1043-01

FLAC3D Stability Analysis of S3 Submains and E5 Panel, Cayuga Mine

Prepared for:

Cargill, Inc.

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1 INTRODUCTION

Cargill, Inc. (Cargill) mines salt from the No. 6 Salt bed at its Cayuga Mine, a conventional underground mine near Lansing, New York. Cargill would like to store water in the S3 Submain and its adjacent panels. Once placed, the water will not be withdrawn (no circulation). The panels consist of small pillars designed to slowly yield over time. Other portions of the mine are and will remain active, and Cargill is interested in assessing the global stability in the flooded area, which if compromised, could potentially cause a sudden flood of the stored water into the active workings.

Cargill retained Agapito Associates, Inc. (Agapito) to assess the effect of the planned flooding on the global stability of the subject panels. Previously, Cargill commissioned a study of panel flooding by RESPEC of Rapid City, South Dakota. While Cargill did not provide the resulting RESPEC report to Agapito, Cargill specified input parameters assumed in the previous study so that the results of the Agapito and RESPEC work could be directly compared by Cargill. Working with Dr. Samrat Mohanty of Cargill, it was agreed that the E5 Panel and the immediately adjacent portion of the S3 Submains would be analyzed, as the E5 Panel represents one of the larger panels inby, with a relatively high flooded water head. The immediate roof over the panel is a claystone that is somewhat water sensitive; therefore, it is expected that the roof will tend to deteriorate over the entries in time after the panels are flooded. Agapito proposed and Cargill approved a criterion for roof deterioration based on the stress state and flexural strength of the claystone, allowing for likely effects of roof deterioration on pillar and panel stability to be included in the analysis.

The analysis includes creep simulation of the salt layers in the overburden and the No. 6 Salt bed. A total of 50 years of creep was included. The E5 Panel was developed approximately 12 years ago, and Cargill estimates that it will take 8 additional years for the water stored in the S3 Submains to reach the roof of the E5 Panel. At this point, flooding will continue updip for 10 years, with the head at the E5 roof increasing from 0 pounds per square inch (psi) to 20 psi in that time (the panels are isolated hydraulically from the overlying strata and Cayuga Lake). Therefore, the analysis has three periods of creep. Years 0 to 20 simulate dry conditions, representing the time from initial mining to the time flooding reaches the E5 roof. Years 20 to 30 simulate active flooding, with water head increasing from 0 to 20 psi in 2-psi annual increments, and years 30 to 50 simulate steady-state flooding with a constant water head of 20 psi. For consistency with RESPEC's results, which are usually referenced to time after flooding, 20 years can be subtracted from the Agapito model year to arrive at the time after flooding.

The analysis was performed using FLAC3D (Itasca 2013), a three-dimensional (3D) finite-difference method.

2 MODEL GEOMETRY AND INPUTS

The geometry of the S3 area provided by Cargill, and the E5 Panel included in the model are shown in Figure 1. Consistent with RESPEC's assumptions, pillars in the E5 Panel were modeled at [redacted] feet (ft) by [redacted] ft, and pillars in the S3 Submains were [redacted] ft by [redacted] ft. Cargill intends that the disposed water will be an almost saturated brine, so potential dissolution of the pillars is

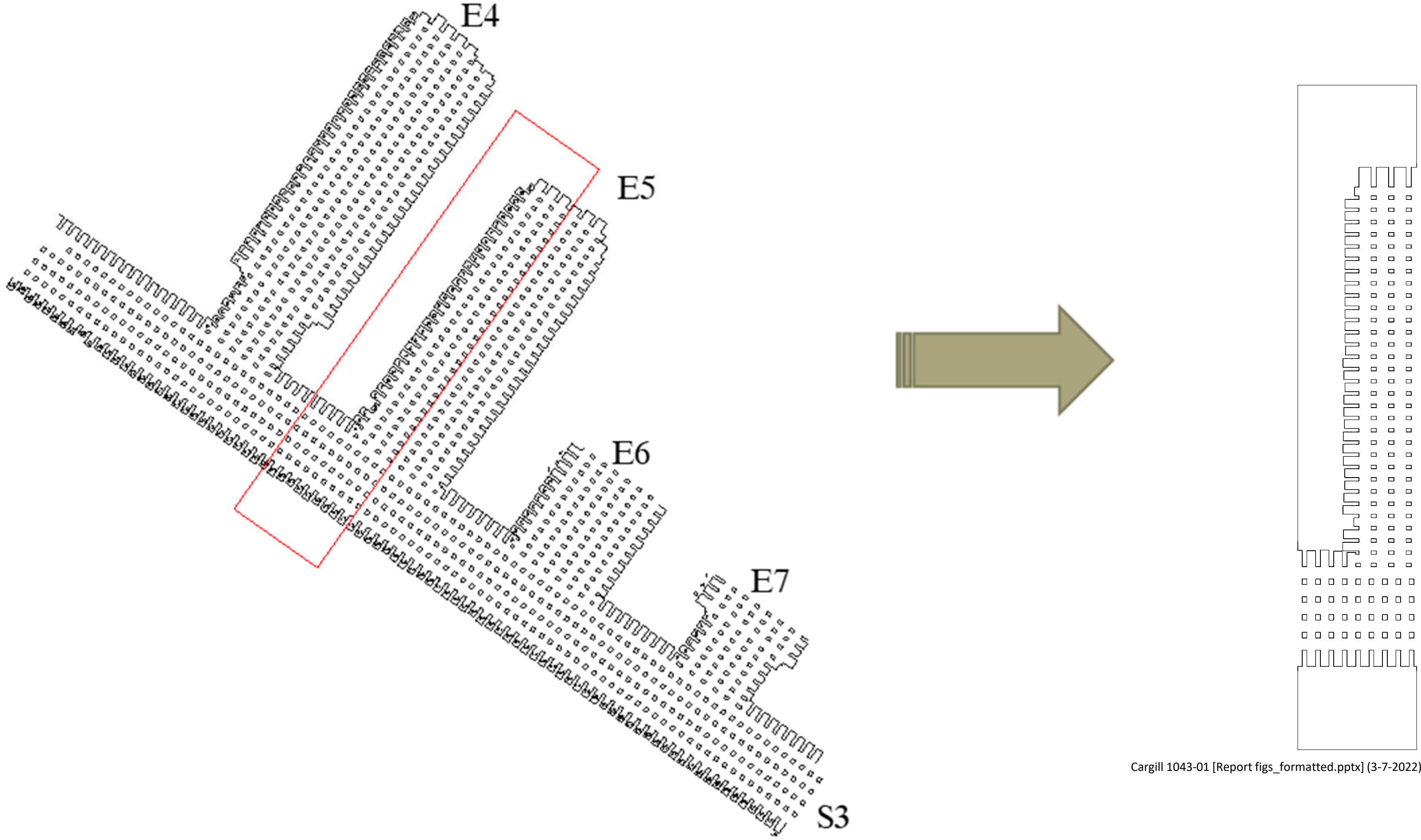

Figure 1. Panels Off the S3 Submain (left) **and Plan View of Area Included in the Model** (right)

not thought to be significant; the modeled pillars allow for approximately 6 inches of dissolution around the pillar perimeter. With an entry width of [redacted] ft and a crosscut width of [redacted] ft, the panel centers are [redacted] ft by [redacted] ft, and the submains are [redacted] ft by [redacted] ft. The panel is basically symmetrical about its centerline, so the centerline was used as a line of symmetry to improve computational efficiency. Elements were one-foot cubes in the No. 6 Salt bed and overlying claystone, with increasing coarseness distant from this zone of detail.

The model extends 1,000 ft above the No. 6 Salt mining horizon, and about 620 ft below it. The model length (parallel to the E5 Panel) is [redacted] ft, and the width is [redacted] ft. The E5 Panel is situated beneath Cayuga Lake, and a surcharge load corresponding to the weight of the lake water, sediments, shale, and a portion of the carbonates was applied to the top of the model (–882.8 ft above mean seal level, AMSL), corresponding to vertical stress of 1,317 psi. The pre-mining vertical stress from overburden loading at the top of the No. 6 Salt is 2,490 psi. A view of the model and the layers included is shown in Figure 2. Even though there is a mild gradient along the submains entries, the modeled strata were assumed to be horizontal for simplicity and computational efficiency.

Lithology and unit properties were provided by Cargill to match those used by RESPEC. Small adjustments were made in consultation with Dr. Mohanty, particularly regarding tensile strengths of the modeled strata units. A summary of the model lithology, thickness, and input properties is given in Table 1.

The unit weights shown in Table 1 were provided by Cargill. The unit weights of non-salt units reflect a 19% increase over the measured unit weights to replicate the vertical stress gradient thought to exist at the mine. Horizontal stresses parallel to the panel centerline were [redacted] times the vertical stress, and horizontal stresses perpendicular to the panel centerline were [redacted] times the vertical stress.

2.1 Material Models

As shown in Table 1, the various non-salt lithologies associated with the S3 Area were modeled either as surcharge loading on the top of the model, elastic materials, or plastic materials following the Mohr-Coulomb failure criterion. Lithologies high in the overburden, or in the main floor were assigned elastic properties, meaning they can accept unlimited load without failing. The relatively distant positions of these layers make them less important with respect to interaction with the salt layers and the stability of the mine workings, and assigning elastic properties to such layers helps to speed run times. Plastic materials were used for non-salt layers interspersed with the salt layers. Plastic materials behave elastically until they reach failure; afterwards they continue to deform without shedding load or taking additional load.

The salt layers were modeled using the Norton power law formulation (Norton 1929) available in FLAC3D that is commonly used to model salt creep behavior. The standard form of the power law is:

$$\dot{\epsilon}_{cr} = A\, \bar{\sigma}^n \qquad \text{(Eqn. 1)}$$

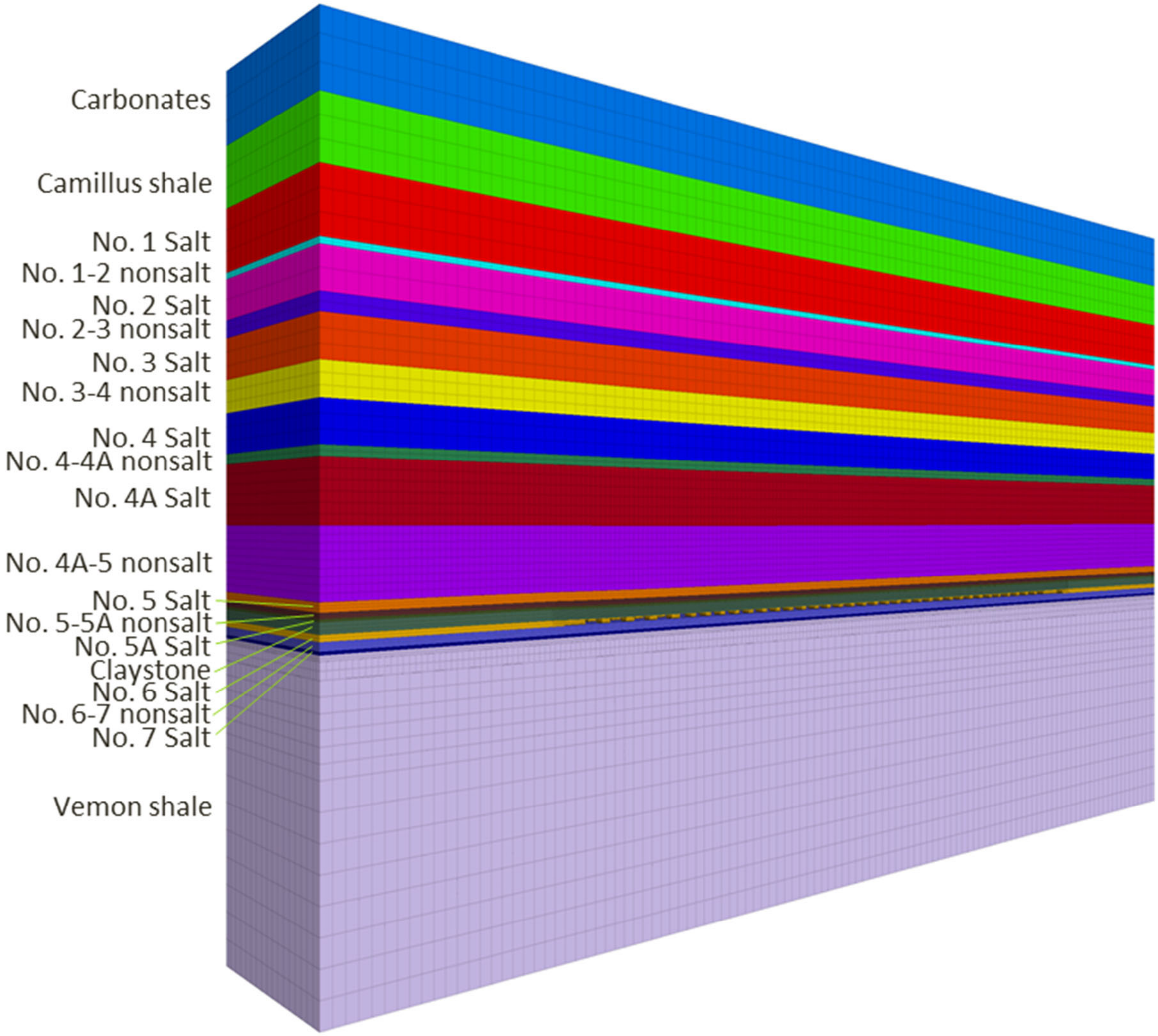

Figure 2. View of Model Extents and Layers

Table 1. Summary of Model Lithology, Unit Thickness, and Physical Properties

Model Layer	Material	Top Elevation (ft AMSL)	Thickness (ft)	Material Model	Young's Modulus ($\times 10^6$ psi)	Poisson's Ratio	Cohesion (psi)	Friction Angle (degrees)	Tensile Strength (psi)	Unit Weight (pcf)
n/a	Lake water	386.0	386.0	[redacted]						
n/a	Sed ments	0.0	200.0	[redacted]						
n/a	De onian Shale	-200.0	345.5	[redacted]						
n/a	arbonates	-545.5	337.3	[redacted]						
1	Ca bonates	-882.8	136.9	[redacted]						
2	Camill s Shale	-1019.7	114.6	[redacted]						
3	No. 1 Salt	-1134.3	117.8	[redacted]						
4	No. 1-2 Non-salt	-1252.1	11.8	[redacted]						
5	No. 2 Salt	-1263.9	74.9	[redacted]						
6	No. 2-3 Non-salt	-1338.8	32.6	[redacted]						
7	No. 3 Salt	-1371.4	77.0	[redacted]						
8	No. 3-4 Non-salt	-1448.4	60.5	[redacted]						
9	No. 4 Salt	-1508.9	74.6	[redacted]						
10	No. 4-4A Non-salt	-1583.5	18.9	[redacted]						
11	No. 4A Salt	-1602.4	111.0	[redacted]						
12	No. 4A-5 Non-salt	-1713.4	122.5	[redacted]						
13	No. 5 Salt	-1835.9	16.7	[redacted]						
14	No. 5-5A Non-salt	-1852.6	10.2	[redacted]						
15	No. 5A Salt	-1862.8	5.0	[redacted]						
16	No. 5A-6 Non-salt (claystone)	-1867.8	5.0	[redacted]						
17	No. 6 Salt (mining horizon)	-1882.8	1.5	[redacted]						
18	No. 6-7 Non-salt	-1871.3	15.1	[redacted]						
19	No. 7 Salt	-1856.2	6.0	[redacted]						
20	Vemon Shale	-1850.2	600.0	[redacted]						

n/a = Not applicable; pcf = pounds per cubic foot

Where $\dot{\epsilon}_{cr}$ is the creep rate, $\bar{\sigma}$ is the von Mises stress, and *A* and *n* are material properties. Based on a recent re-evaluation of salt creep testing results by RESPEC that excluded samples from pillar ribs that may have undergone damage, Cargill recommended application of the two-component form of the power law:

$$\dot{\epsilon}_{cr} = A_1\,\bar{\sigma}^{n_1} + A_2\,\bar{\sigma}^{n_2} \quad \text{(Eqn. 2)}$$

where [redacted]

2.2 Roof Deterioration Conceptual Model

The precise behavior of the claystone roof when exposed to water is unknown, but it is hypothesized that the current mining state has caused relaxation of the roof over the entries, meaning that portions of the roof are in tension or relatively low compression, and that these portions of the roof are especially susceptible to deterioration and failure as the panel is flooded.

Preliminary modeling runs confirmed that the claystone roof is in compression over the pillars, and in low compression or tension over the entries, with tensile stresses more prevalent near the pillar ribsides. In consultation with Dr. Mohanty, initial modeling results were reviewed, and several approaches to simulating roof deterioration were trialed. [redacted]

For comparison purposes, two models were run, a "base model" with no roof deterioration to simulate non-flooded conditions, and a "flooded model" with roof deterioration. The first 20 years of each scenario are dry, so that portion of the analysis was run with a single model. The base and flooded models were each run for 30 additional years to cover years 20 through 50 (in RESPEC's terminology, 0 through 30 years after flooding).

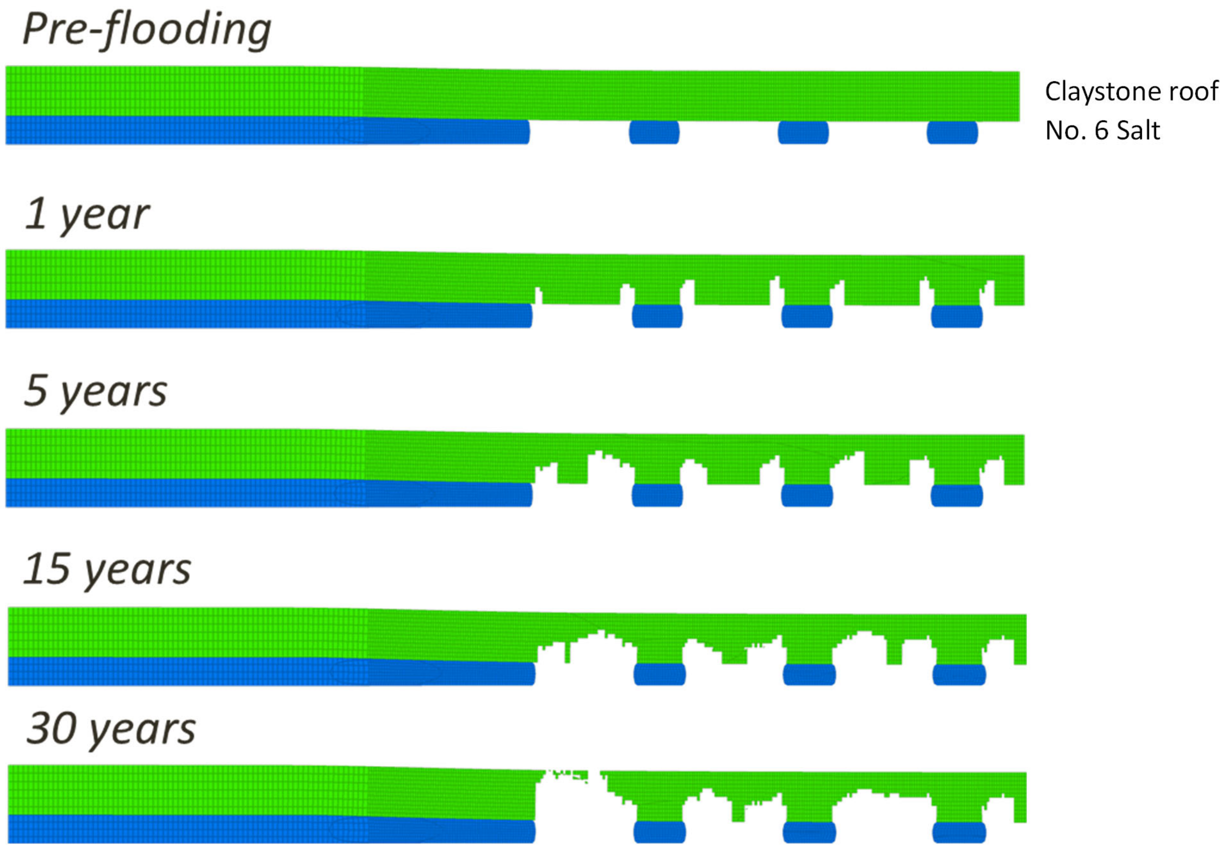

Figure 3. Vertical Section of Roof Deterioration Over Time with 240-psi Tensile Stress Criterion

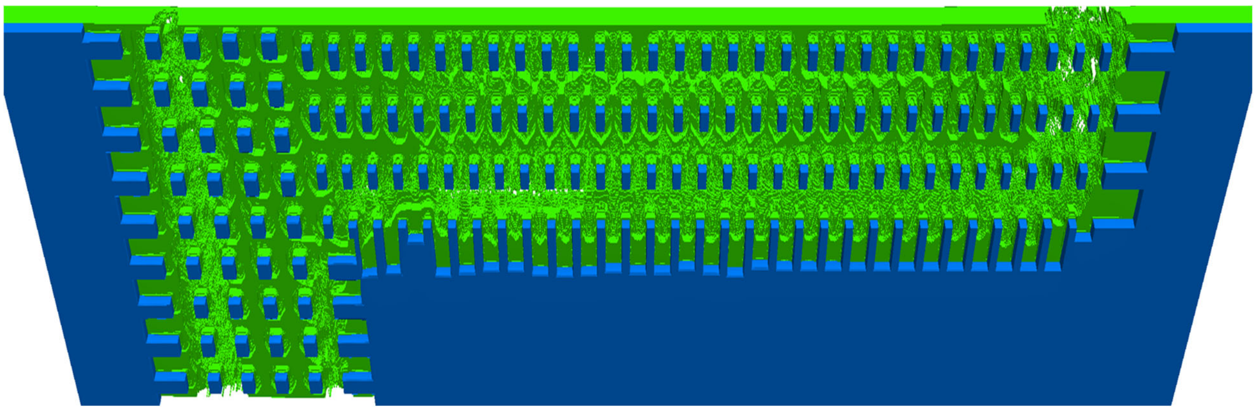

Figure 4. Pattern of Roof Degradation 30 Years after Flooding (looking up at roof surface)

3 Model Results

For ready comparison with previous RESPEC modeling results, Cargill requested specific modeling outputs to include:

- Vertical stress in the No. 6 Salt
- Vertical stress in sections along and perpendicular to the E5 Panel centerline
- Mohr-Coulomb safety factors in non-salt layers
- Damage potential in salt beds
- Closure in the No. 6 Salt
- Closure rate in the No. 6 Salt
- Overall convergence and convergence at specific points in the E5 Panel
- Hydraulic potential (the difference between depth below the Cayuga Lake surface expressed as water head and vertical stress in the model) in a section along the E5 Panel centerline

In general, the above outputs were developed for both the base and flooded models, with comparison times of 1 year, 5 years, 15 years, and 30 years after flooding (overall model times of 21, 25, 35, and 50 years).

The remainder of this section will present model outputs as figures and briefly comment on each set. However, there are only very subtle differences between the base and flooded models. The reasons for this are that as roof deterioration occurs in the flooded models, overburden loads are still carried by the pillars in a similar manner as in the base models. The deterioration does not lead to massive roof failure over the pillars, as the roof directly above the pillars remains intact and in compression. The overall pillar load and distribution, for practical purposes, is unaffected by the roof deterioration.

Somewhat counterintuitively, the flooded model shows slightly improved conditions. The reasons for this are twofold:

1) Deletion of roof elements causes a slight decrease in load carried by pillars. In the model, these roof elements are removed, while in reality, failed roof would fall to and rest on the floor, slightly increasing floor loading.
2) Flooding in the model is simulated as hydrostatic pressure applied to the roof, pillars, and floor. This pressure serves to confine the pillars, limiting their vertical shortening and dilation, and helps to support the roof.

Figures 5 through 8 show vertical stress at mid-height in the No. 6 Salt in plan view and along sections across and parallel to the E5 Panel. The (a) portion of each figure is the base case, and the (b) portion is the flooded case. Although the time periods referenced are "after flooding," note that the base case represents no flooding. The stress distribution 1 year after flooding is very similar for the base case (Figure 5a) and the flooded case (Figure 5b). Stresses in the panel pillars are about 3,400 psi, and in the submains are slightly higher (up to about 4,000 psi) due to their being larger and having undergone less deformation. The salt creep tends to transfer loads from the panel and submains pillar to the abutments (solid salt). The abutments south of the submains have peak stresses of about 9,000 psi, while the end-panel abutment peak stress is about 8,000 psi.

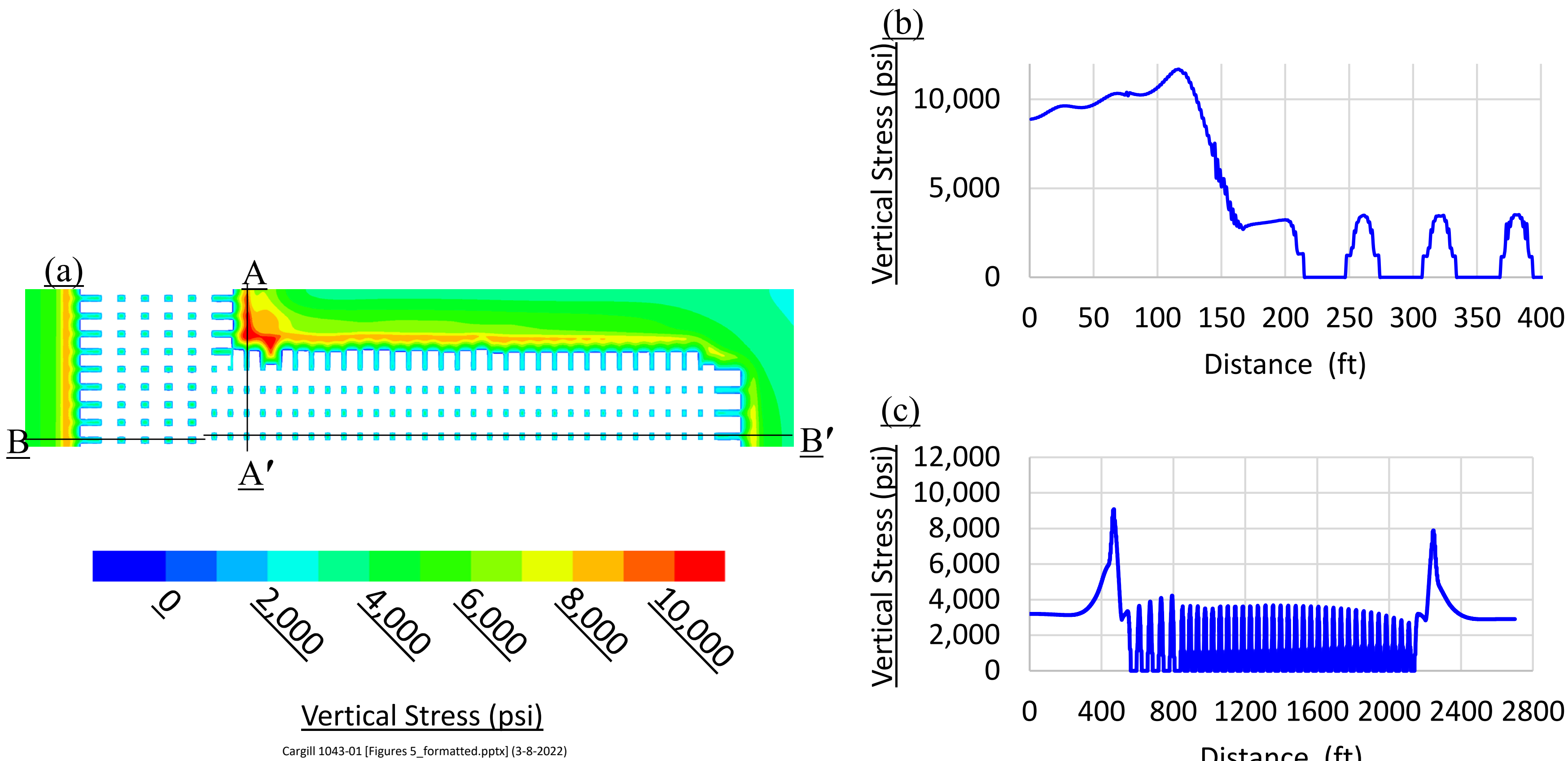

Figure 5a. Vertical Stress at Mid-Seam Height 1 Year after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

