**	0	0
B	**Fz**	556	155
C	**Fx**	0	0
C	**Fy**	2009	512
C	**Fz**	322	90
D	**Fx**	0	0
D	**Fy**	0	0
D	**Fz**	323	90

Source: Figure K11 and Figure K15

Table K15: Comparison of Cask Handling Crane Uncoupled and Coupled Interface Reactions

Node	Force	NOG-1	
		Uncoupled	Coupled
		[kips]	[kips]
A	**Fx**	1209	355
A	**Fy**	1478	572
A	**Fz**	218	172
B	**Fx**	1209	341
B	**Fy**	0	0
B	**Fz**	218	169
C	**Fx**	0	0
C	**Fy**	337	144
C	**Fz**	44	49
D	**Fx**	0	0
D	**Fy**	0	0
D	**Fz**	44	49

Source: Figure K10 and Figure K15

From the comparison results in Table K14 and Table K15 it is noted that the interface reactions from the coupled approach is significantly less than the uncoupled approach. Hence, it is concluded that the coupled model using NOG-1 boundary conditions is advantageous.