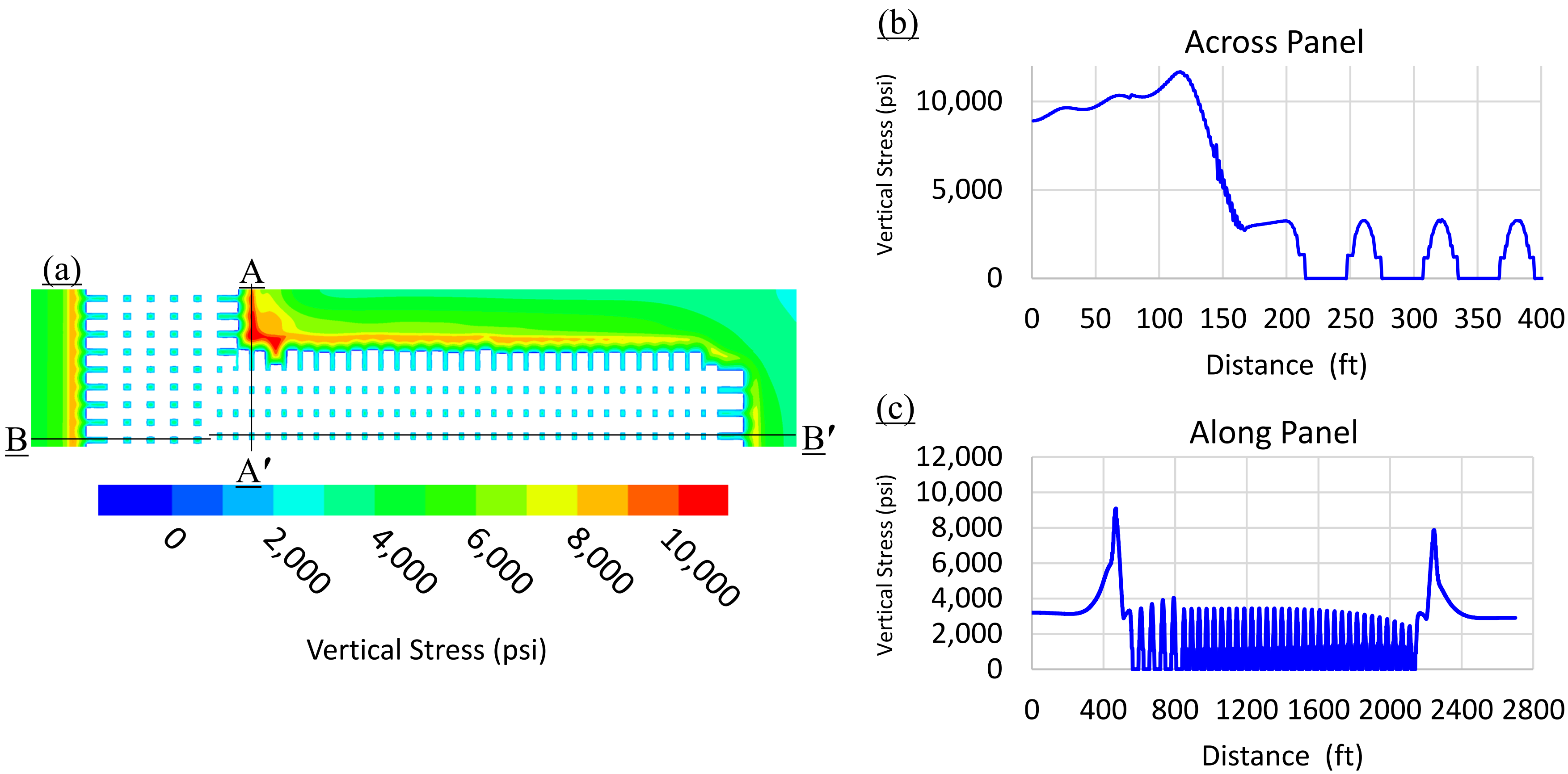

Figure 5b. Vertical Stress at Mid-Seam Height 1 Year after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

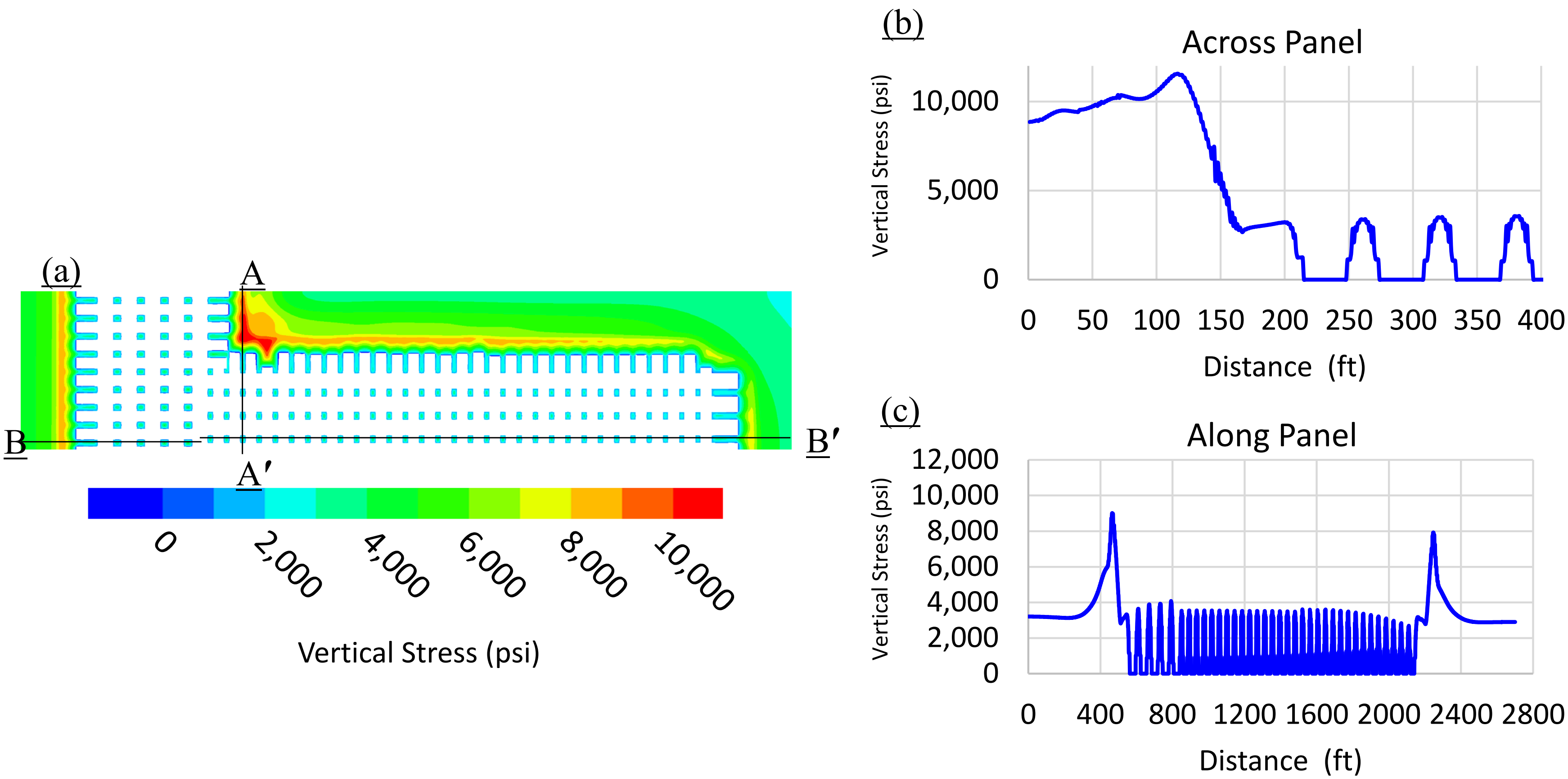

Figure 6a. Vertical Stress at Mid-Seam Height 5 Years after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

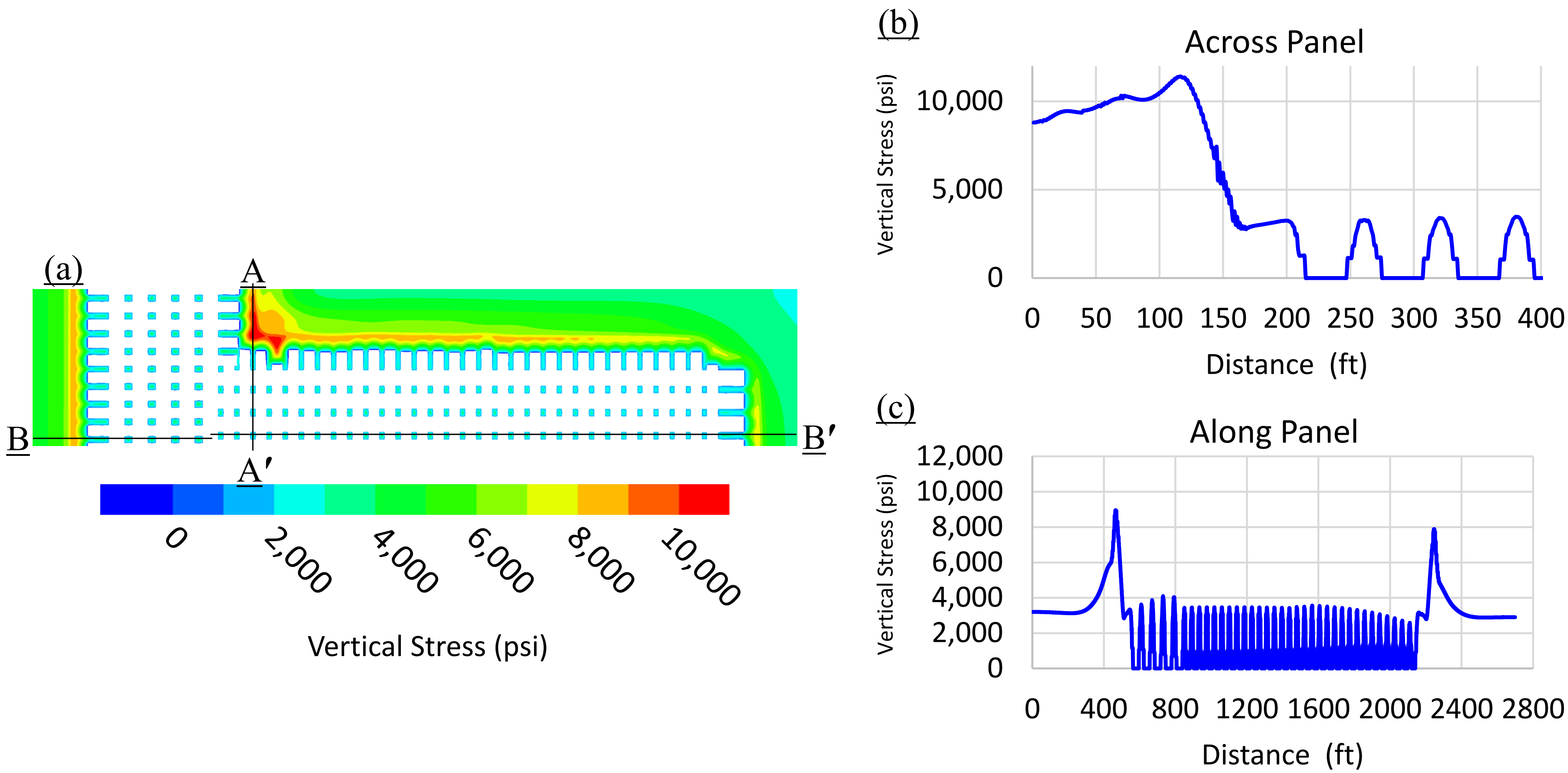

Figure 6b. Vertical Stress at Mid-Seam Height 5 Years after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

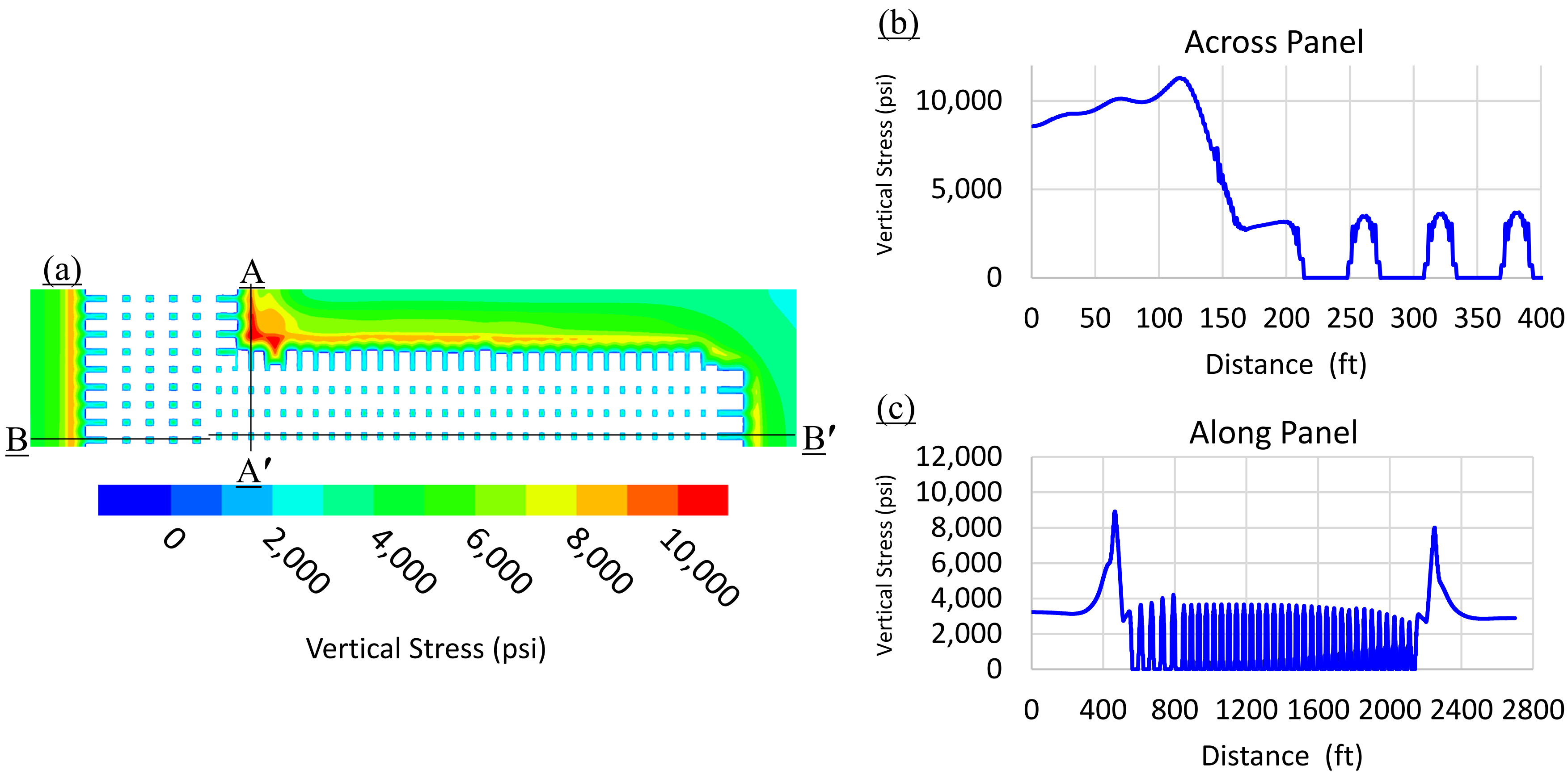

Figure 7a. Vertical Stress at Mid-Seam Height 15 Years after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

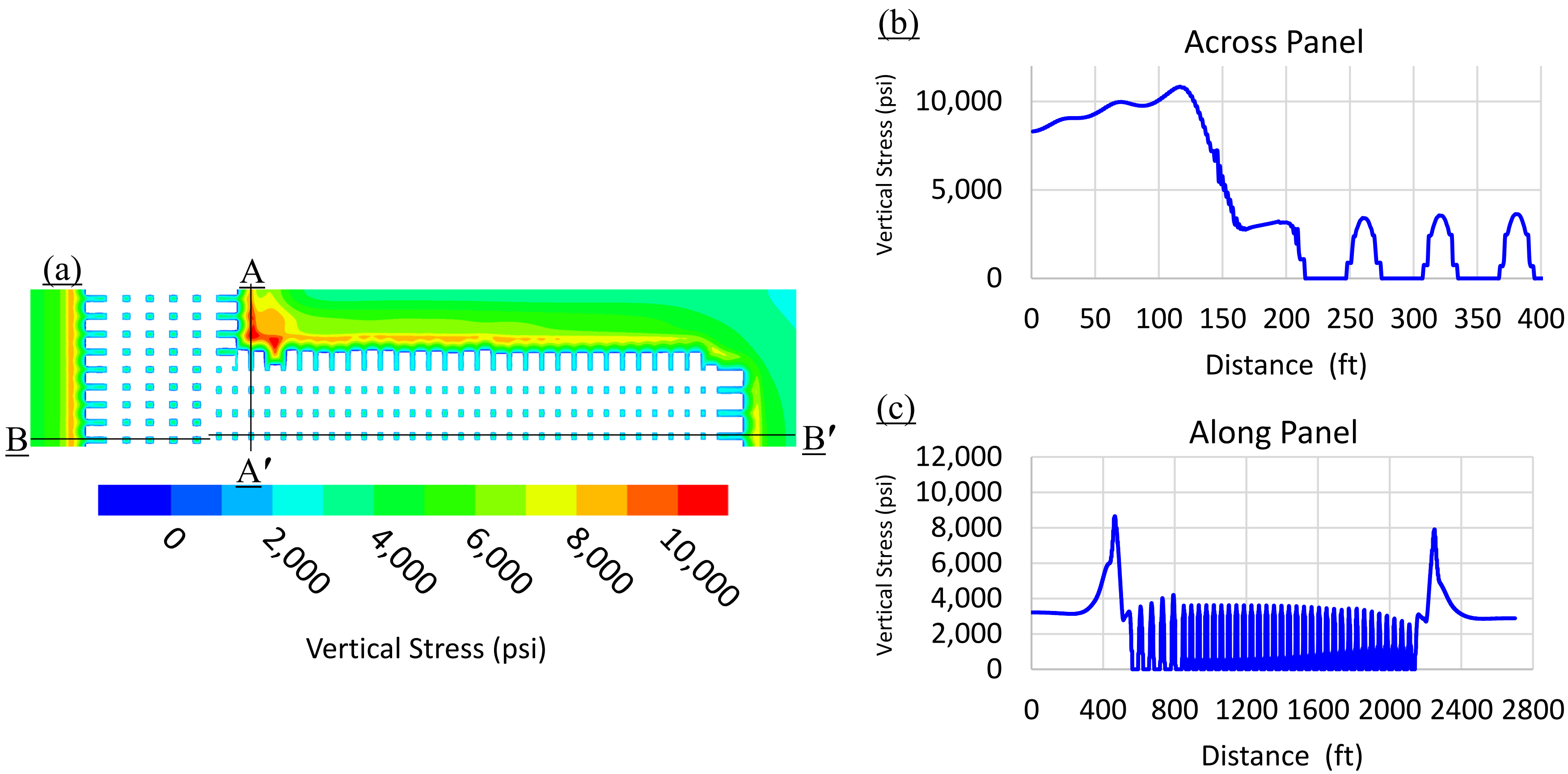

Figure 7b. Vertical Stress at Mid-Seam Height 15 Years after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

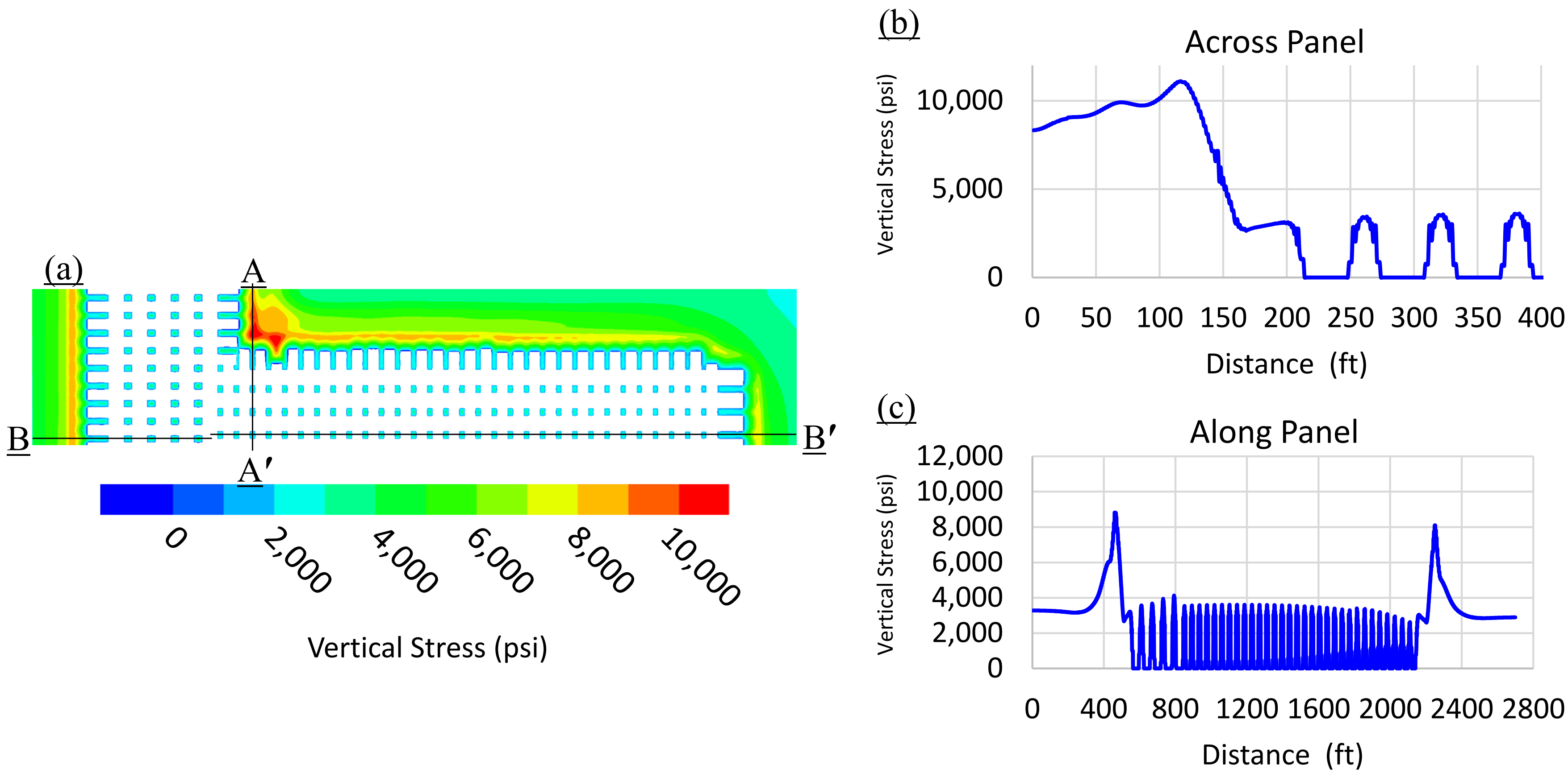

Figure 8a. Vertical Stress at Mid-Seam Height 30 Years after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

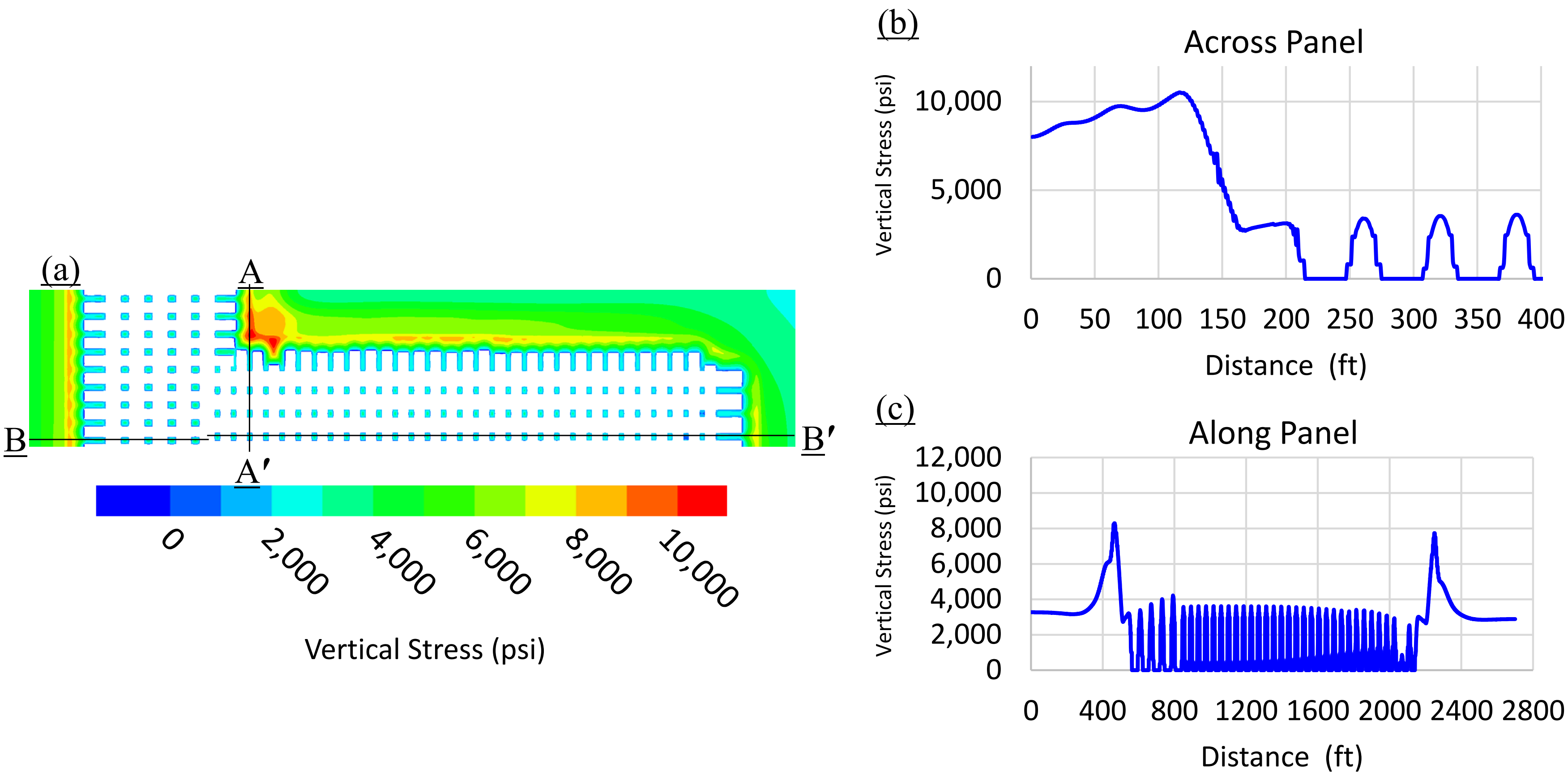

Figure 8b. Vertical Stress at Mid-Seam Height 30 Years after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

The corner of the submain/panel has a peak abutment stress of about 11,600 psi. Close comparison of the base and flooded cases show that in the flooded case, the pillars are carrying slightly lower stress, due to the roof deterioration and internal pressure assumptions previously discussed.

As time progresses to 5, 15, and 30 years after flooding (Figures 6, 7, and 8, respectively), the slight differences between the base and flooded models are accentuated slightly with the increasing internal pressure from the increasing stored water volumes. Peak stresses decrease somewhat as the salt continues to creep.

Figures 9 through 12 show vertical stress in sections across and parallel to the E5 Panel (the (b) and (c) parts of each figure; the (a) portion shows the plan view stress in the No. 6 Salt for reference. Close inspection of the size of the stress-relieved zone above the submain and E5 Panel shows a) an increase in the size of this zone with time as the salt pillars creep and b) a slightly smaller zone in the flooded model than the base model.

Mohr-Coulomb safety factors are shown in vertical section for the non-salt layers over time in Figure 13. The red areas indicate zones where tensile stresses approach or reach the tensile strength (600 psi). These zones may be subject to crack formation outside the panel boundaries. However, it should be noted that Dr. Mohanty is of the opinion that the assumed rock mass tensile strengths are rather conservative for the non-salt overburden layers at the Cayuga Mine, and Agapito concurs. Therefore, these red zones in the model are unlikely to have a practical effect on panel stability. Layers higher than the 4A-5 show high safety factors. Plan view plots of non-salt safety factors 30 years after flooding (Figures 14 through 17) show that the low safety factors are concentrated over the panel/abutment boundaries, where bending of the layer is greatest. In particular, Figure 14 b shows that the claystone roof above the pillars is stable in the flooded model, while the white areas in the plot show cavities where the claystone roof has deteriorated over the entries. Again, flooding has a small stabilizing effect on global stability.

Damage potential for salt layers was calculated using equations supplied by Cargill based on like calculations by RESPEC. The criteria given by RESPEC for salt damage potential (DP) is:

- [redacted]
- [redacted]
- [redacted]

Contours of damage potential are shown in vertical section for the salt layers over time in Figure 18. As is shown, significant damage potential is limited to the overlying No. 5A and No. 5 salt beds where bending is induced, with slightly less damage potential in the flooded model. Note that the elevated damage potential seen at the top of the No. 1 salt beginning in year 15 is related to the interface between the salt and the Camillus shale (assumed to be elastic). This is a modeling edge artifact of no practical consequence.

Further details of the salt damage potential are shown in plan view plots of the various salt layers 30 years after flooding (Figures 19 through 22). Like the non-salt layers, damage potential is greatest at panel/abutment boundaries, and flooding has a small stabilizing effect. Damage potential in the No. 4A salt (Figure 22) and overlying layers is negligible.

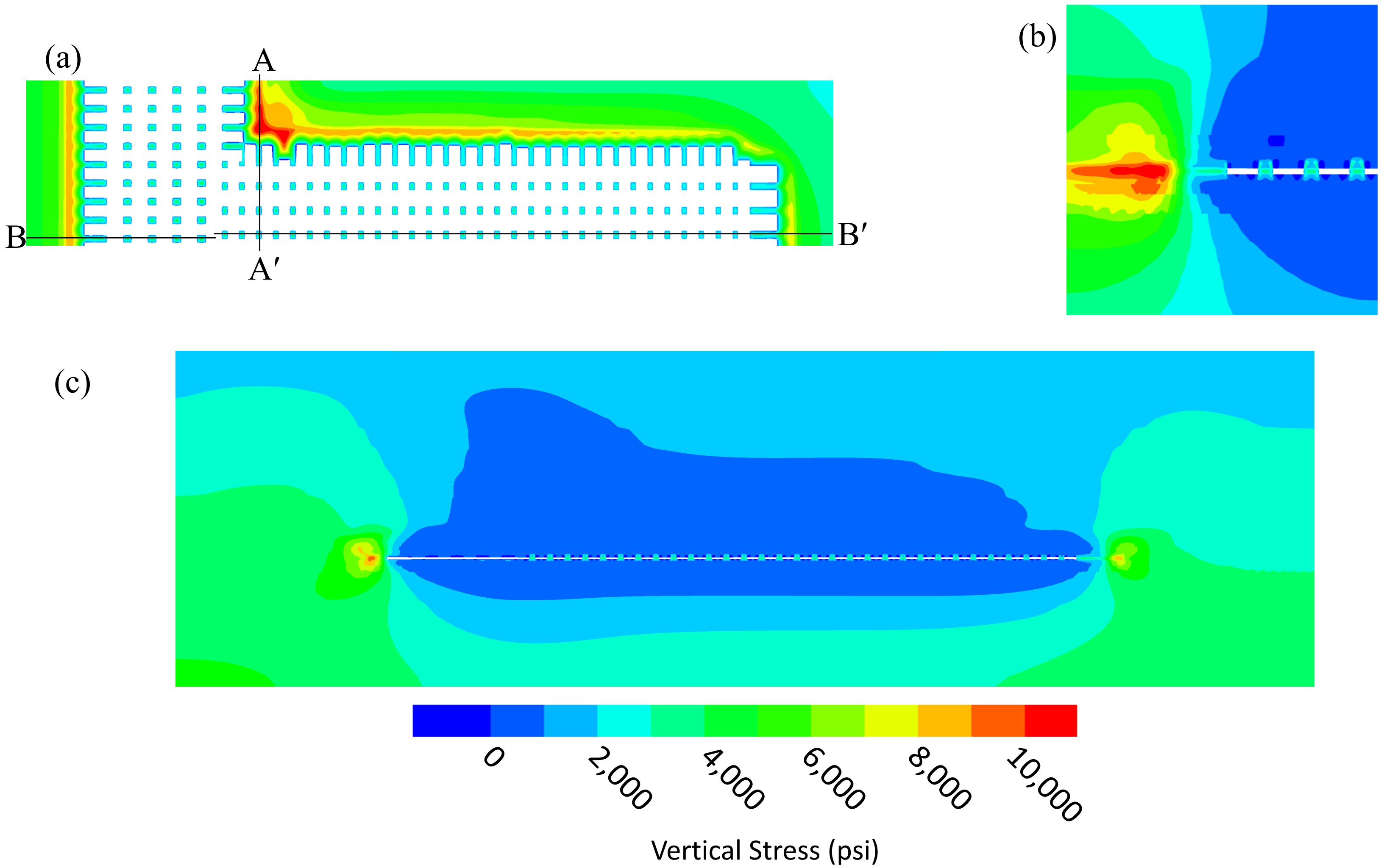

Figure 9a. Vertical Stress 1 Year after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

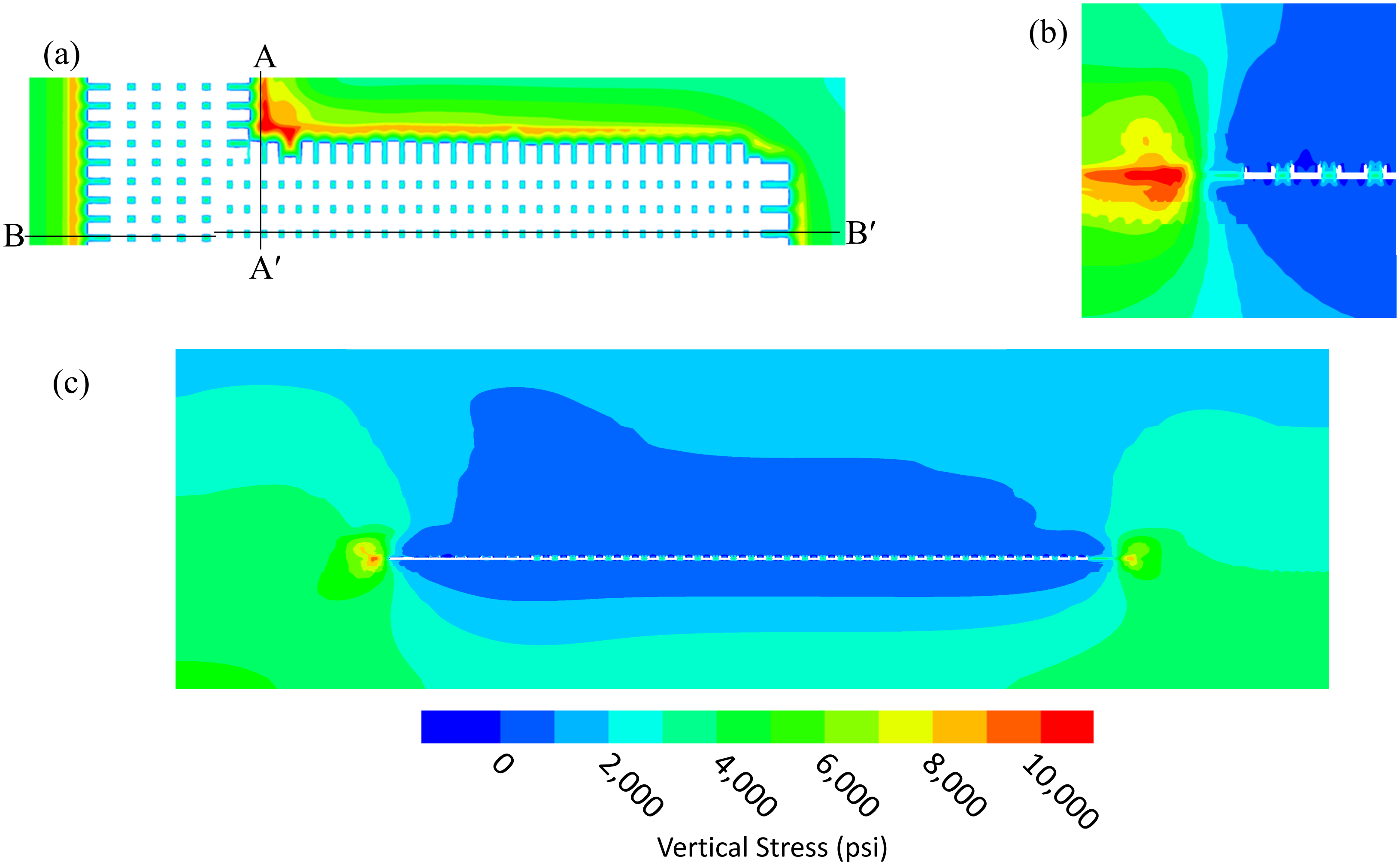

Figure 9b. Vertical Stress 1 Year after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

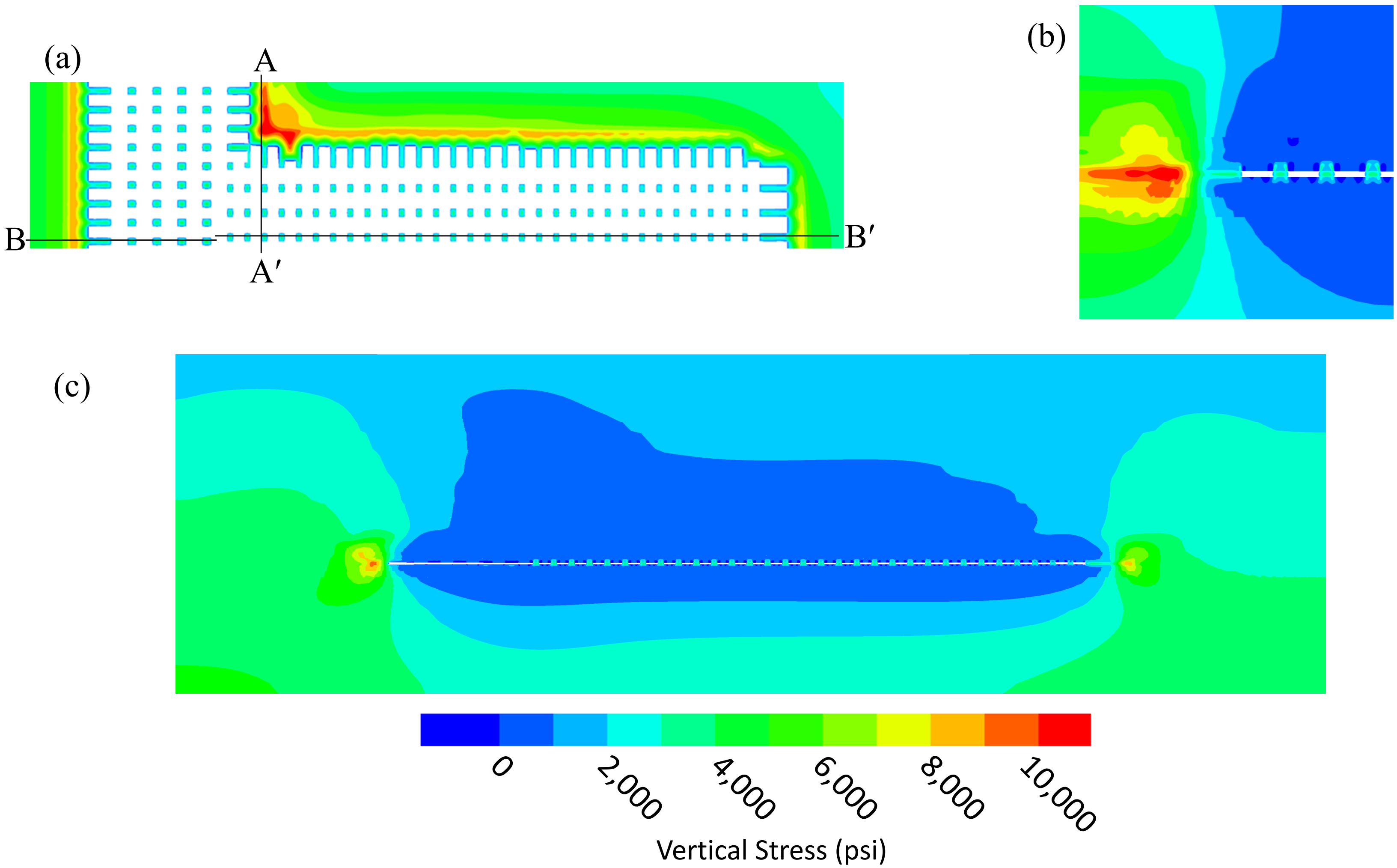

Figure 10a. Vertical Stress 5 Years after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

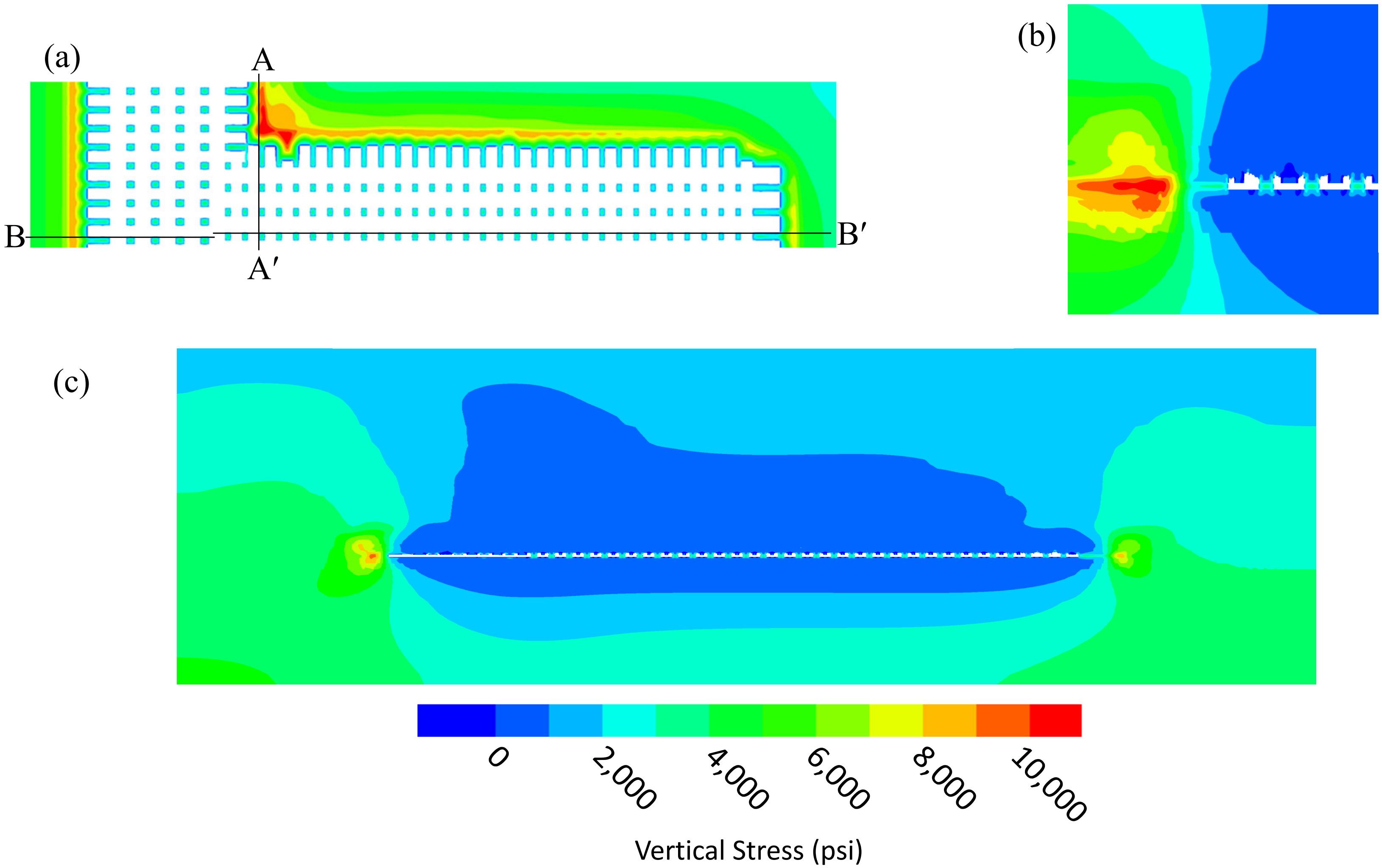

Figure 10b. Vertical Stress 5 Years after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

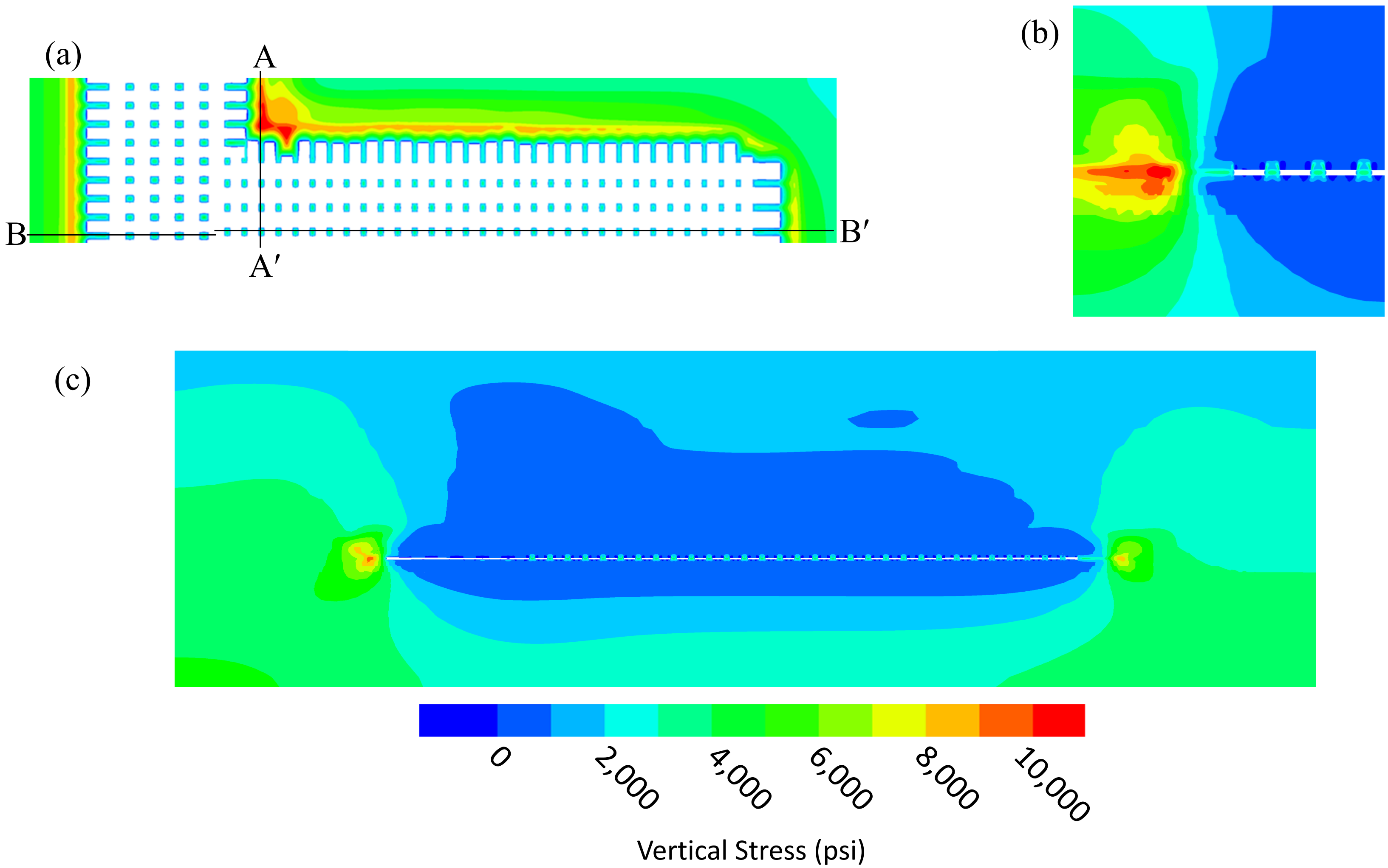

Figure 11a. Vertical Stress 15 Years after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

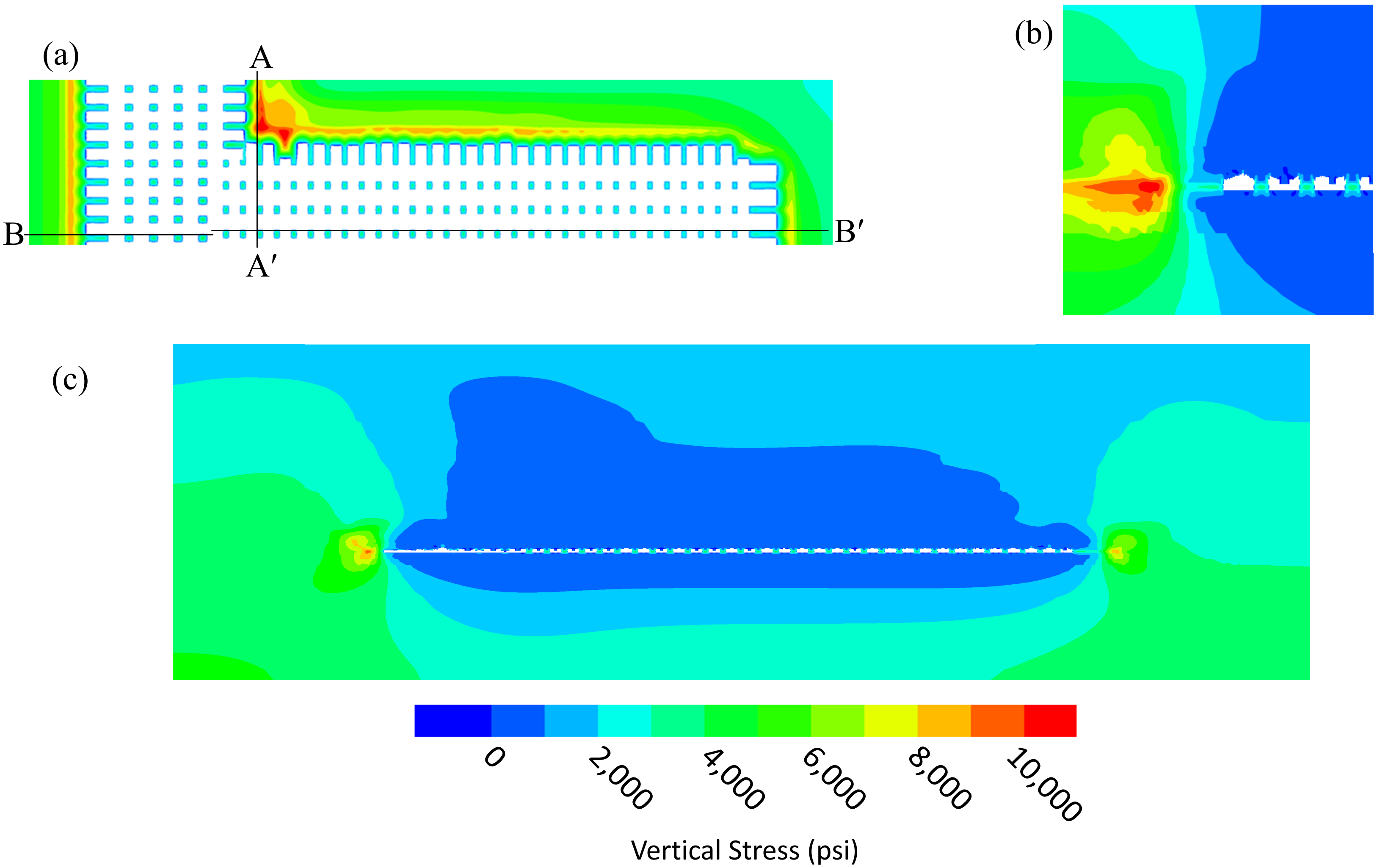

Figure 11b. Vertical Stress 15 Years after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

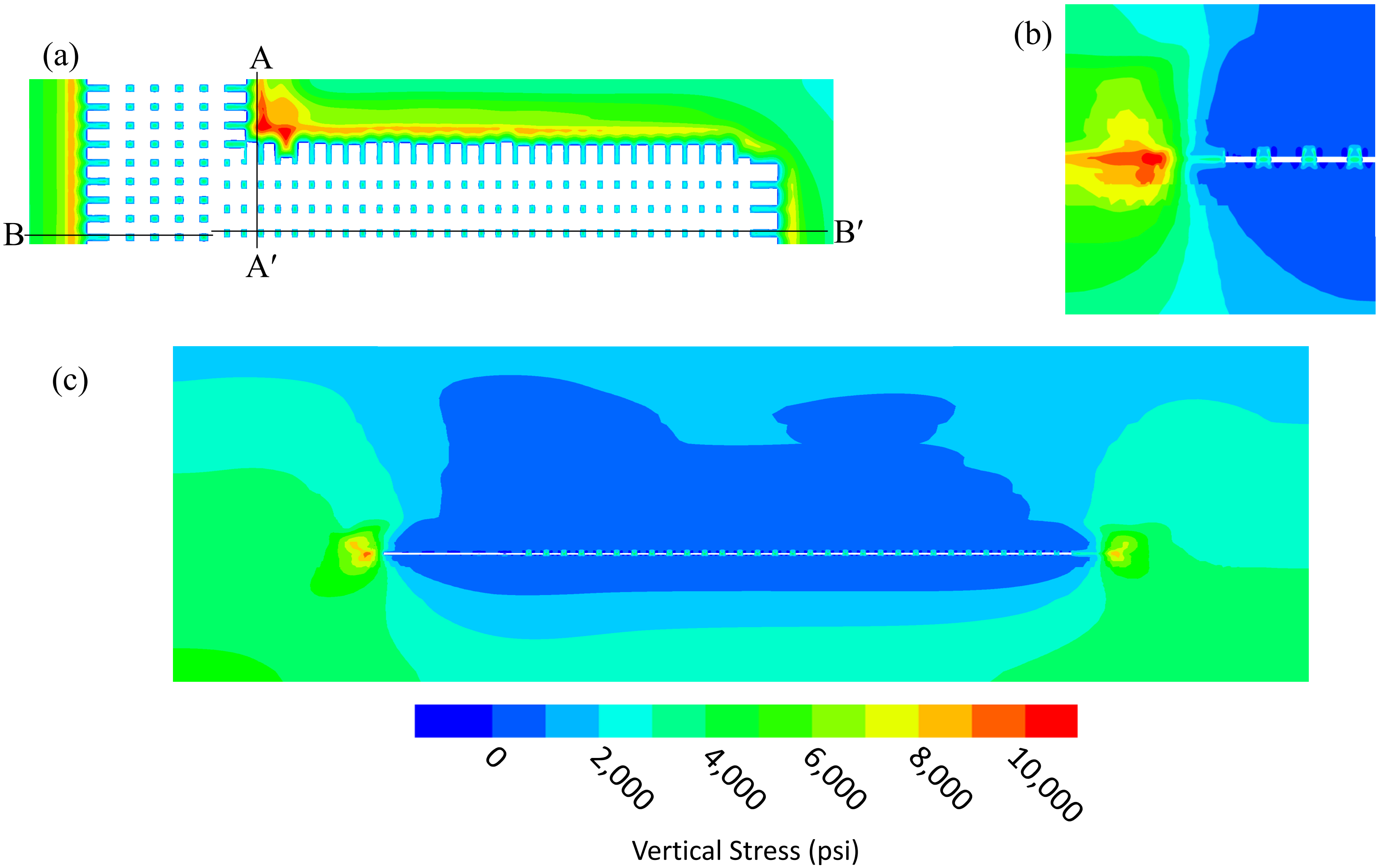

Figure 12a. Vertical Stress 30 Years after Flooding, Base Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

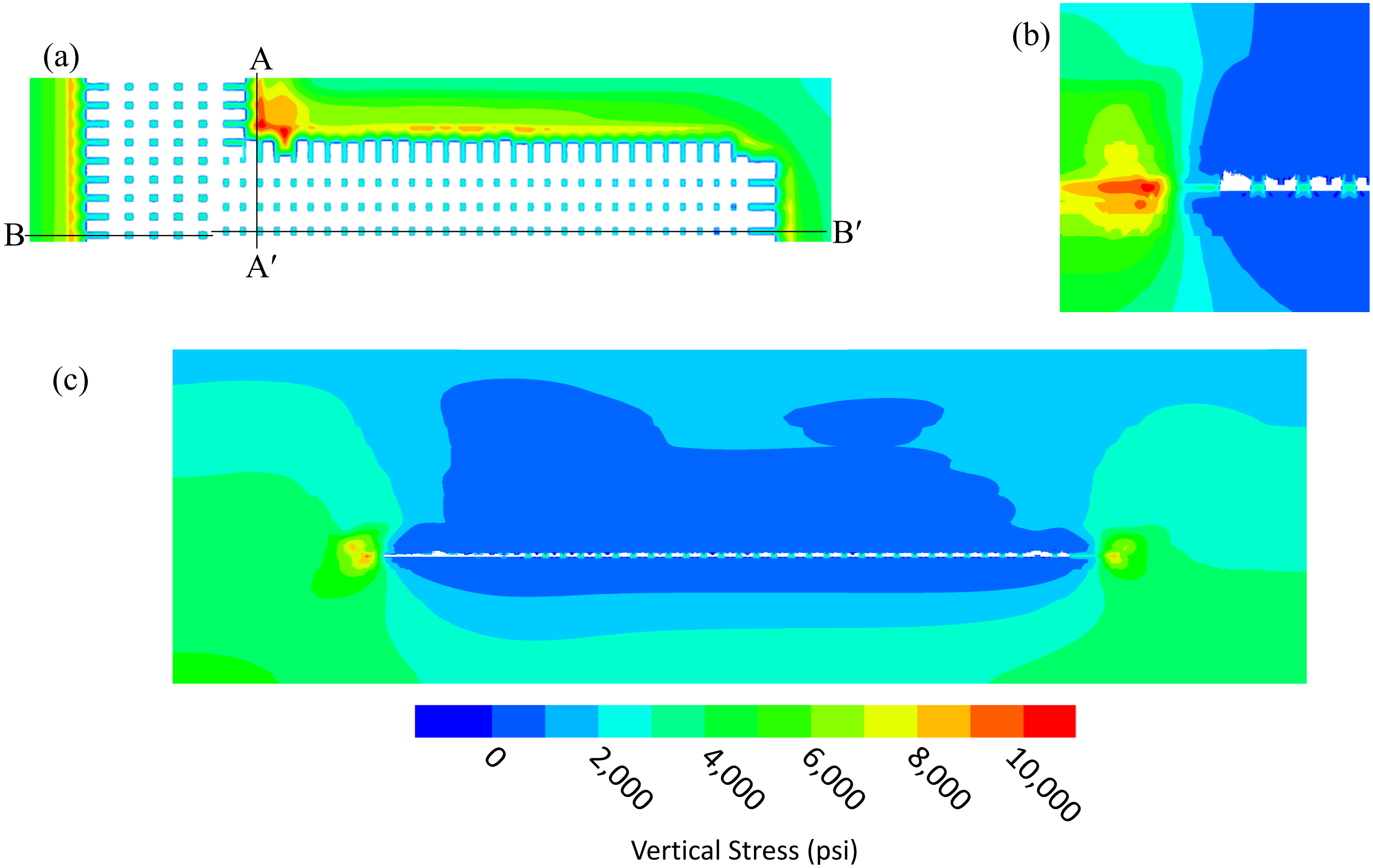

Figure 12b. Vertical Stress 30 Years after Flooding, Flooded Model (a) Plan View, (b) Section A-A′, (c) Section B-B′

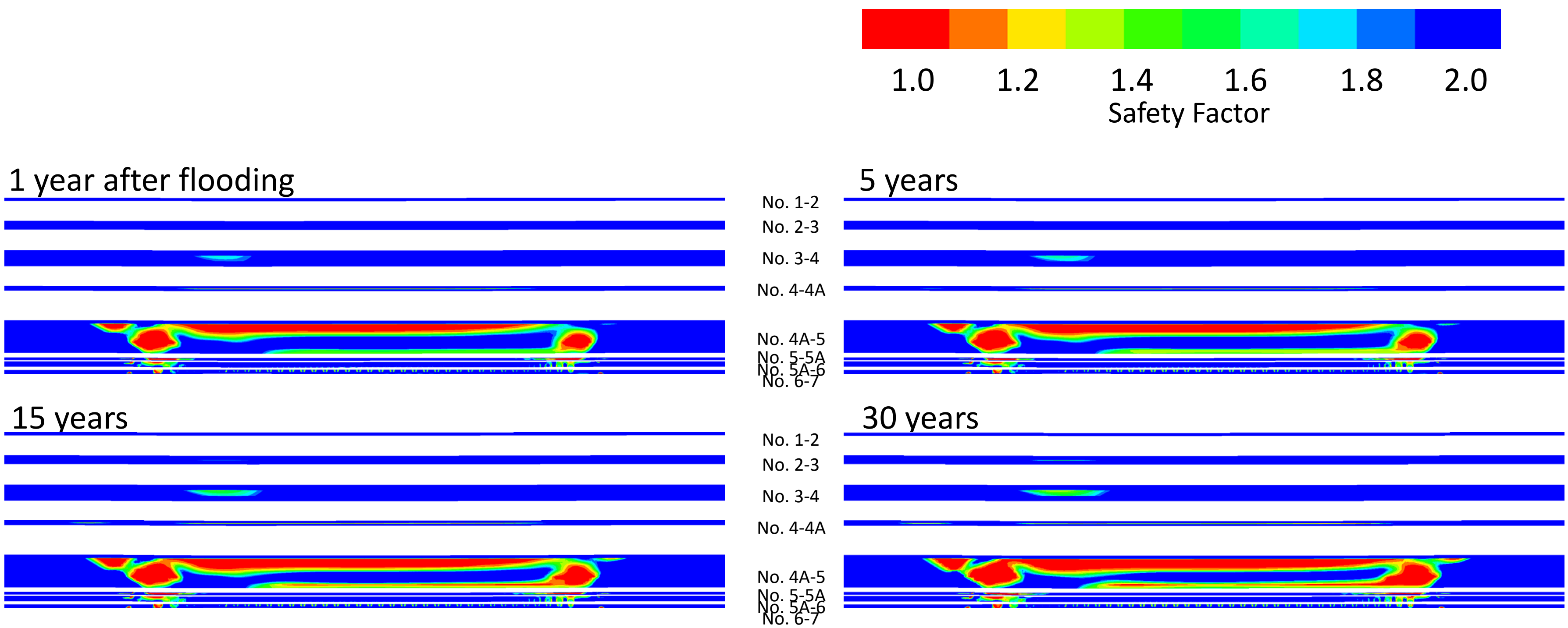

Figure 13a. Mohr-Coulomb Safety Factors in Non-Salt Layers, Base Model
(vertical section along E5 Panel centerline)

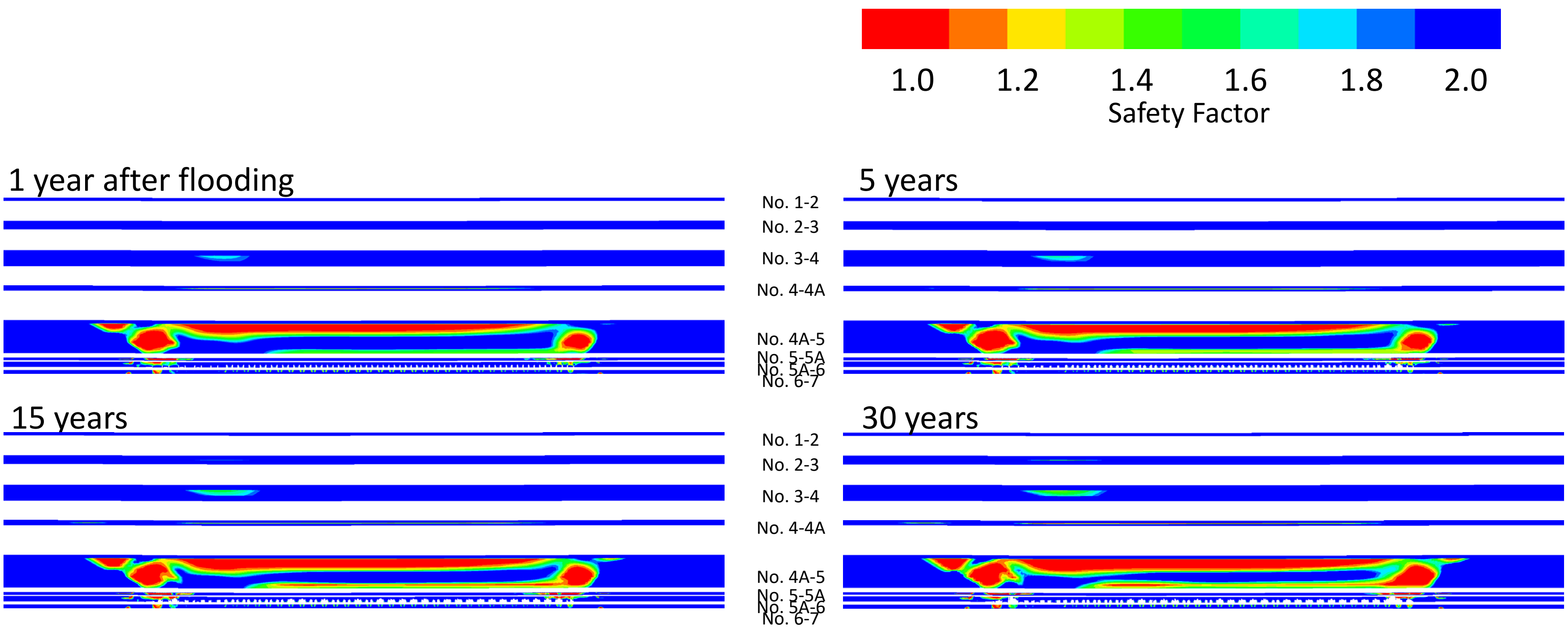

Figure 13b. Mohr-Coulomb Safety Factors in Non-Salt Layers, Flooded Model
(vertical section along E5 Panel centerline)

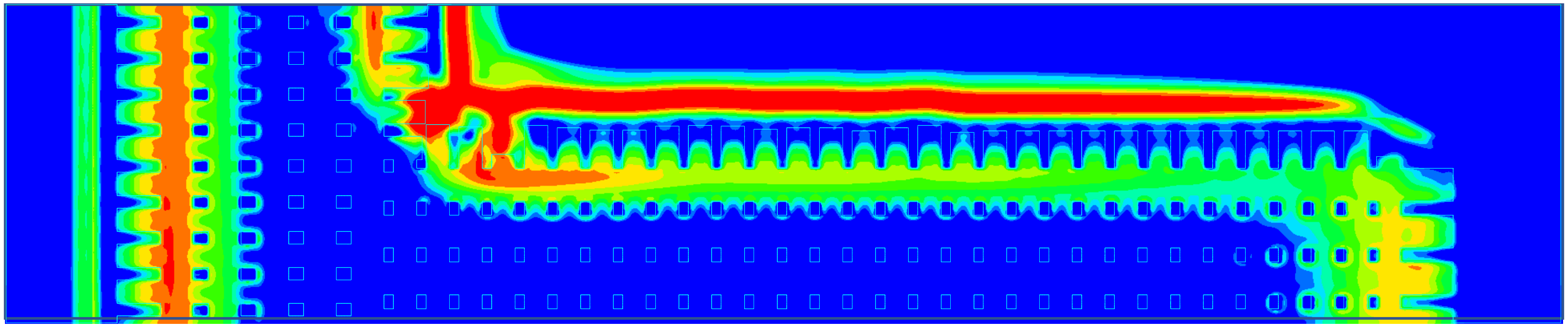

Figure 14a. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 5A-6 Non-Salt (claystone roof), **Base Model**

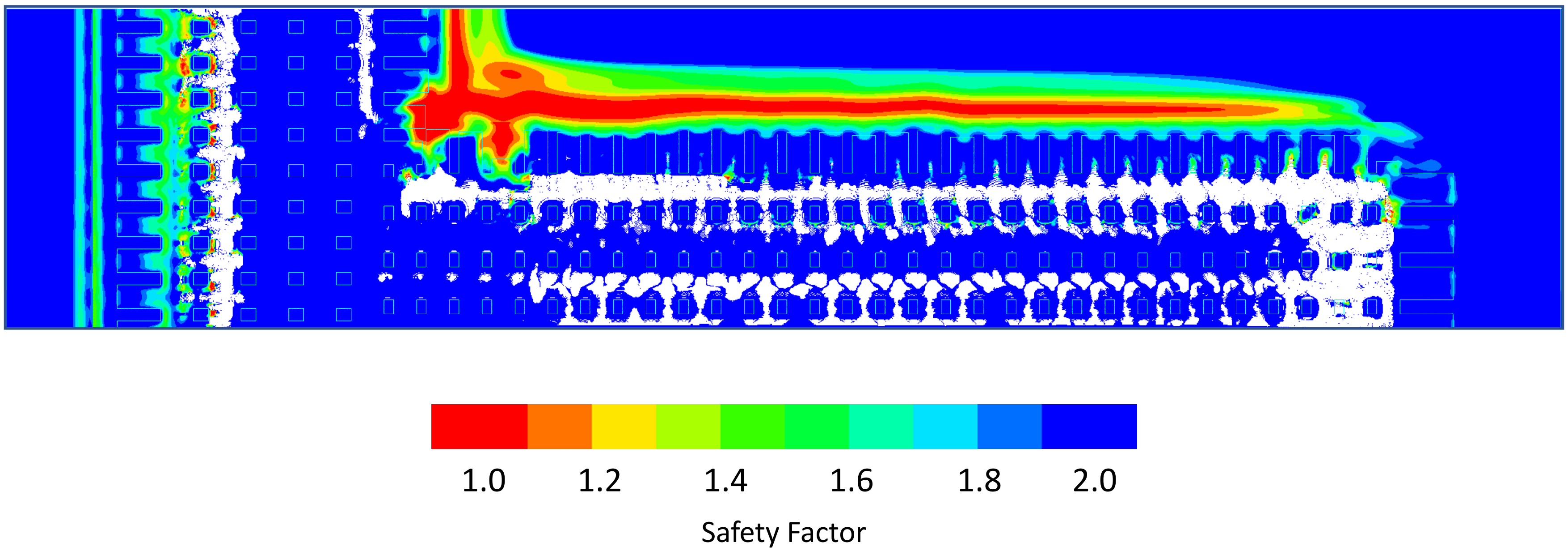

Figure 14b. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 5A-6 Non-Salt (claystone roof), **Flooded Model**

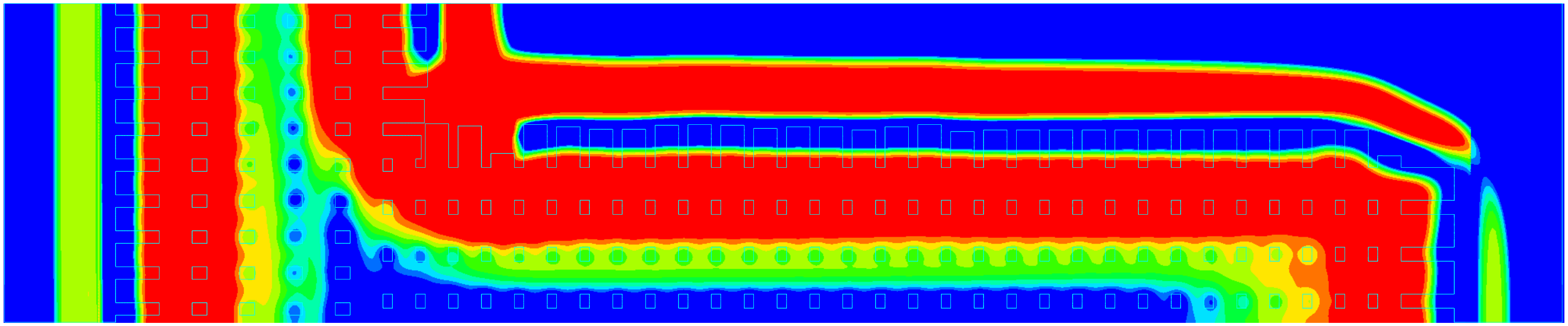

Figure 15a. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 5-5A Non-Salt, Base Model

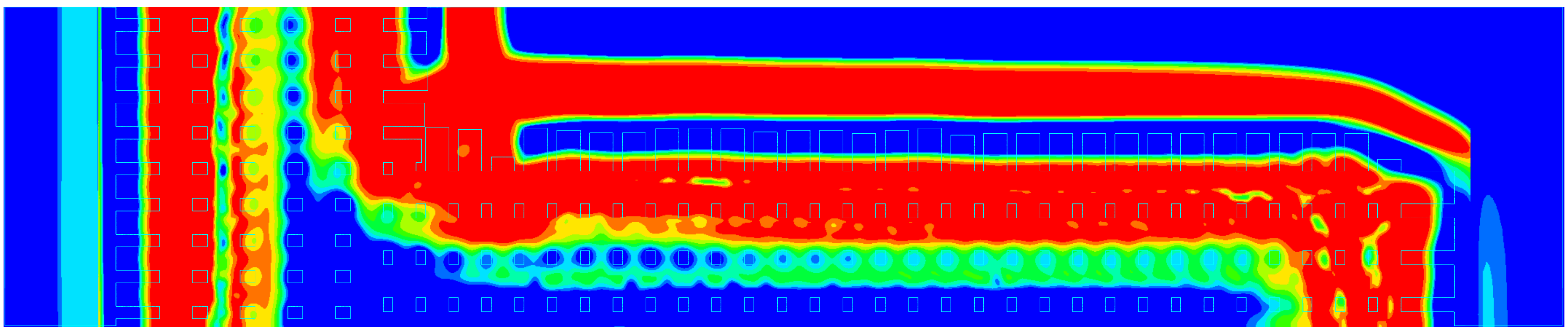

Figure 15b. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 5-5A Non-Salt, Flooded Model

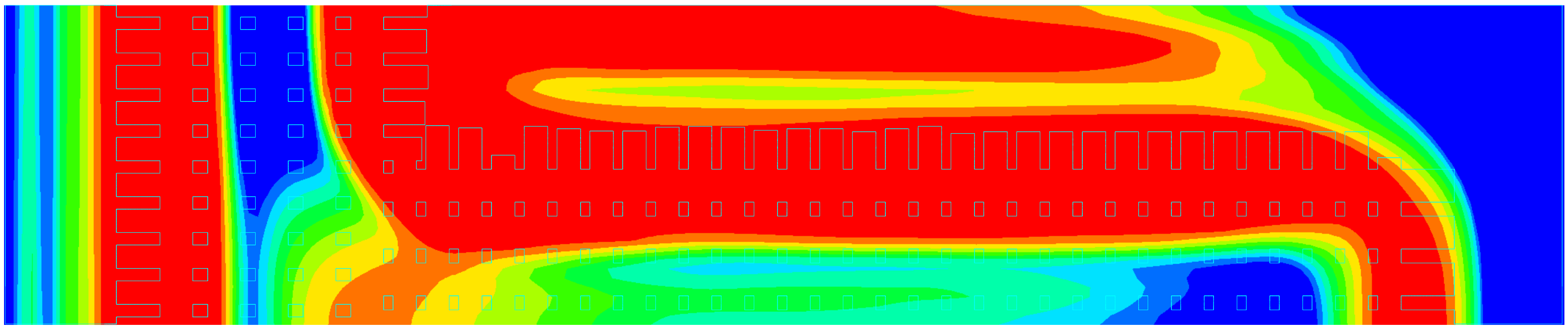

Figure 16a. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 4A-5 Non-Salt, Base Model

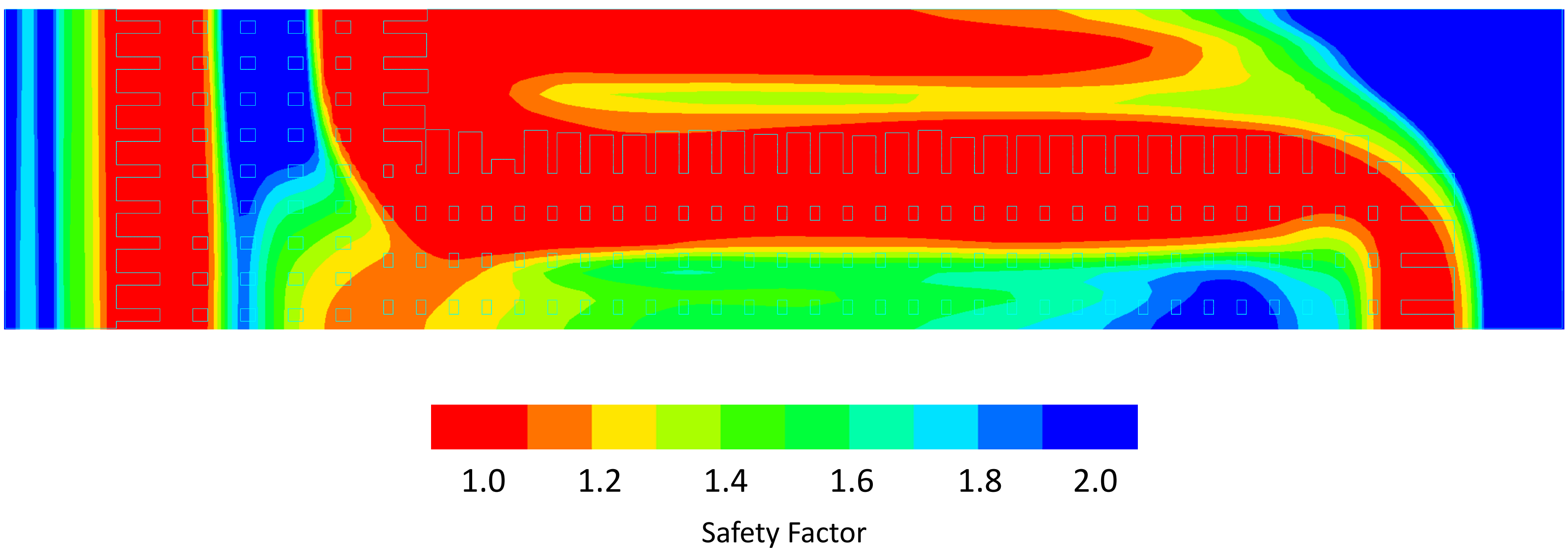

Figure 16b. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 4A-5 Non-Salt, Flooded Model

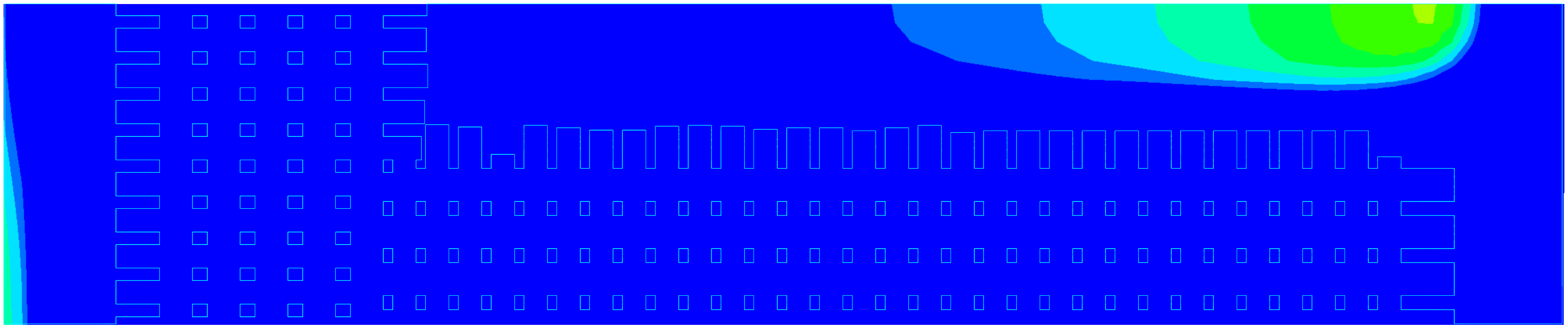

Figure 17a. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 4-4A Non-Salt, Base Model

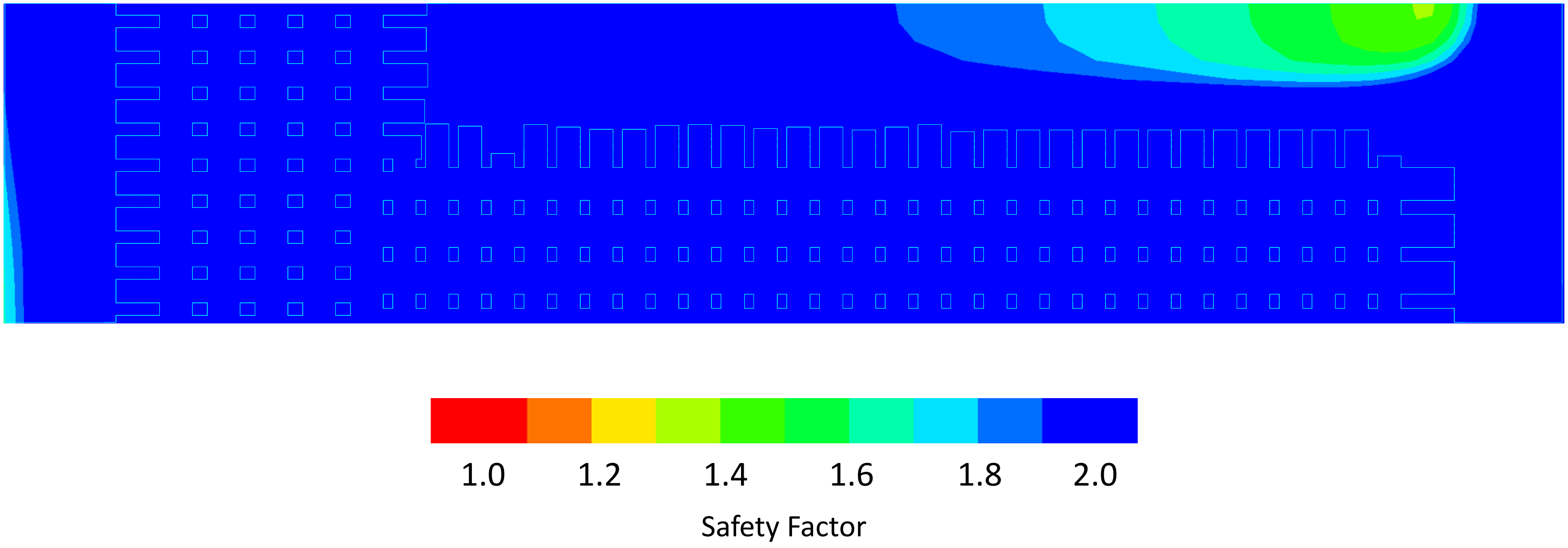

Figure 17b. Mohr-Coulomb Safety Factors 30 Years after Flooding, No. 4-4A Non-Salt, Flooded Model

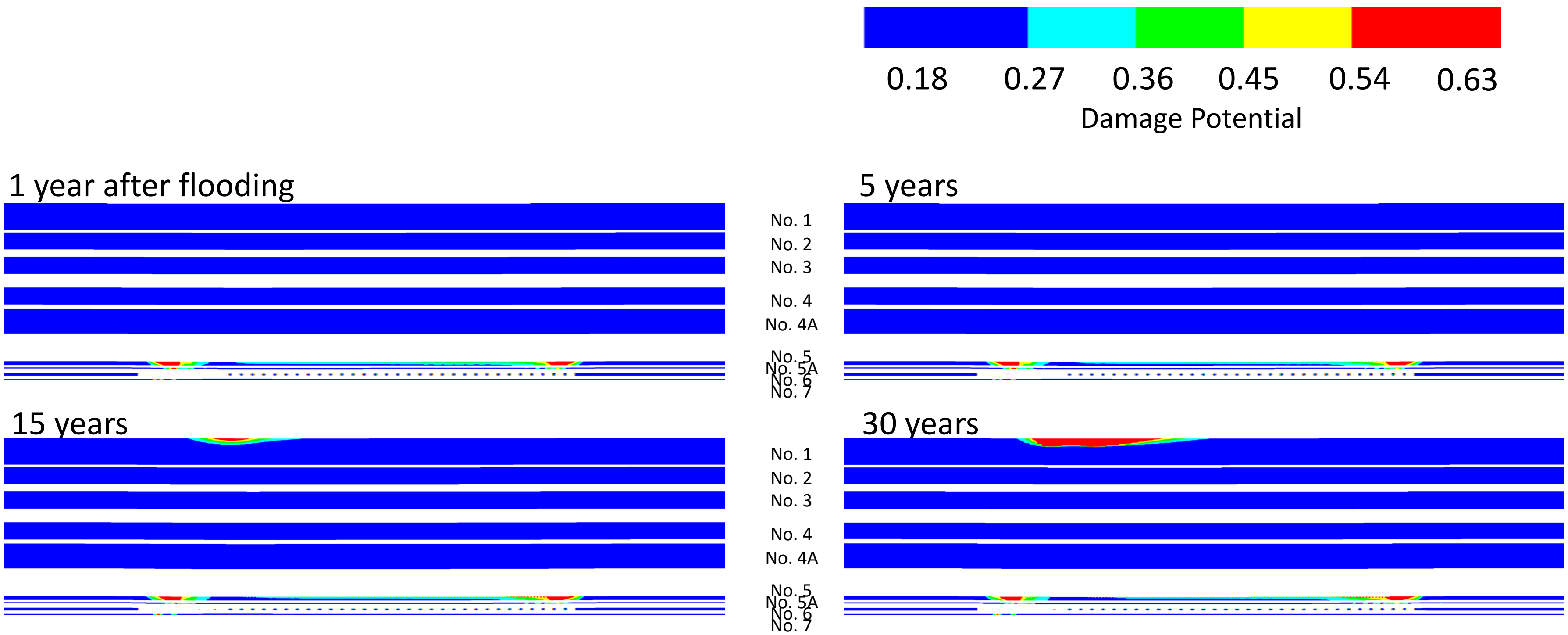

Figure 18a. Salt Damage Potential, Base Model (vertical section along E5 Panel centerline)

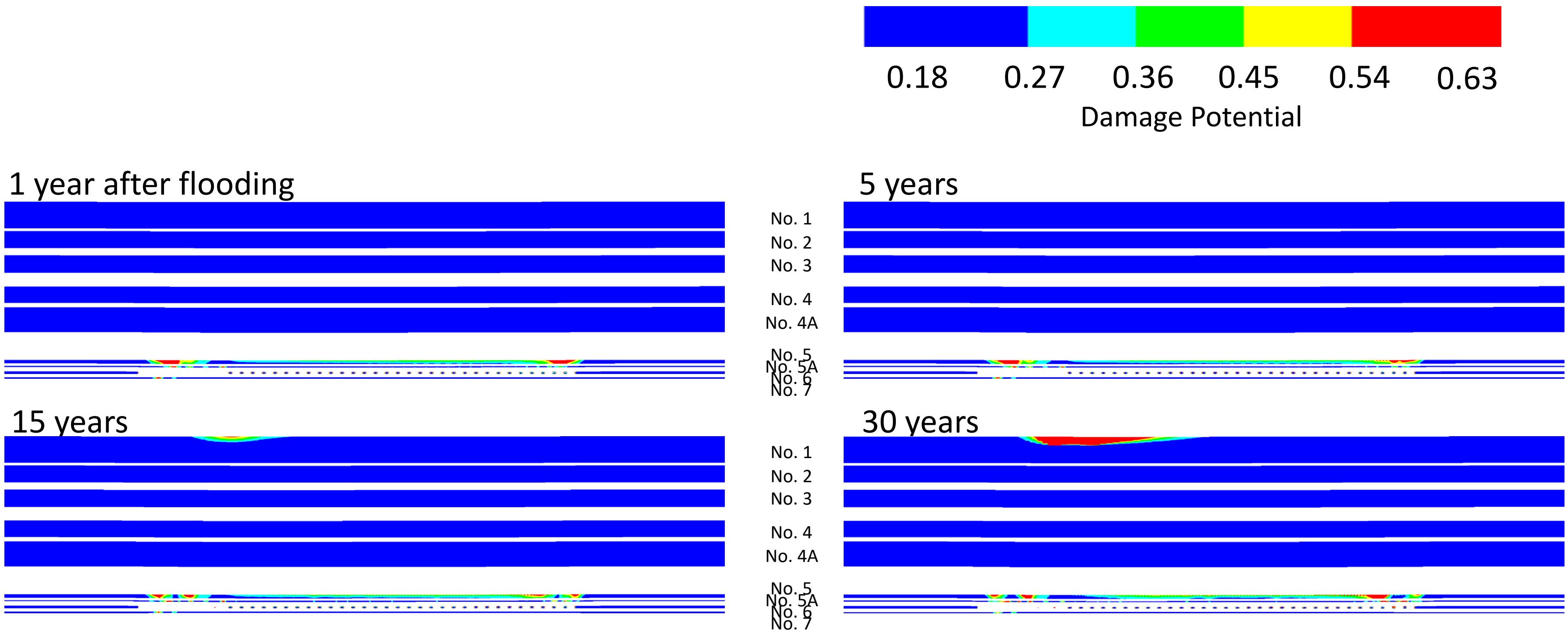

Figure 18b. Salt Damage Potential, Flooded Model (vertical section along E5 Panel centerline)

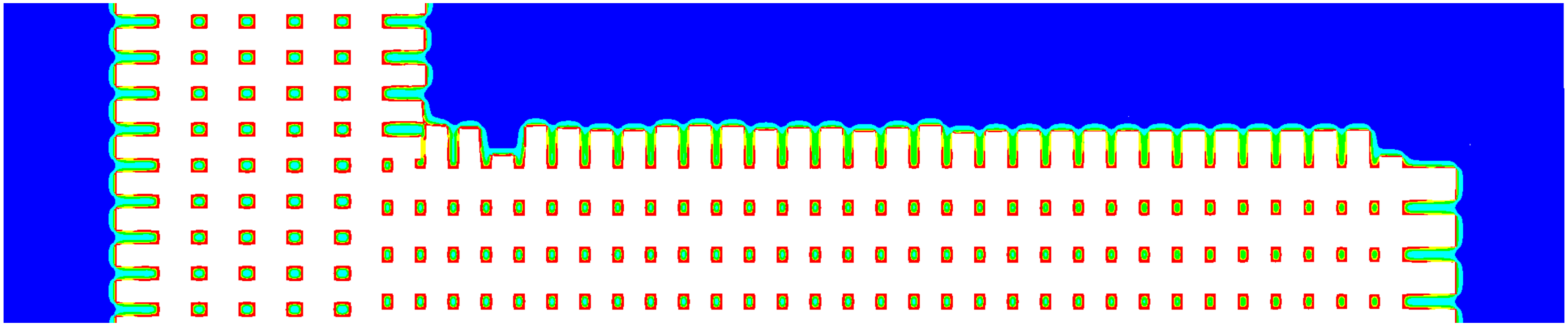

Figure 19a. Damage Potential 30 Years after Flooding, No. 6 Salt, Base Model

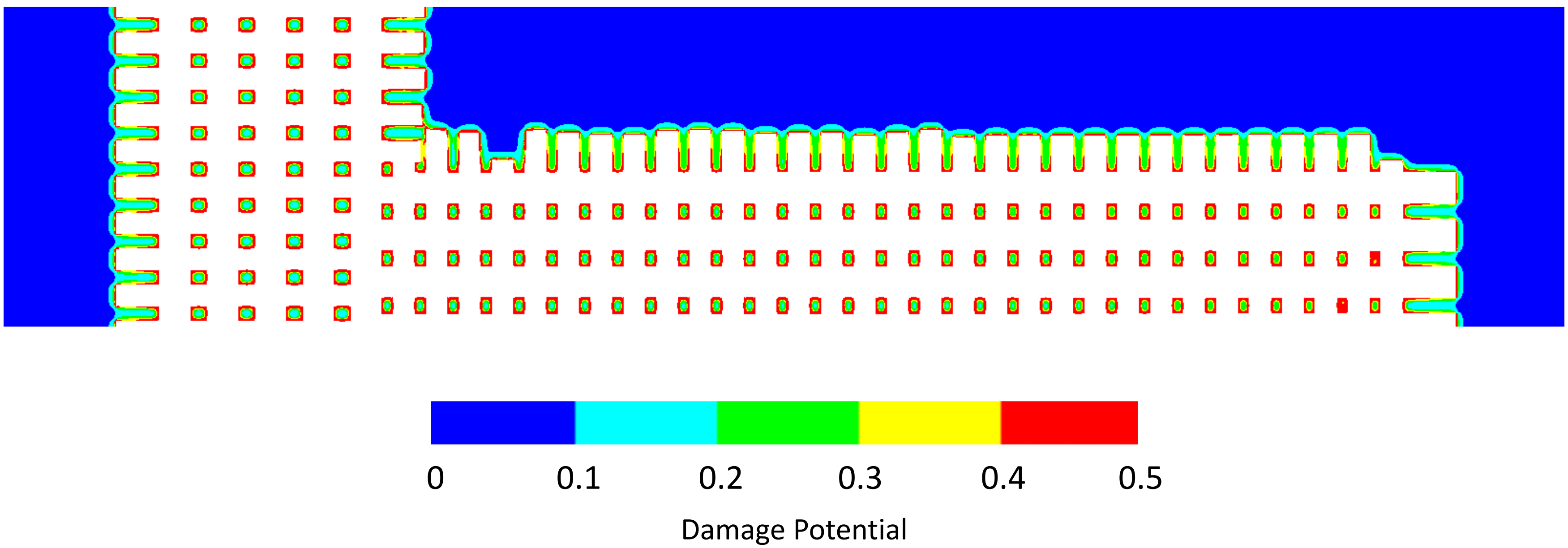

Figure 19b. Damage Potential 30 Years after Flooding, No. 6 Salt, Flooded Model

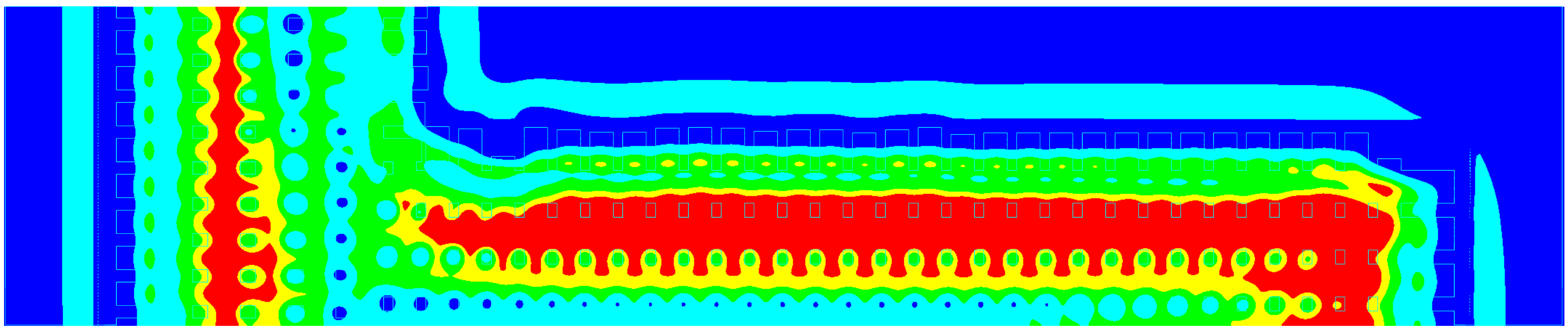

Figure 20a. Damage Potential 30 Years after Flooding, No. 5A Salt, Base Model

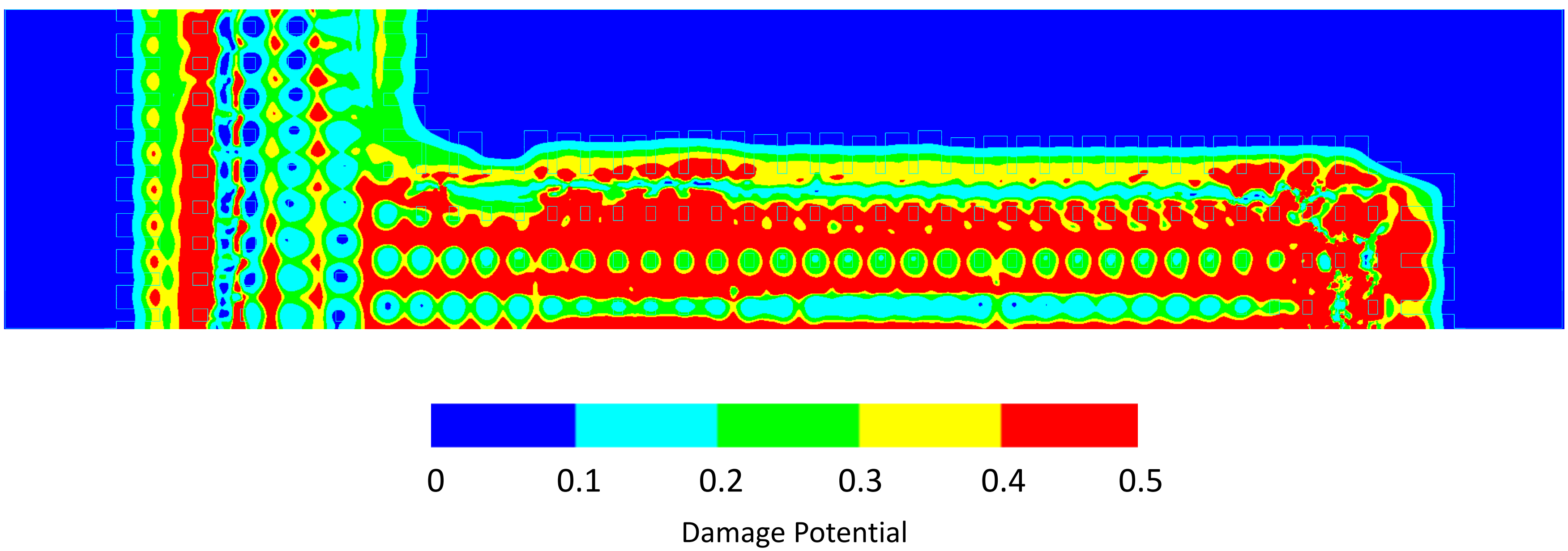

Figure 20b. Damage Potential 30 Years after Flooding, No. 5A Salt, Flooded Model

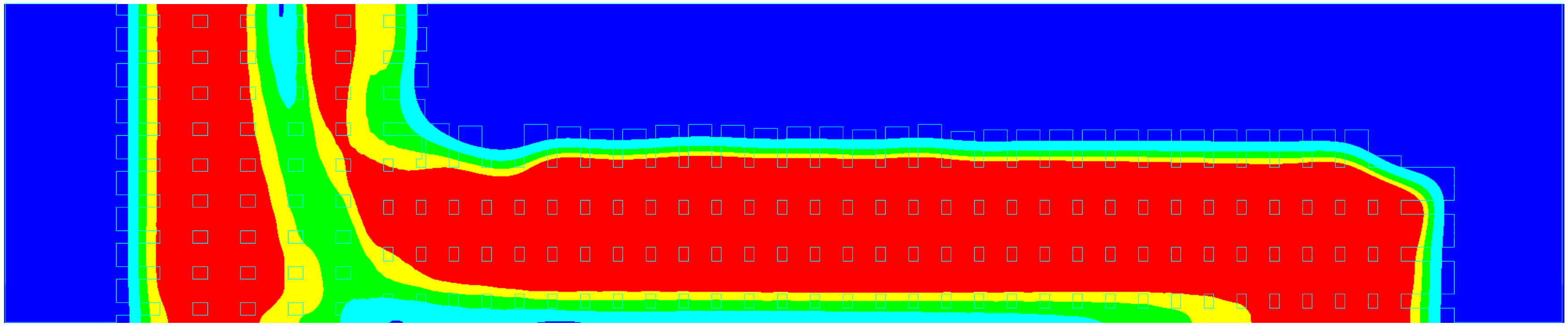

Figure 21a. Damage Potential 30 Years after Flooding, No. 5 Salt, Base Model

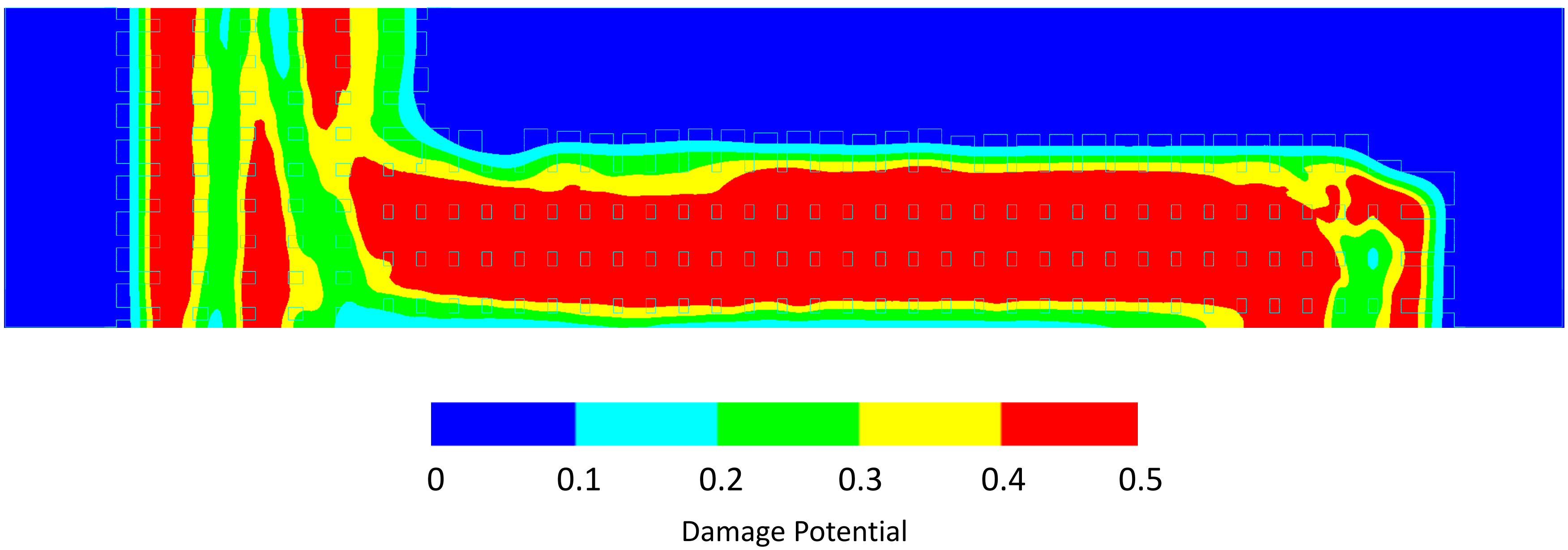

Figure 21b. Damage Potential 30 Years after Flooding, No. 5 Salt, Flooded Model

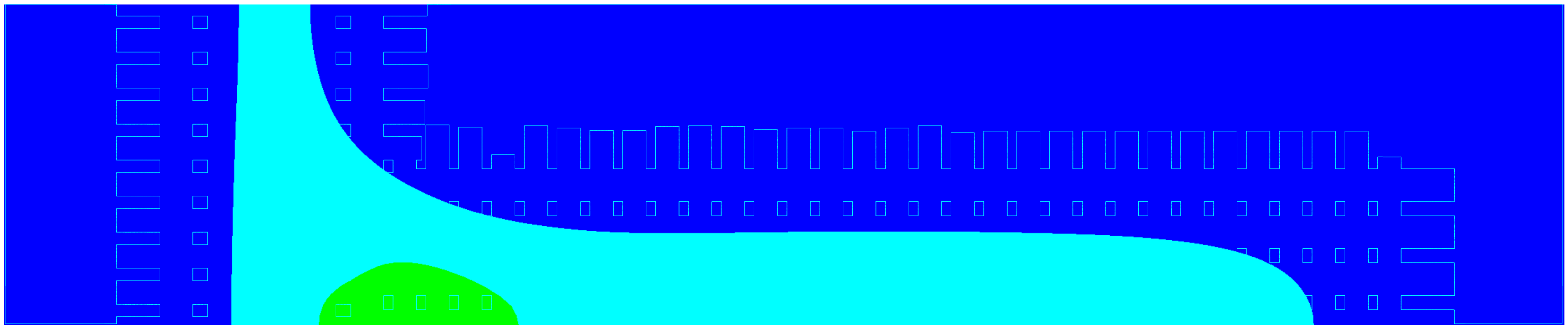

Figure 22a. Damage Potential 30 Years after Flooding, No. 4A Salt, Base Model

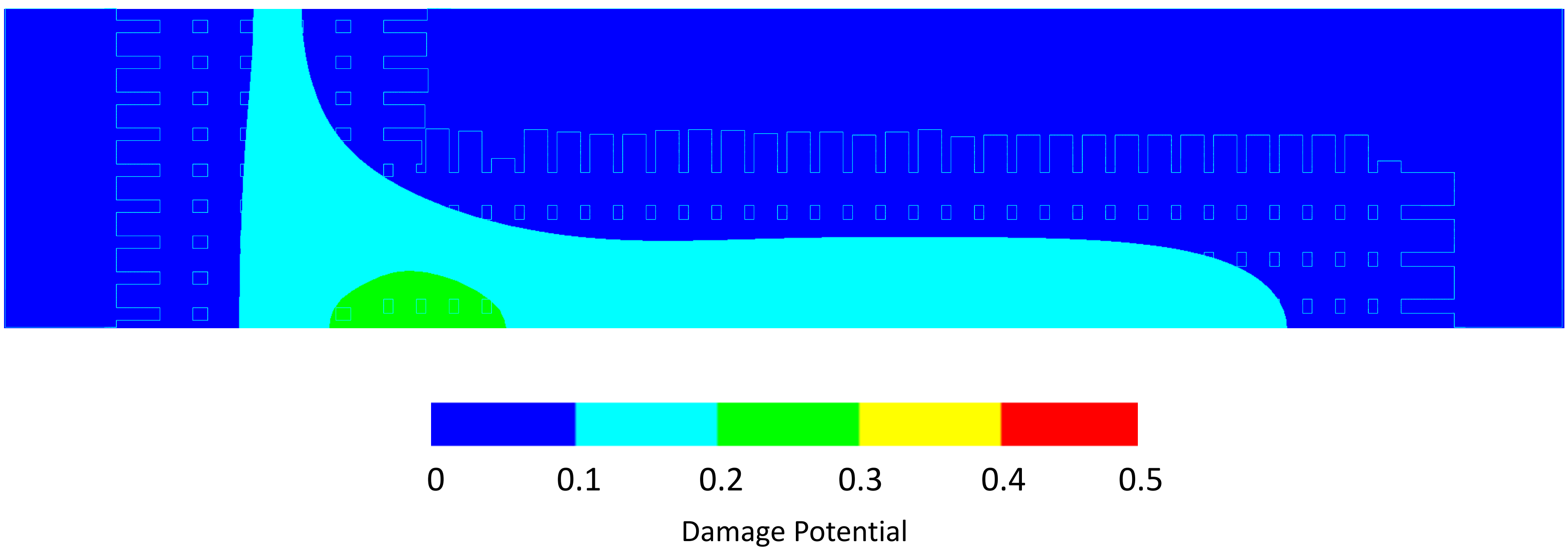

Figure 22b. Damage Potential 30 Years after Flooding, No. 4A Salt, Flooded Model

Closure in the No. 6 Salt is shown over time in Figure 23. Maximum closure at the center of the E5 Panel is in excess of 3.5 ft 30 years after flooding, and again slightly less closure is shown in the flooded model compared to the base model (Figure 23b). It should be noted that this degree of closure is much greater than historically observed at the Cayuga Mine. The closure in the model is driven by the creep parameters supplied by Dr. Mohanty and Cargill, and no attempt was made to calibrate the model to observed closure or closure rates. The value of closure and closure rate plots presented in this study is therefore in comparing values between the base and flooded models. As closure in the models is greater than that observed historically, the model is conservative regarding the impacts of strata movement.

Closure rate is shown over time for the flooded model in Figure 24. Closure rates in excess of 0.6 inches per year are seen at the panel center 1 year after flooding, decreasing to over 0.4 inches per year after 30 years. Plan view closure rate was not calculated for the base model, but is expected to be very similar to the flooded models, with slightly higher closure rates. This is borne out in the discussion below.

Further examination of closure and closure rate was made by plotting these parameters at various monitoring points in the base and flooded models. Monitoring point locations are shown in Figure 25. Convergence over the 50 years of modeled creep is shown at the monitoring points in Figure 26. Close examination shows slightly less convergence in the flooded model. To better illustrate this, the base and flooded model convergence for the point with greatest convergence (P1) are shown on the same plot in Figure 27. The convergence in the flooded model is very slightly less beginning in year 21 (1 year after flooding). Figure 28 shows the closure rate at monitoring point P1. The rates are nearly identical.

Finally, Agapito was asked to plot hydraulic potential, defined by Cargill as the depth below the Cayuga Lake surface expressed as pounds per square inch of water head minus the vertical stress at a given point in the model. Hydraulic potential is shown for years 1, 5, 15, and 30 after flooding in Figure 29 parts (a) through (d). Theoretically, the potential for any water present in the overburden to migrate to the mine workings increases as the hydraulic potential becomes more positive. As shown, the hydraulic potential turns positive about 500 ft above the No. 6 Salt, with a zone in excess of 750 psi about 200 ft above the mine workings. Agapito is not familiar with the hydrogeology of the Cayuga Mine, and details of hydraulic potential and its implications were beyond the scope of the study. From our discussions with Dr. Mohanty, we understand that the closest aquifer is about 1,200 ft above the No. 6 Salt, so the figures indicate that the potential for paths between the aquifer and the mine workings is low. As is the trend with other parameters examined in this study, the hydraulic potential increases over time with salt creep, and the flooded model has a slightly lower hydraulic potential than the base model.

4 CONCLUDING REMARKS

The FLAC3D modeling performed in the project is intended to be used as a comparison to similar modeling previously performed for Cargill by RESPEC. The Agapito flooded model includes an alternative approach to simulate potential claystone roof deterioration. Using this alternative approach, the roof deteriorates in the flooded model over mine openings but is intact and stable over pillars. Therefore, overburden loads are transmitted through the pillars to the floor

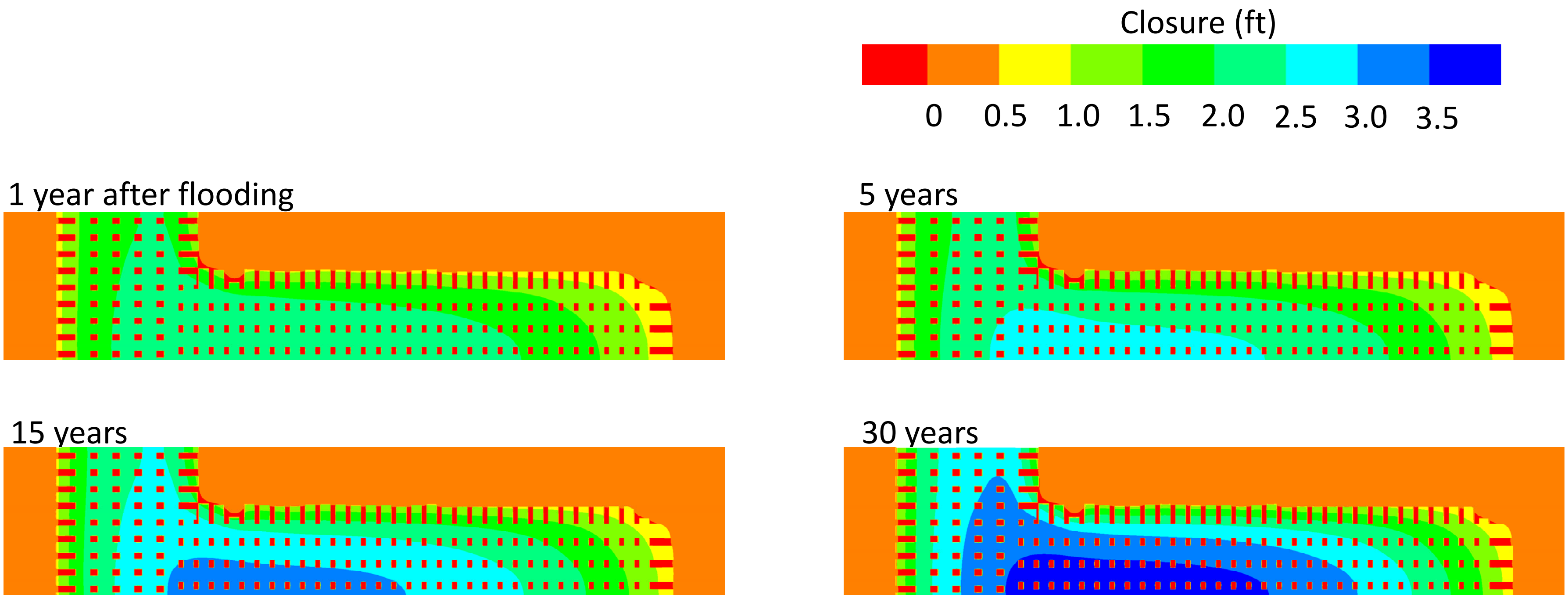

Figure 23a. No. 6 Seam Convergence, Base Model

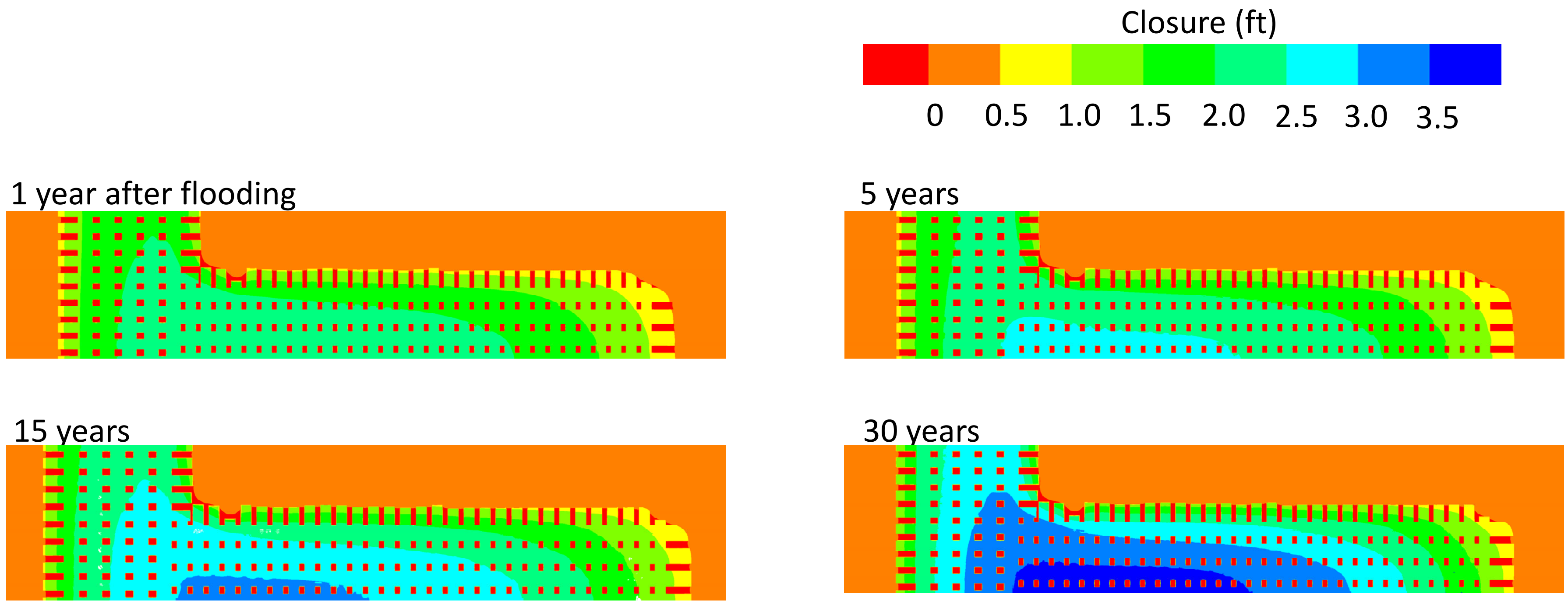

Figure 23b. No. 6 Seam Convergence, Flooded Model

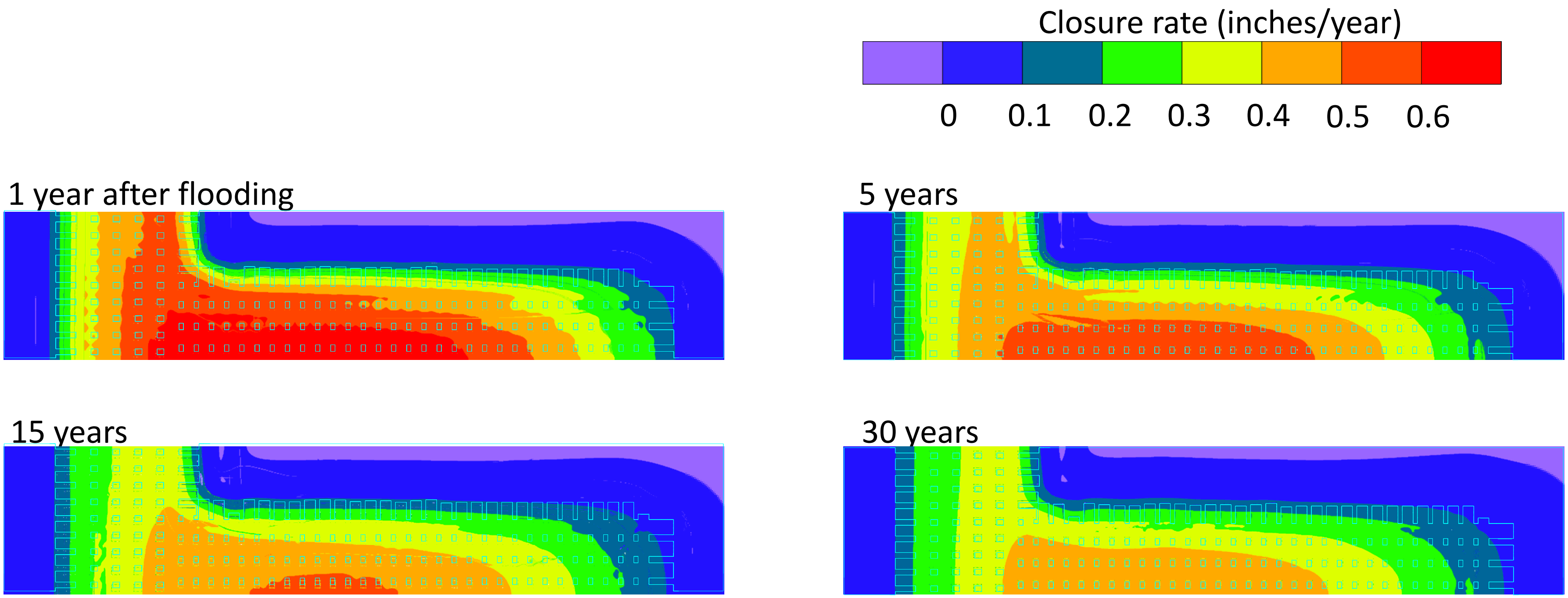

Figure 24. No. 6 Seam Convergence Rate, Flooded Model

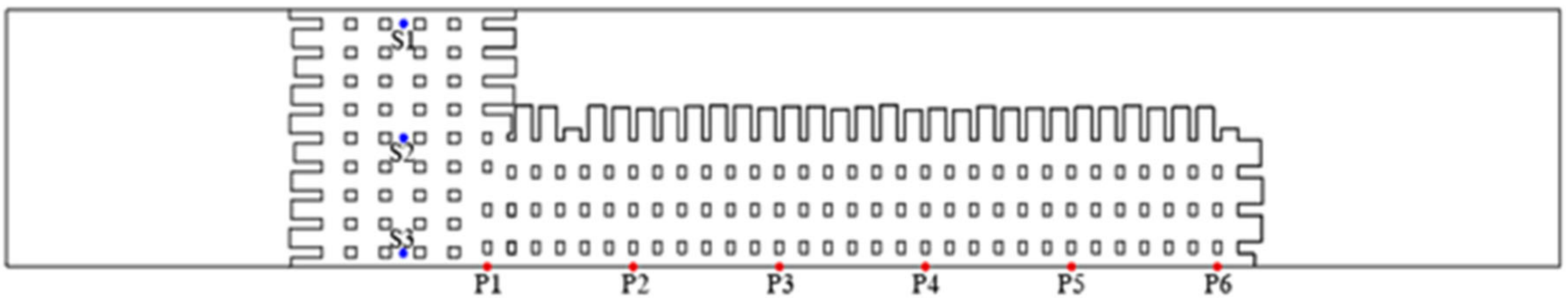

Figure 25. Monitoring Point Locations

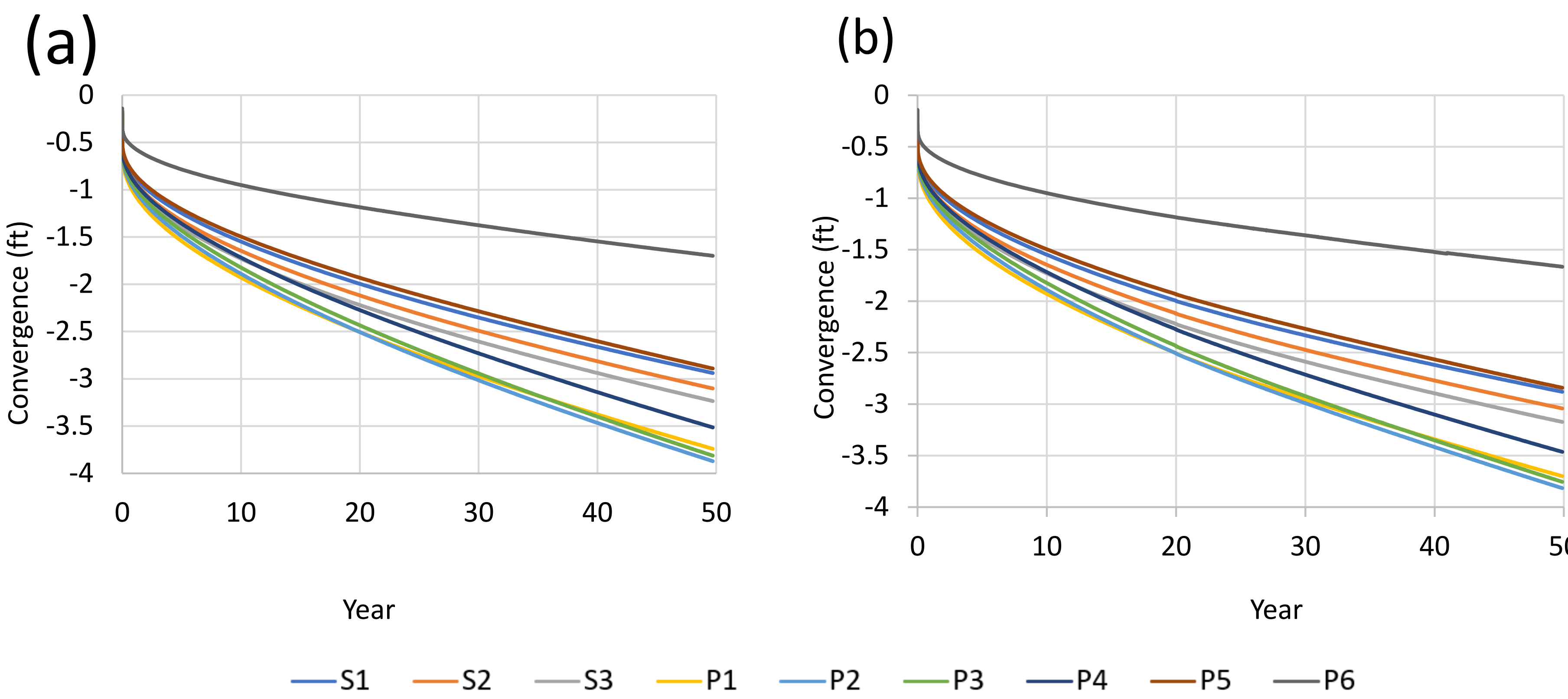

Figure 26. Convergence versus Time (a) Base Model (b) Flooded Model

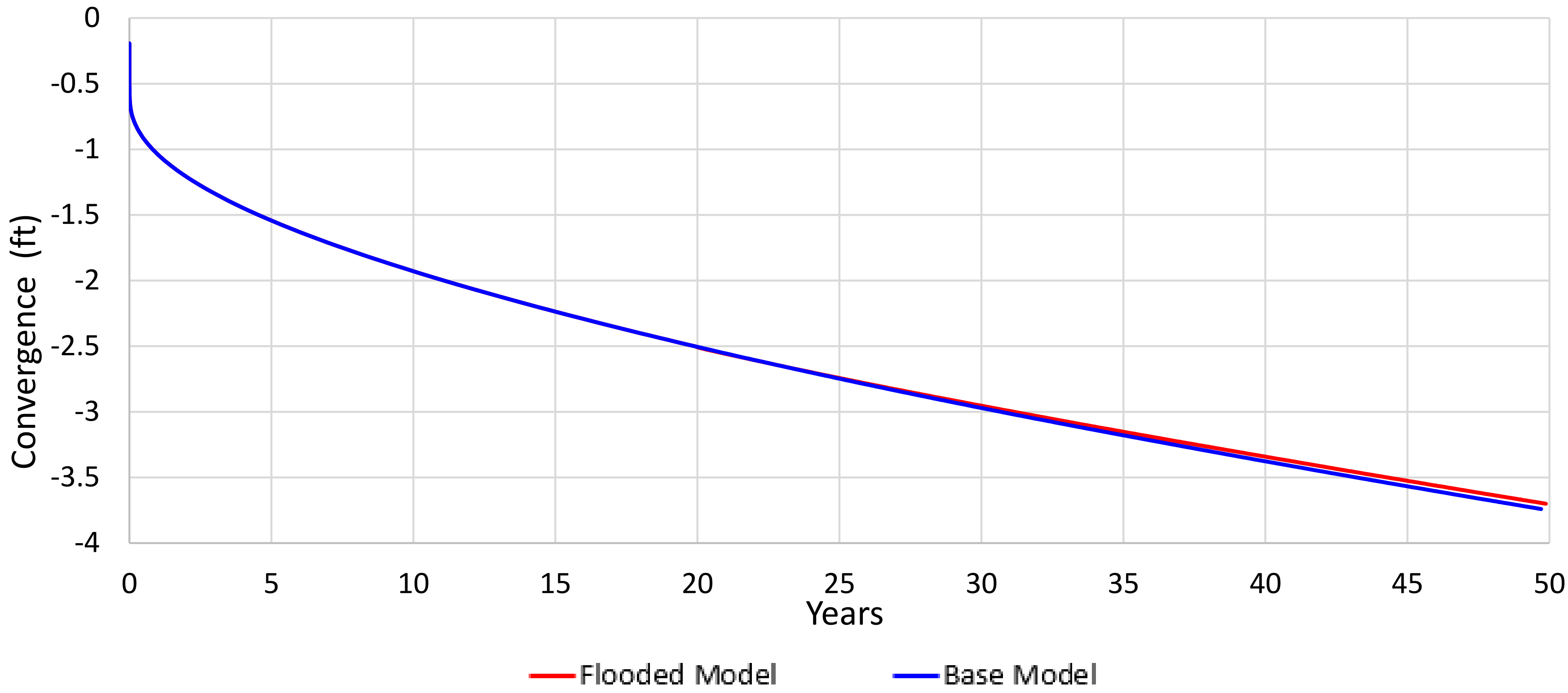

Figure 27. Convergence versus Time for Monitoring Point P1

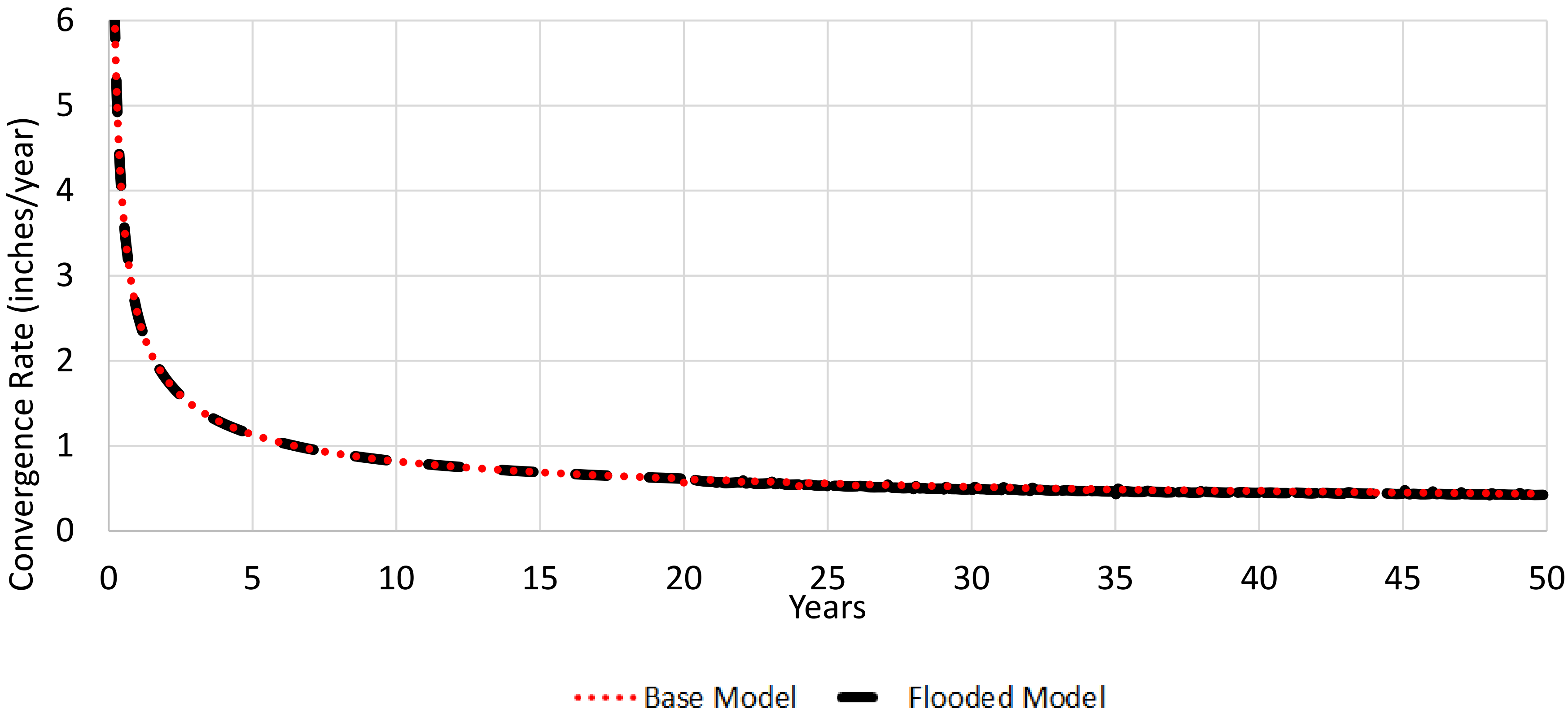

Figure 28. Convergence Rate versus Time for Monitoring Point P1

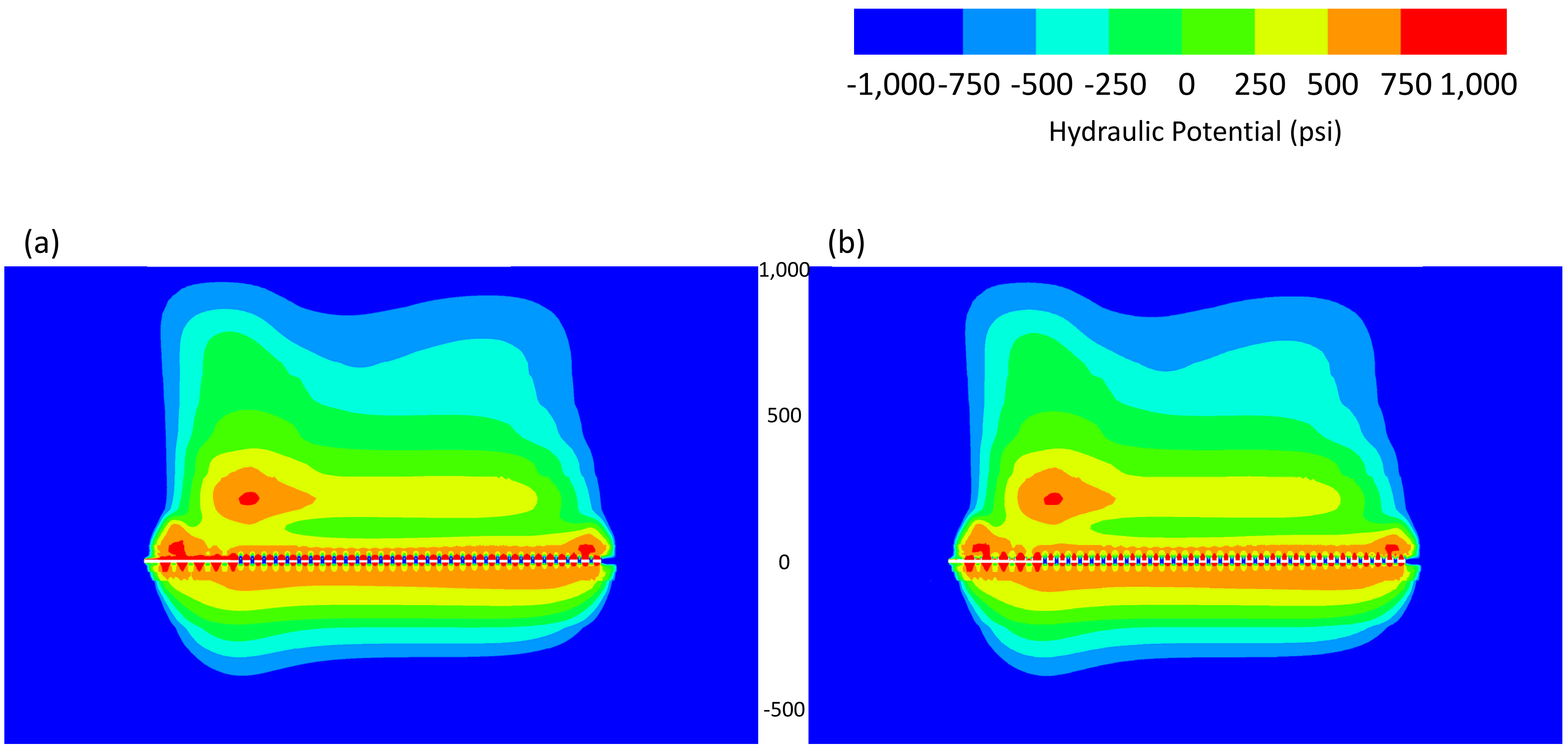

Figure 29a. Hydraulic Potential 1 Year after Flooding (a) Base Model (b) Flooded Model (vertical section along E5 Panel centerline)

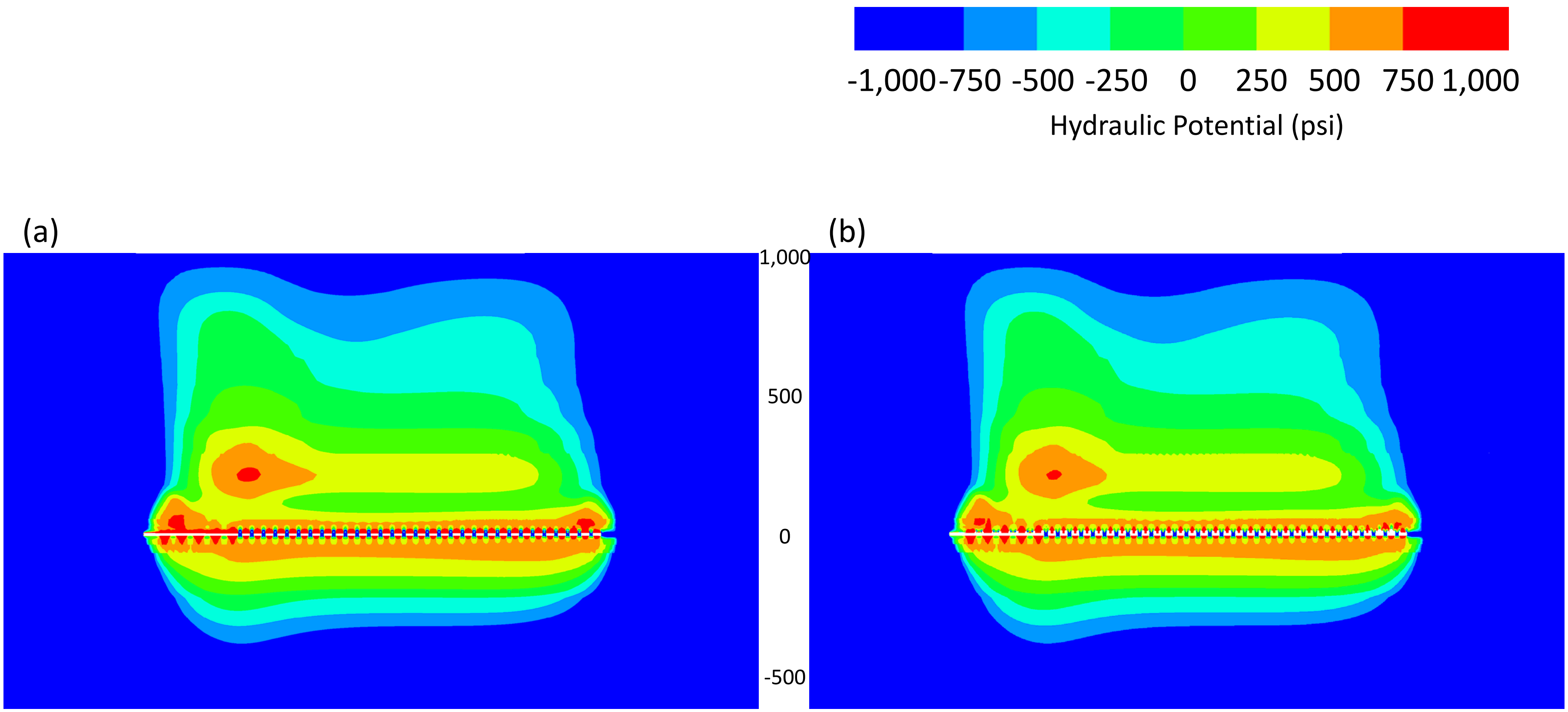

Figure 29b. Hydraulic Potential 5 Years after Flooding (a) Base Model (b) Flooded Model (vertical section along E5 Panel centerline)

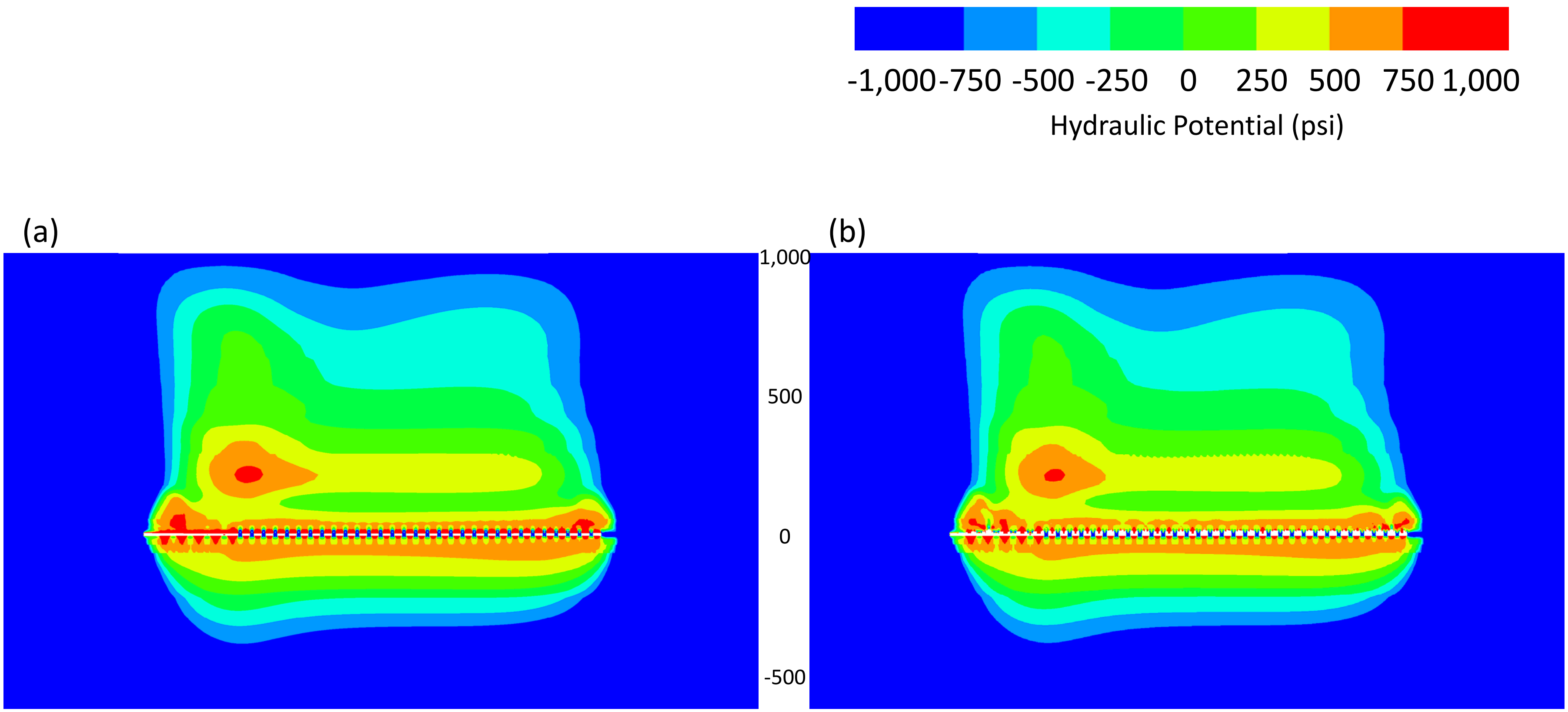

Figure 29c. Hydraulic Potential 15 Years after Flooding (a) Base Model (b) Flooded Model (vertical section along E5 Panel centerline)

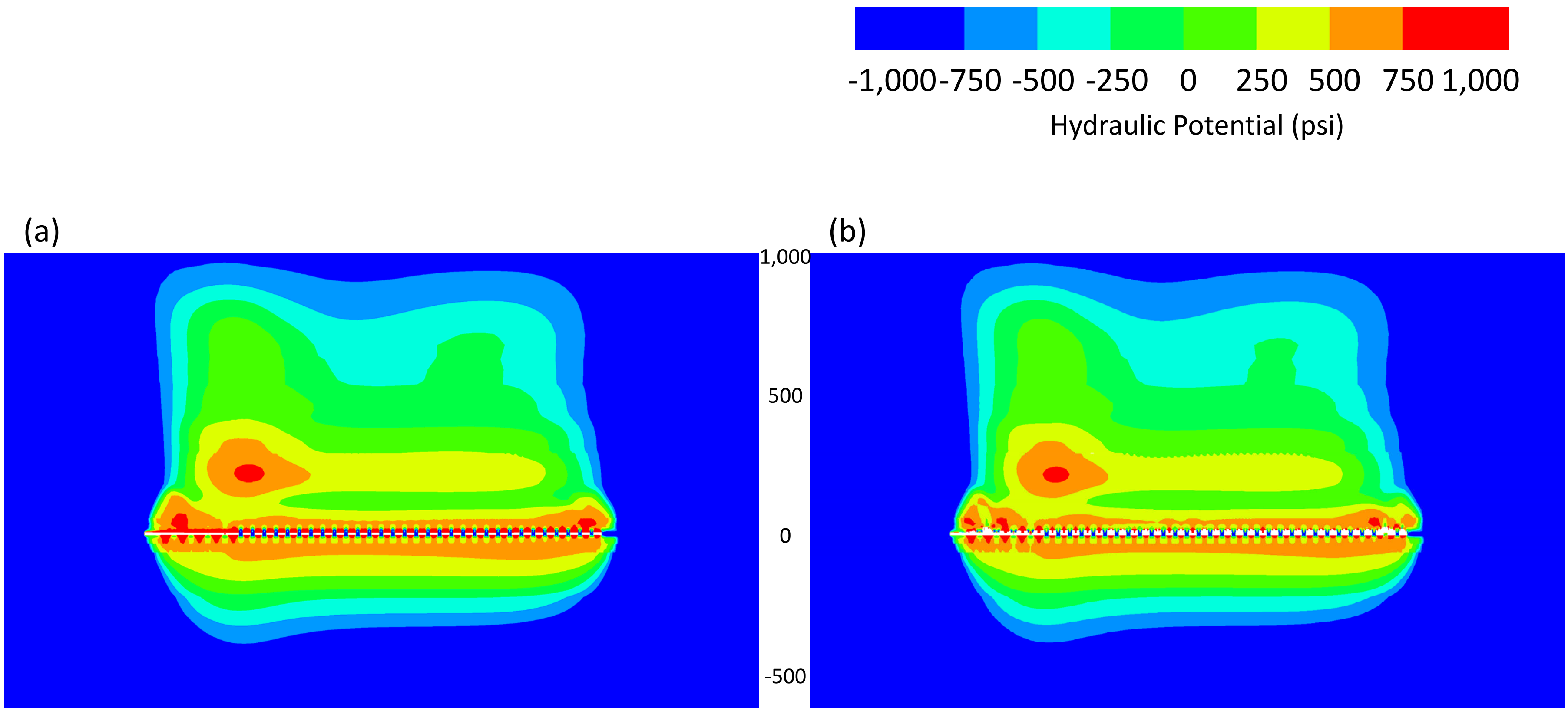

Figure 29d. Hydraulic Potential 30 Years after Flooding (a) Base Model (b) Flooded Model (vertical section along E5 Panel centerline)

in a similar manner as the base model with no roof deterioration. The model inputs were specified by Cargill to provide easy comparison with a similar RESPEC model, and as such Agapito has not calibrated the model to mining experience.

Acknowledging these limitations, the flooded model indicates that flooding the panels off the S3 Submain for water storage is not likely to give rise to global instability that could potentially cause rapid ejection of water from the panels and potential flooding of other mine areas. The results for the flooded model are very similar to the base model and show even slightly more stability due to the removal of roof weight and the slight confinement provided to the roof, pillars, and floor by the stored water.

5 References

Itasca (2013), "FLAC3D User's Manual," Itasca Consulting Group, Inc.

Merrill, R. H., and T. A. Morgan (1958), "Method of Determining the Strength of a Mine Roof," USBM RI 5406, 25 pp.

Molinda, G. M., D. C. Oyler, and H. Gurgenli (2006), "Identifying Moisture Sensitive Roof Rocks in Coal Mines," *Proceedings, 25th International Conference on Ground Control in Mining*, Morgantown, WV, pp. 57–64.

Norton, F. H. (1929). "Creep of Steel at High Temperatures," McGraw-Hill, New York, 116 pp.

Agapito Associates, Inc.
ENGINEERS & GEOLOGISTS

www.agapito.com

December 8, 2022 1043-01

Dr. Samrat Mohanty
Cargill Salt
2400 Ships Channel
Cleveland, OH 44113

Re: **Comment on the Potential Geomechanical Impacts Associated with Flooding S3 Submains on the Neighboring U12 Panel**

Dear Dr. Mohanty:

This letter is an addendum to Agapito Associates, Inc. (AAI) report 1043-01[1], which provided the results of a stability analysis undertaken on S3 Submains and E5 Panel at the Cayuga Mine. The analysis assessed the effects of the planned flooding on the global stability of the S3 Submains through numerical modelling of the E5 Panel. This addendum addresses two queries from the New York Department of Environmental Conservation (NYDEC) with regard to the conclusions in AAI's report. The two items are as follows:

1. Based on modelling results of S3 Submains and E5 Panel, are there any geomechanical impacts of storing water on the neighboring panels with a history of high ground convergence, particularly U12 Panel which is located north of S3 Submains.
2. Comment on the impacts of increased humidity (if any) from the introduction of brine in S3 Submains on ground convergence in U12 Panel.

In regard to Item 1, AAI do not anticipate any significant geomechanical impacts on any parts of U12 Panel associated with storing water in S3 Submains. The rational in regard this assessment are as follows:

- The modeling results indicated a slight increase in global stability of the S3 mine workings after flooding. This is mainly attributed to the removal of roof weight during flooding and the slight confinement provided to the roof, pillars, and floor by the stored water.
- The modelling results also indicated that the mining induced stresses return to virgin stress conditions approximately 300 feet (ft) from the edge of the mine workings. This

[1] Agapito Associates, Inc. (2021). FLAC3D Stability Analysis of S3 Submains and E5 Panel, Cayuga Mine. Report 1043-01.

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therefore suggests that any mining areas located greater than 300 ft from an adjacent stable mining area will not be subjected any significant stress surcharges.

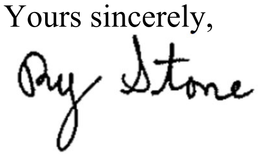
- The mine has indicated that the extent of the stored water level will be limited to the midpoint between SW2 Mains and E3 Panel, an approximate distance of 1,500 ft from the nearest point of U12 Panel. AAI understands that the mine workings in S3 Submains dip away from U12 Panel.

In regard to Item 2, AAI agree with Cargill's approach to monitor for any significant impacts that any increased levels of humidity from storing water in S3 Submains will have on ground conditions in U12 Panel. In arriving at this agreement, AAI have considered the following points:

- The workings in S3 Submains will be flooded in a gradual and controlled manner over approximately 18 years starting at the back of the workings. This will allow Cargill to progressively monitor the impacts, if any, of the stored water on the humidity levels in adjacent areas of the mine.
- Humidity monitoring in the U12 Panel indicates seasonal peaks of relatively high humidity and as such, Cargill believes that any increased levels of humidity from the stored water in S3 Submains will have an insignificant impact on the humidity levels beyond the levels historically measured in the panel.

Thank you for the opportunity to assist in this matter for the Cayuga Mine. We will be happy to discuss any modifications, clarifications, or questions you may have, at your convenience.

Yours sincerely,



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Senior Associate
ry.stone@agapito.com

RS:klg

Transmitted in PDF format via email to samratmohanty@cargill.com

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Kenney Geotechnical Services

Project No.:	**2021-146**	**Date:**	**10/3/2021**
Project Name:	**CARGILL LANSING**		

DIRECT SHEAR TEST OF ROCK
ASTM D5607

SAMPLE A-1

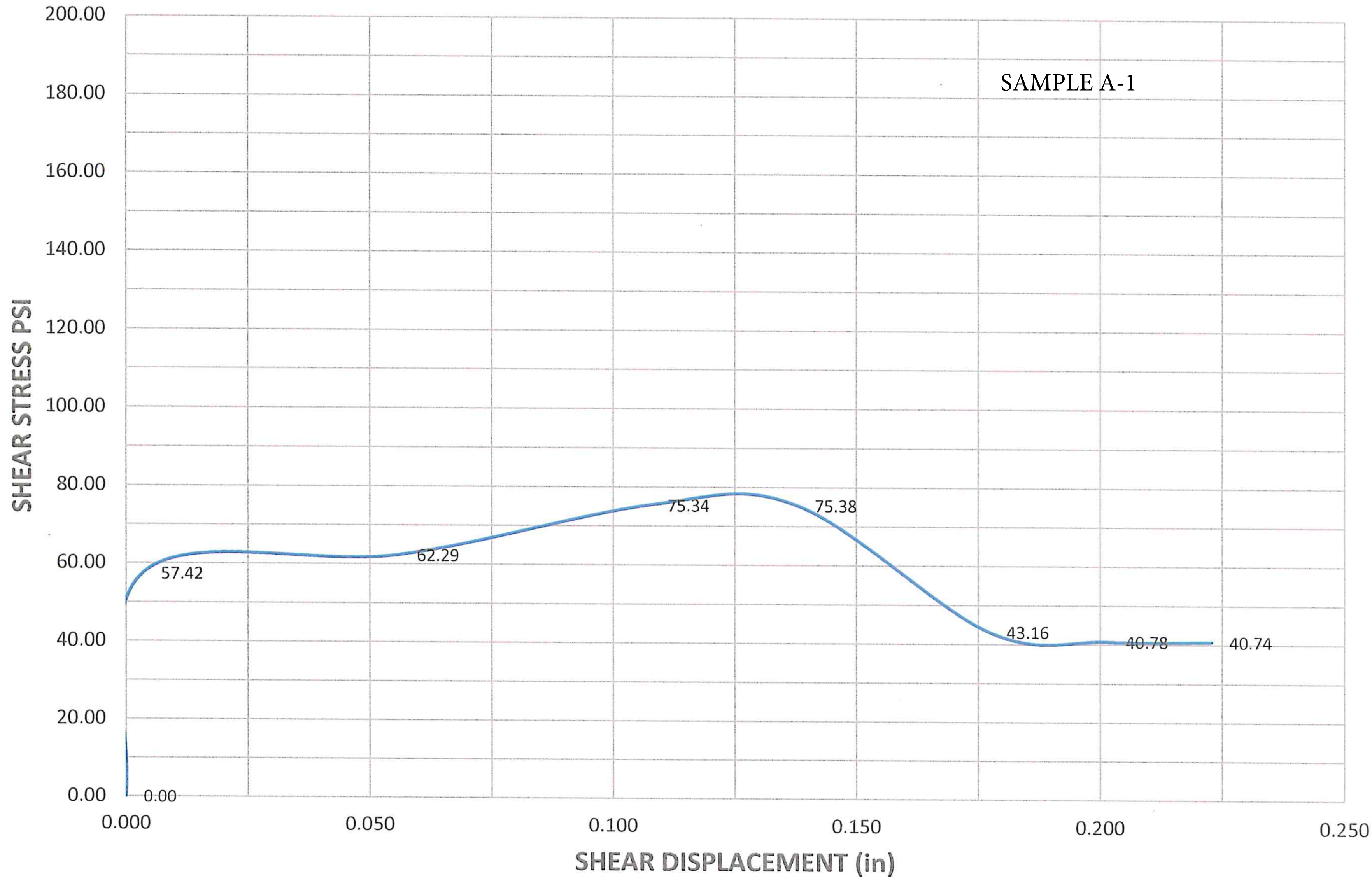
NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING TESTING

AREA:	3.14	INCHES SQ.	MOISTURE CONDITION:	DRY	JOINT ROUGHNESS:	1

NORMAL STRESS: 115.4 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.004	180.4	57.42
0.056	195.7	62.29
0.107	236.7	75.34
0.138	236.8	75.38
0.177	135.6	43.16
0.202	128.1	40.78
0.223	128	40.74

DIRECT SHEAR ASTM D-5607
Prepared By: Kenney Geotechnical Engineering Services, PLLC



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Kenney Geotechnical Services

Project No.:	**2021-146**	**Date:**	**10/3/2021**
Project Name:	CARGILL LANSING		

DIRECT SHEAR TEST OF ROCK
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SAMPLE A-2

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING SHEARING

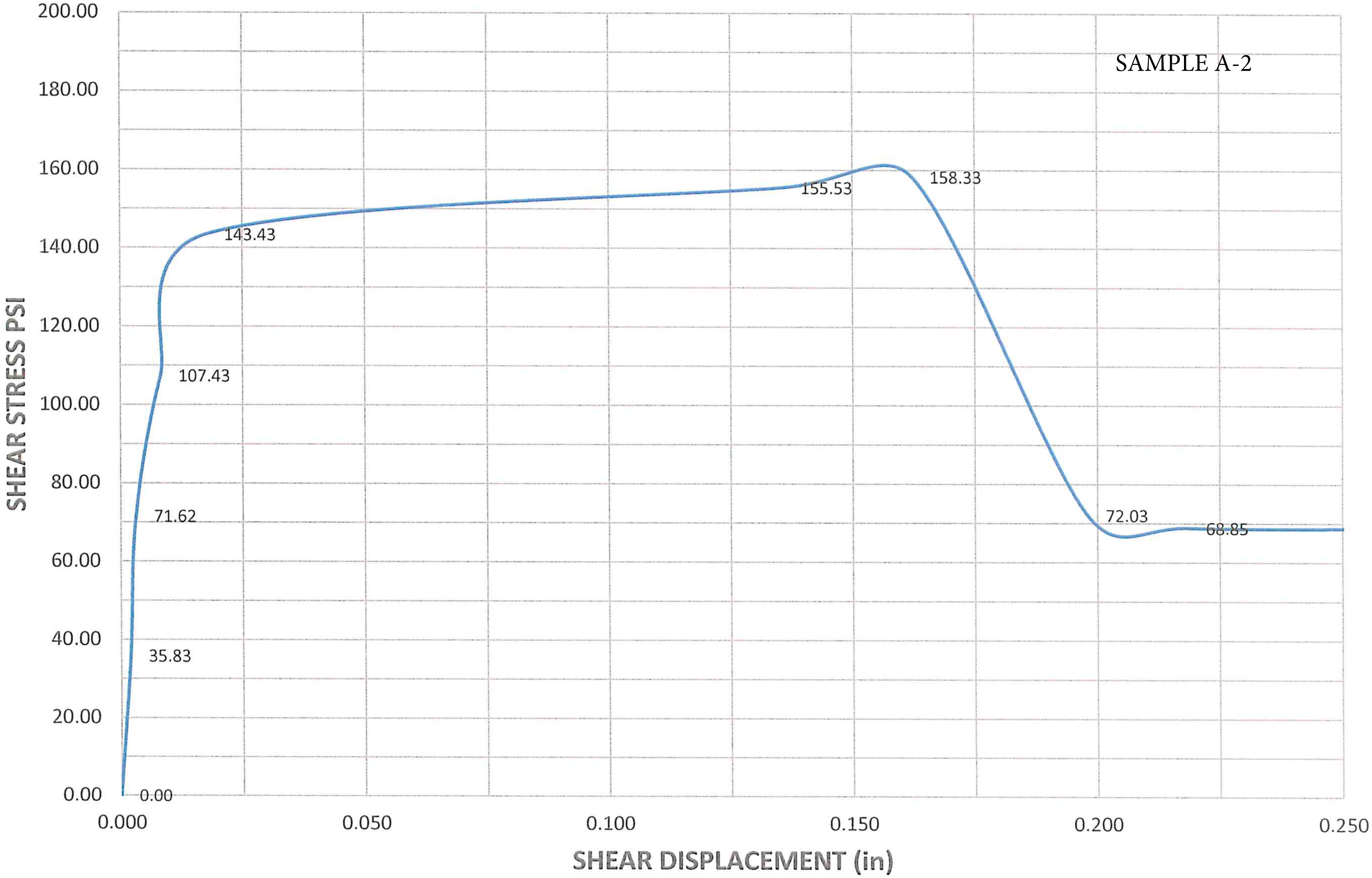
AREA: 3.14 INCHES SQ. MOISTURE CONDITION: DRY JOINT ROUGHNESS: 1

NORMAL STRESS: 260.5 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.002	112.5	35.83
0.003	225	71.62
0.008	337.5	107.43
0.018	450.6	143.43
0.136	488.6	155.53
0.162	497.4	158.33
0.198	226.3	72.03
0.22	216.3	68.85
0.28	216.2	68.82

DIRECT SHEAR ASTM D-5607

Prepared By: Kenney Geotechnical Engineering Services, PLLC



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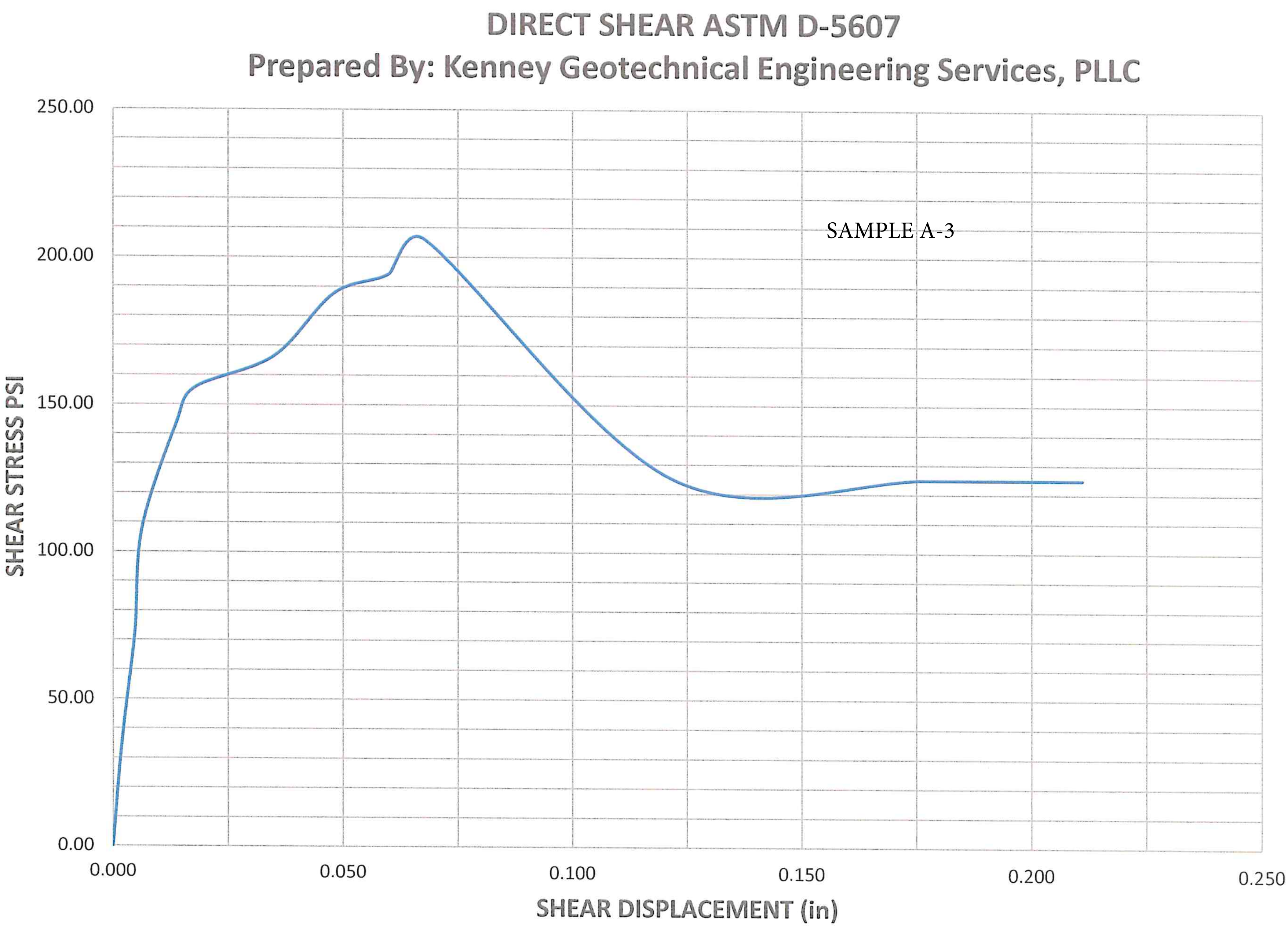
SAMPLE A-3

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING SHEARING

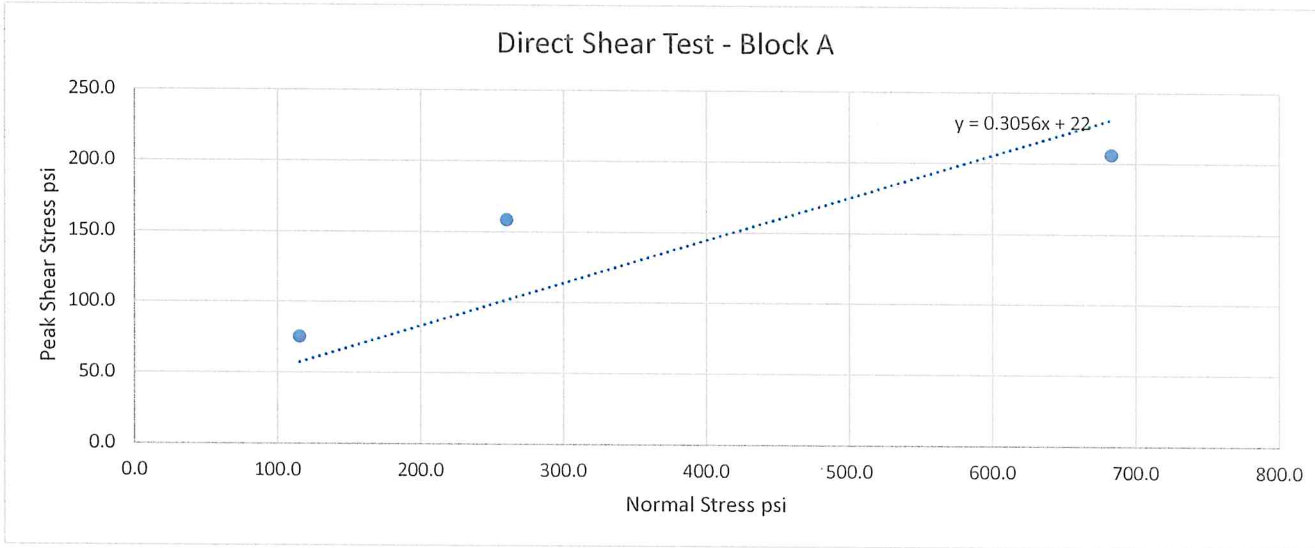
AREA: 3.14 INCHES SQ. MOISTURE CONDITION: DRY JOINT ROUGHNESS: 1

NORMAL STRESS: 683 PSI

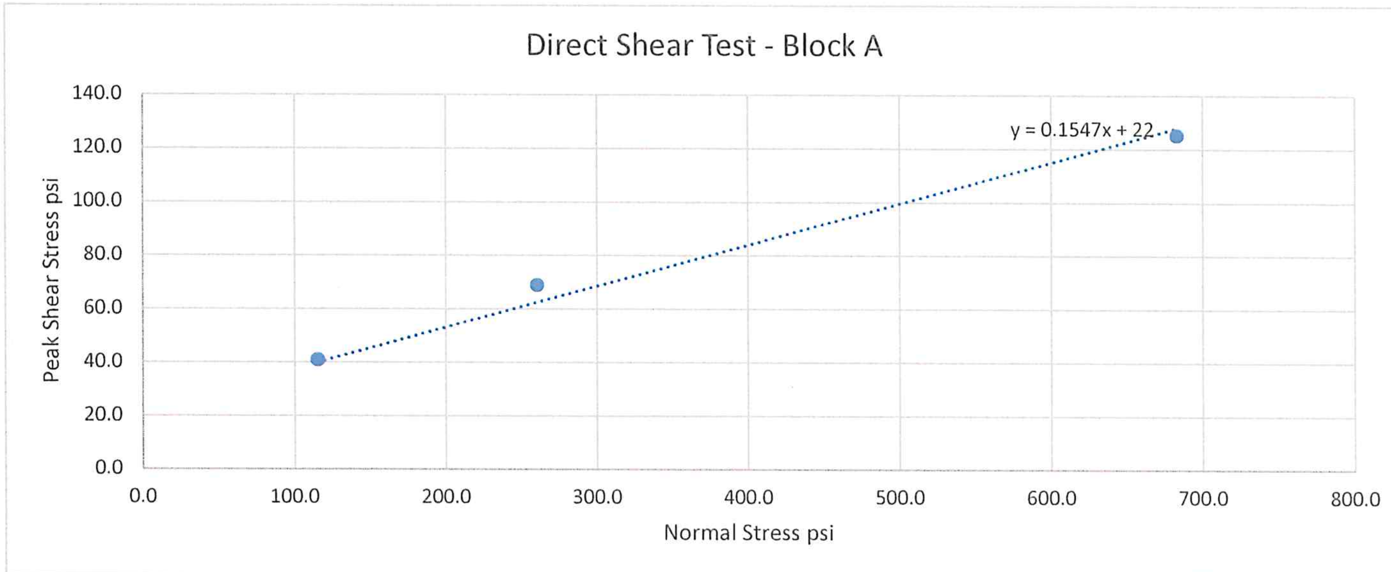
DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.002	112.5	35.83
0.005	225	71.62
0.006	337.5	107.43
0.014	450.6	143.43
0.018	488.6	155.53
0.035	522.4	166.29
0.048	589.3	187.58
0.06	610.2	194.23
0.07	647.2	206.01
0.12	392.5	124.94
0.18	392.6	124.97
0.21	392.5	124.94


DIRECT SHEAR ASTM D-5607
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SAMPLE A-3
SHEAR STRESS PSI
250.00
200.00
150.00
100.00
50.00
0.00
0.000
0.050
0.100
0.150
0.200
0.250
SHEAR DISPLACEMENT (in)

Block A	
Normal Stress psi	Peak Shear Stress psi
115.4	75.4
260.5	158.4
683.0	206.0



Block A	
Normal Stress psi	Residual Shear Stress psi
115.4	40.8
260.5	68.9
683.0	125.0



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Project No.:	**2021-146**	**Date:**	**10/3/2021**
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DIRECT SHEAR TEST OF ROCK
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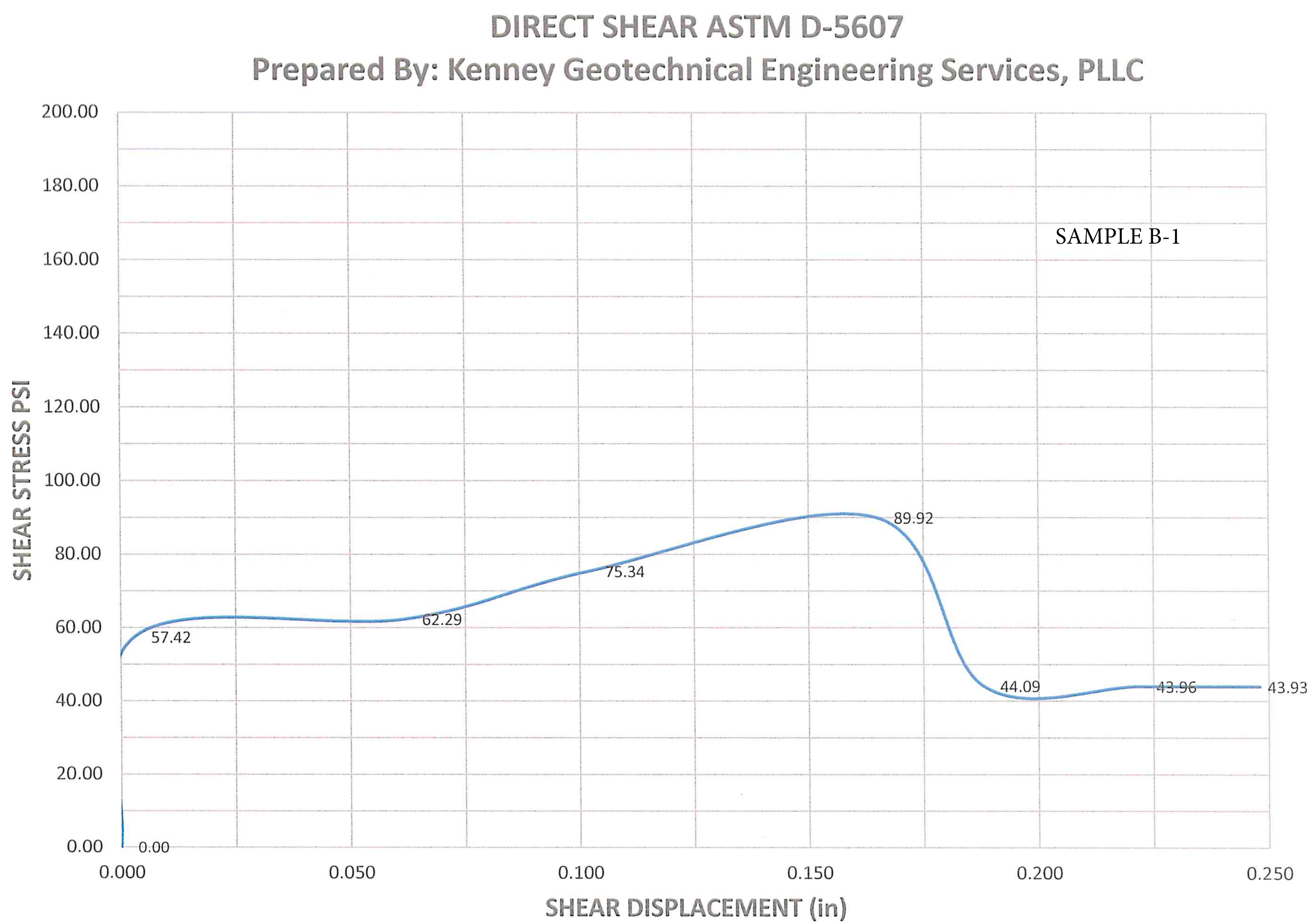
SAMPLE B-1

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING TESTING

AREA:	3.14	INCHES SQ.	MOISTURE CONDITION:	DRY	JOINT ROUGHNESS:	1	

NORMAL STRESS: 115.4 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.003	180.4	57.42
0.062	195.7	62.29
0.102	236.7	75.34
0.165	282.5	89.92
0.188	138.5	44.09
0.222	138.1	43.96
0.248	138	43.93


DIRECT SHEAR ASTM D-5607
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SAMPLE B-1
SHEAR STRESS PSI
SHEAR DISPLACEMENT (in)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.000
0.050
0.100
0.150
0.200
0.250
0.00
57.42
62.29
75.34
89.92
44.09
43.96
43.93

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Kenney Geotechnical Services

Project No.:	2021-146	Date:	10/3/2021
Project Name:	CARGILL LANSING		

DIRECT SHEAR TEST OF ROCK
ASTM D5607

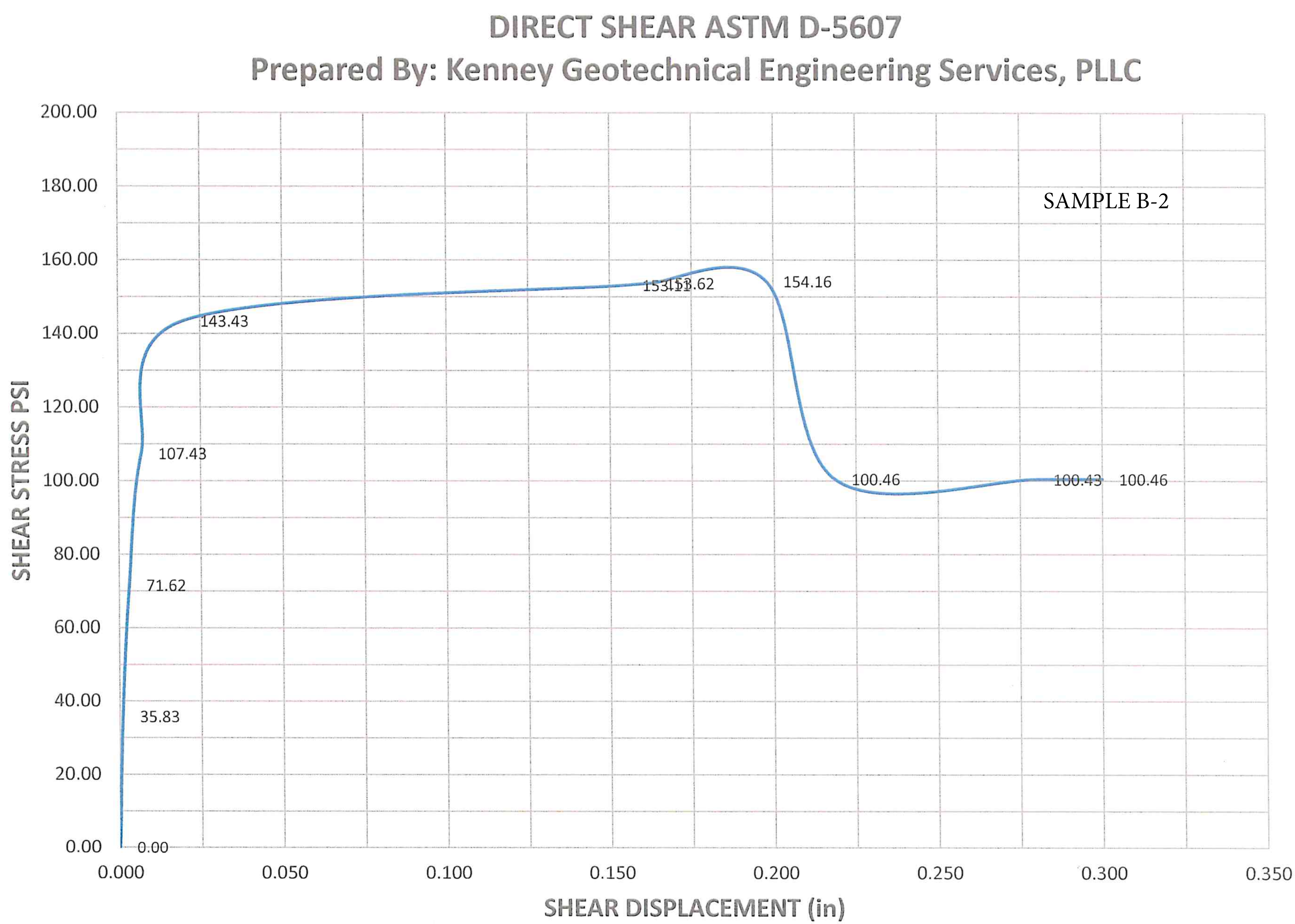
SAMPLE B-2

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING SHEARING

AREA: 3.14 INCHES SQ. MOISTURE CONDITION: DRY JOINT ROUGHNESS: 1

NORMAL STRESS: 260.5 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.001	112.5	35.83
0.003	225	71.62
0.007	337.5	107.43
0.020	450.6	143.43
0.155	481	153.11
0.162	482.6	153.62
0.198	484.3	154.16
0.22	315.6	100.46
0.28	315.5	100.43
0.30	315.6	100.46


DIRECT SHEAR ASTM D-5607
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SAMPLE B-2
SHEAR STRESS PSI
SHEAR DISPLACEMENT (in)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.00
35.83
71.62
107.43
143.43
154.16
100.46
100.43
100.46

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Kenney Geotechnical Services

Project No.:	**2021-146**	**Date:**	**10/3/2021**
Project Name:	CARGILL LANSING		

DIRECT SHEAR TEST OF ROCK
ASTM D5607

SAMPLE B-3

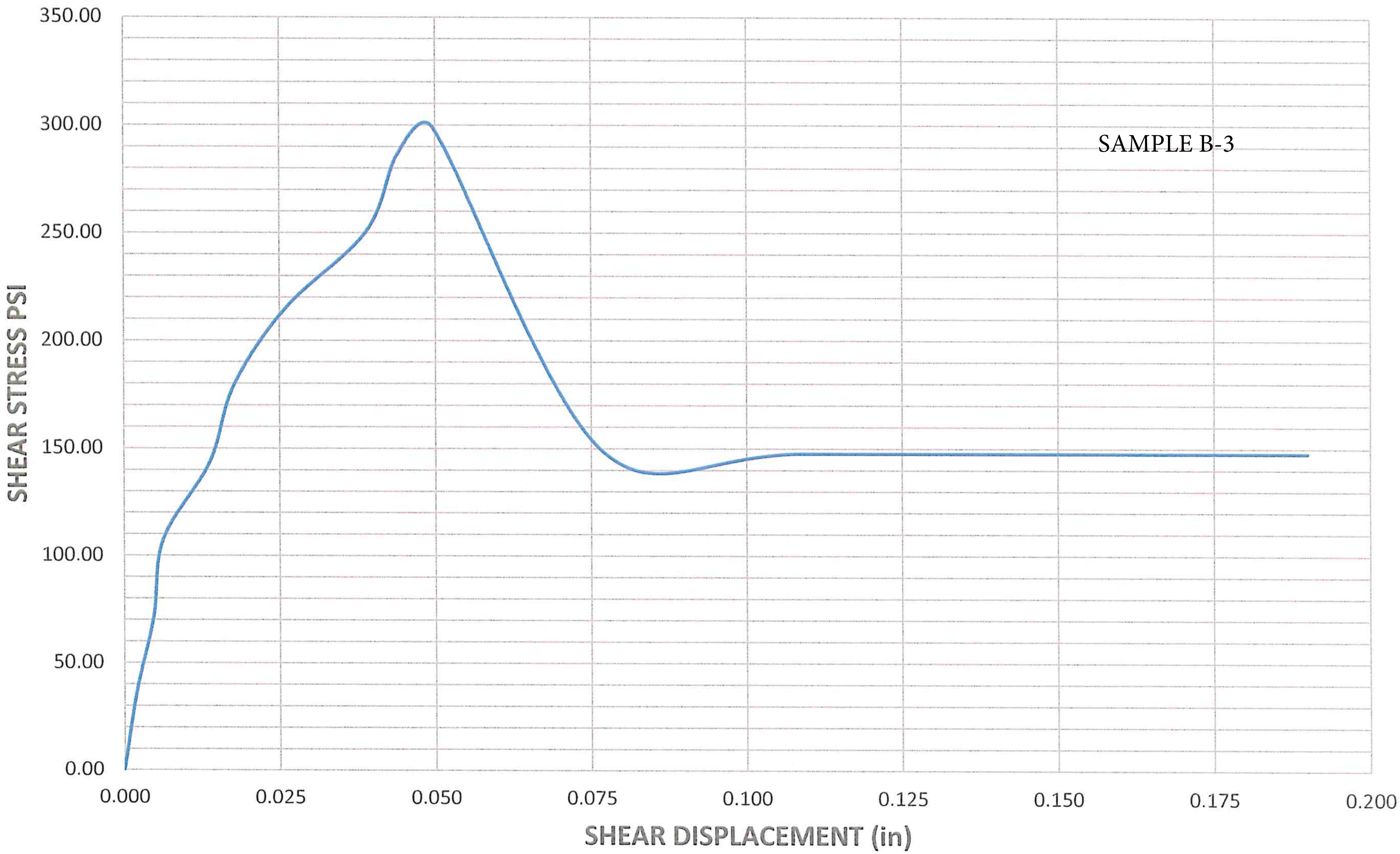
NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING SHEARING

AREA: 3.14 INCHES SQ. MOISTURE CONDITION: DRY JOINT ROUGHNESS: 1

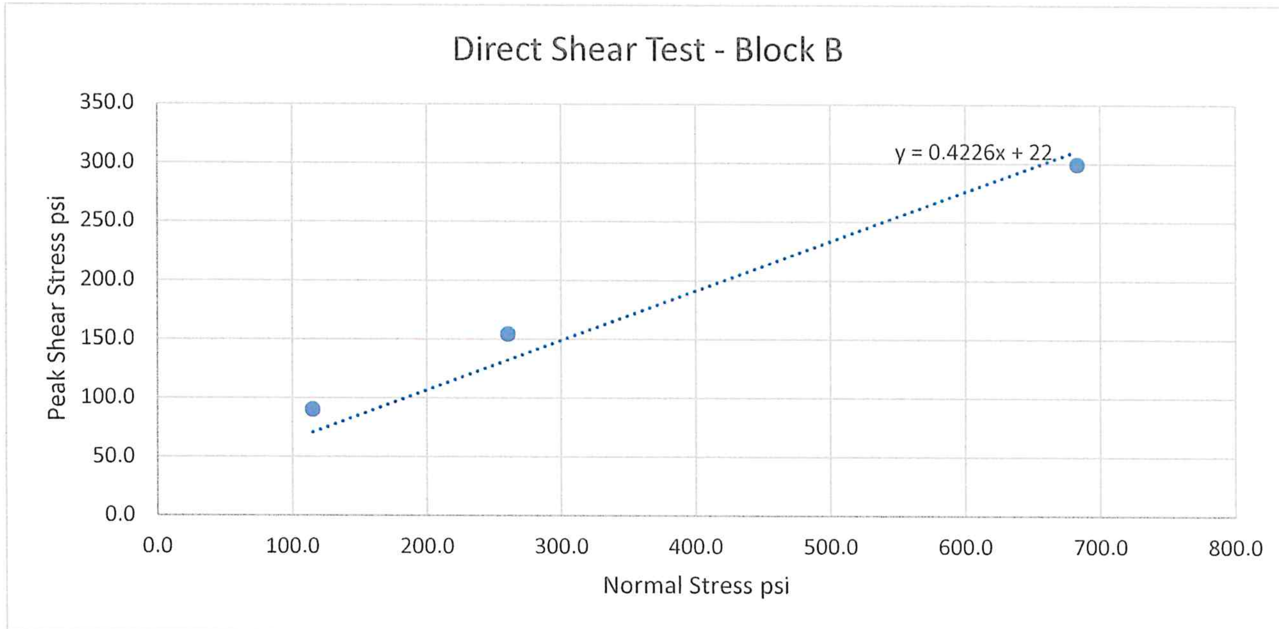
NORMAL STRESS: 683 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.002	112.5	35.83
0.005	225	71.62
0.006	337.5	107.43
0.014	450.6	143.43
0.018	563	179.21
0.026	675	214.86
0.039	787.5	250.67
0.04	900	286.48
0.05	939.3	298.99
0.08	466	148.33
0.11	464	147.70
0.19	464	147.70

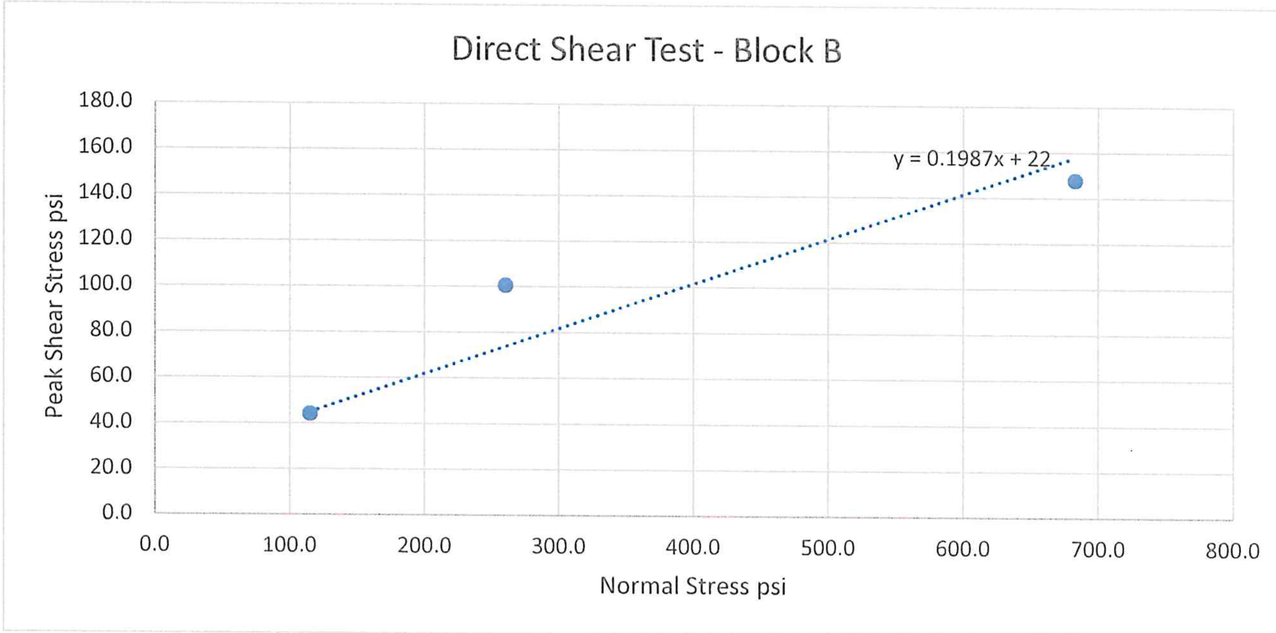
DIRECT SHEAR ASTM D-5607
Prepared By: Kenney Geotechnical Engineering Services, PLLC



Block B	
Normal Stress psi	Peak Shear Stress psi
115.4	89.9
260.5	154.2
683.0	299.0



Block B	
Normal Stress psi	Residual Shear Stress psi
115.4	44.0
260.5	100.5
683.0	147.7



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Kenney Geotechnical Services

Project No.:	**2021-146**	**Date:**	**10/3/2021**
Project Name:	CARGILL LANSING		

DIRECT SHEAR TEST OF ROCK
ASTM D5607

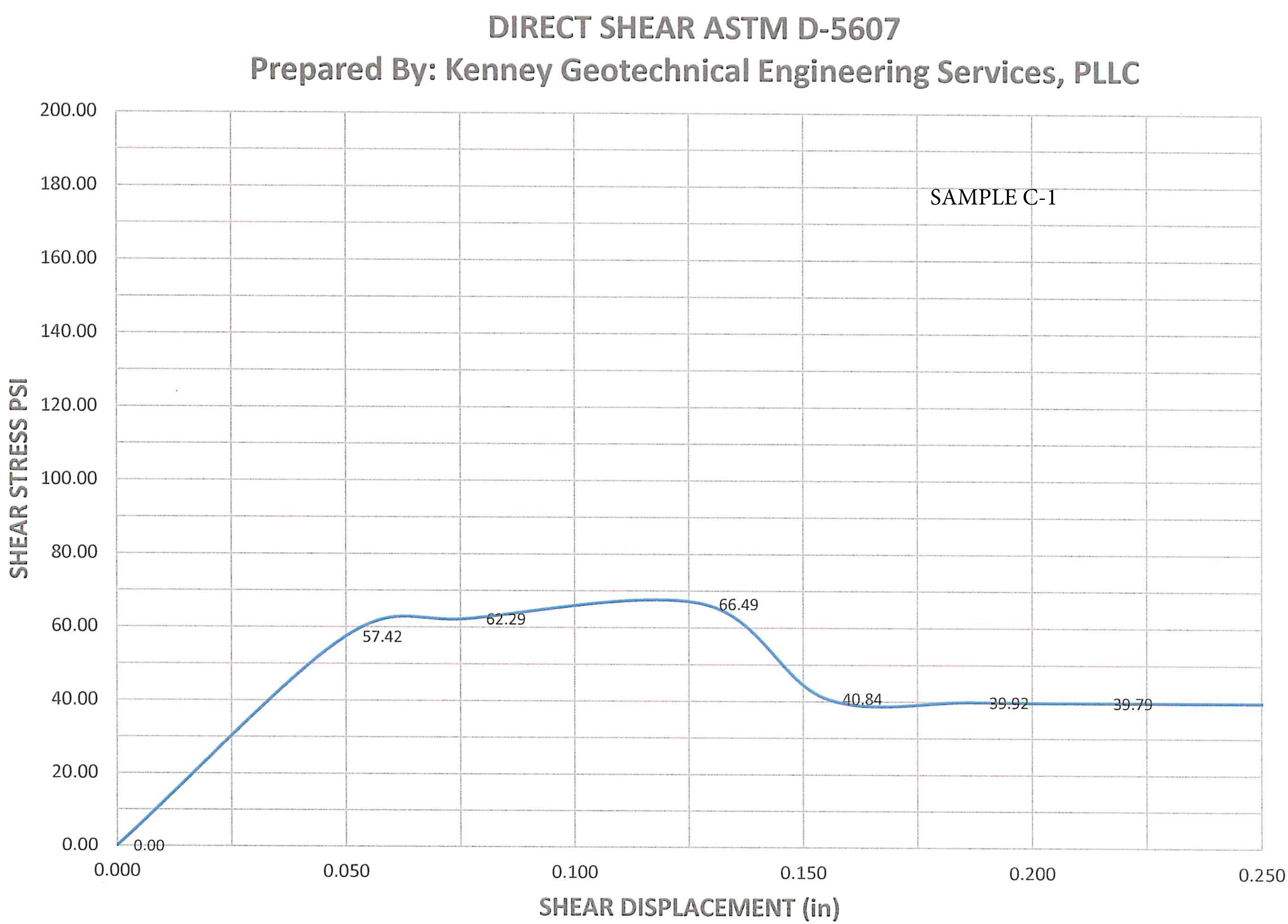
SAMPLE C-1

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING TESTING

AREA: 3.14 INCHES SQ. MOISTURE CONDITION: DRY JOINT ROUGHNESS: 1

NORMAL STRESS: 115.4 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.050	180.4	57.42
0.077	195.7	62.29
0.128	208.9	66.49
0.155	128.3	40.84
0.187	125.4	39.92
0.214	125	39.79
0.255	125	39.79


DIRECT SHEAR ASTM D-5607
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SAMPLE C-1
SHEAR STRESS PSI
SHEAR DISPLACEMENT (in)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.000
0.050
0.100
0.150
0.200
0.250
0.00
57.42
62.29
66.49
40.84
39.92
39.79

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Kenney Geotechnical Services

Project No.:	**2021-146**	**Date:**	**10/3/2021**
Project Name:		**CARGILL LANSING**	

DIRECT SHEAR TEST OF ROCK
ASTM D5607

SAMPLE C-2

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING SHEARING

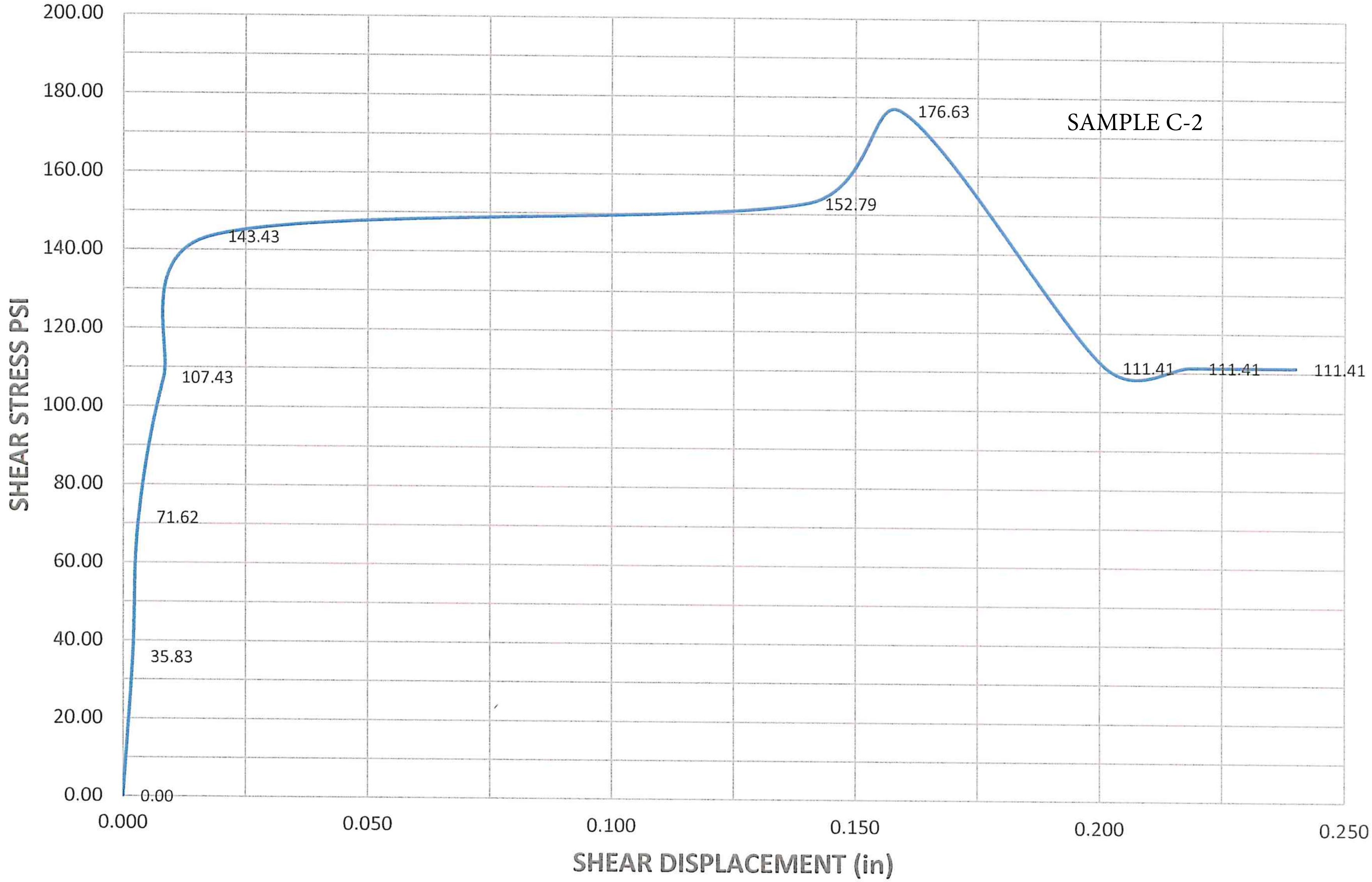
AREA:	3.14	INCHES SQ.	MOISTURE CONDITION:	DRY	JOINT ROUGHNESS:	1

NORMAL STRESS: 260.5 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.002	112.5	35.83
0.003	225	71.62
0.008	337.5	107.43
0.018	450.6	143.43
0.140	480	152.79
0.159	554.9	176.63
0.201	350	111.41
0.22	350	111.41
0.24	350	111.41

DIRECT SHEAR ASTM D-5607

Prepared By: Kenney Geotechnical Engineering Services, PLLC



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Kenney Geotechnical Services

Project No.:	**2021-146**	**Date:**	**10/3/2021**
Project Name:	**CARGILL LANSING**		

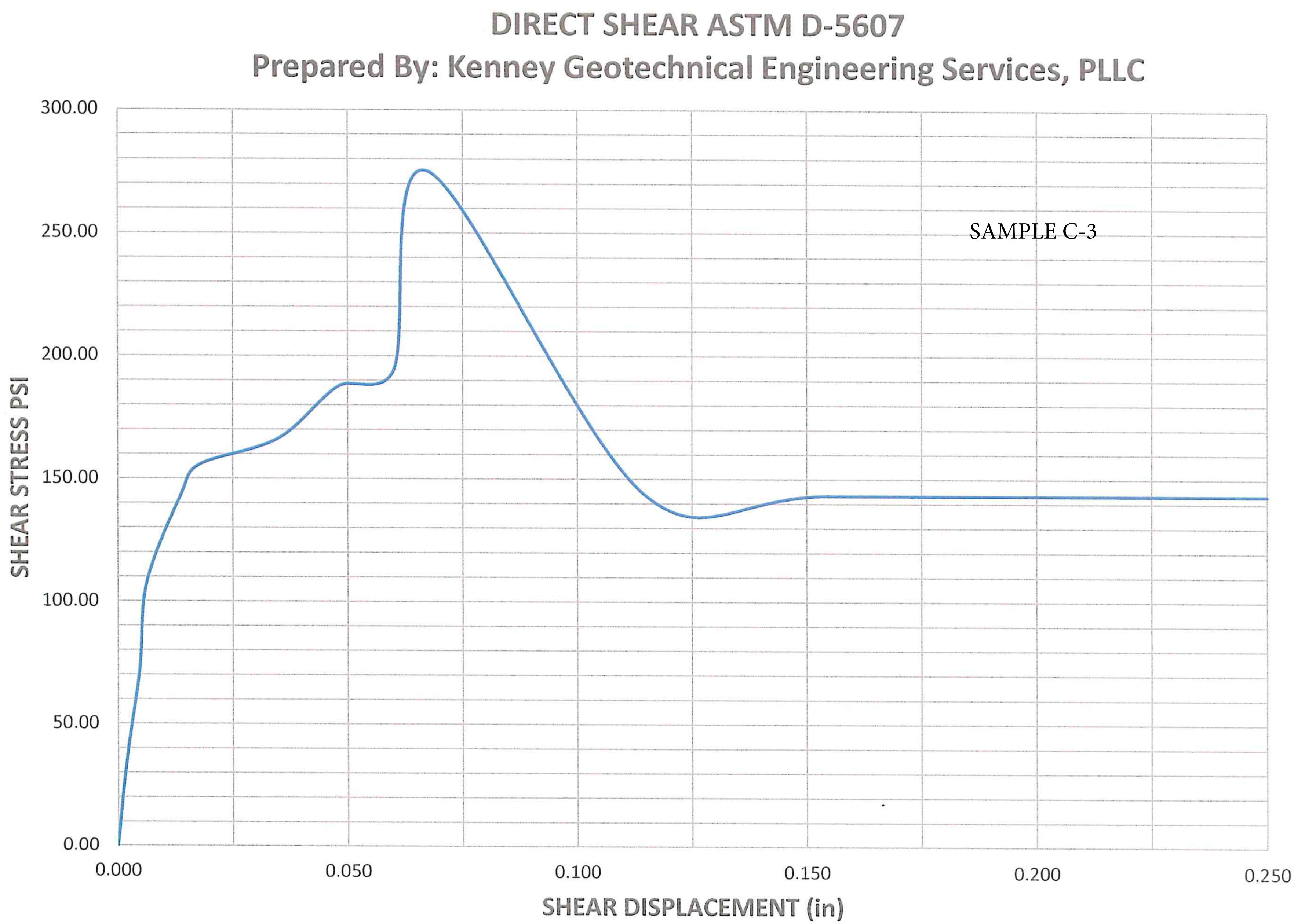
DIRECT SHEAR TEST OF ROCK
ASTM D5607

SAMPLE C-3

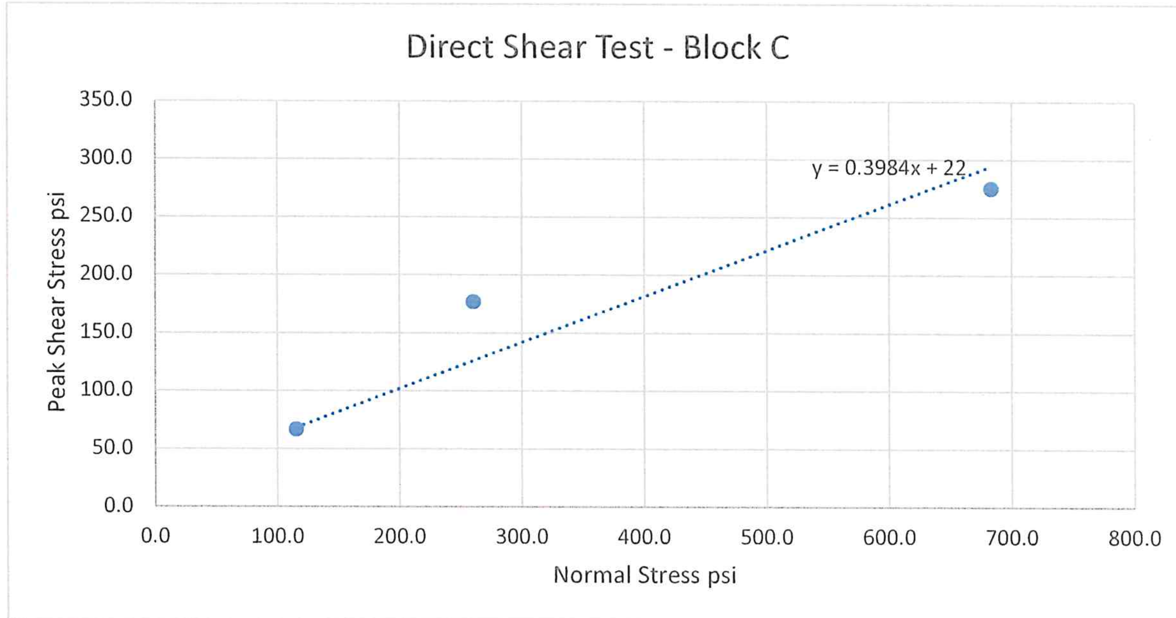
NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLE PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. 2" CORE OBTAINED FROM BLOCK. CORE INTACT UPON ENCAPULATION IN HIGH STRENGTH GYPSUM. SAMPLE SHEARED UPON APPLICATION OF SEATING LOAD (APPROX. 22 PSI). SHEAR PLANE WAS APPROXIMATELY 10 TO 15 DEGREES FROM HORIZONTAL. SHEAR PLANE CRUSHED DURING SHEARING

AREA:	3.14	INCHES SQ.	MOISTURE CONDITION:	DRY	JOINT ROUGHNESS:	1
NORMAL STRESS:	683	PSI				

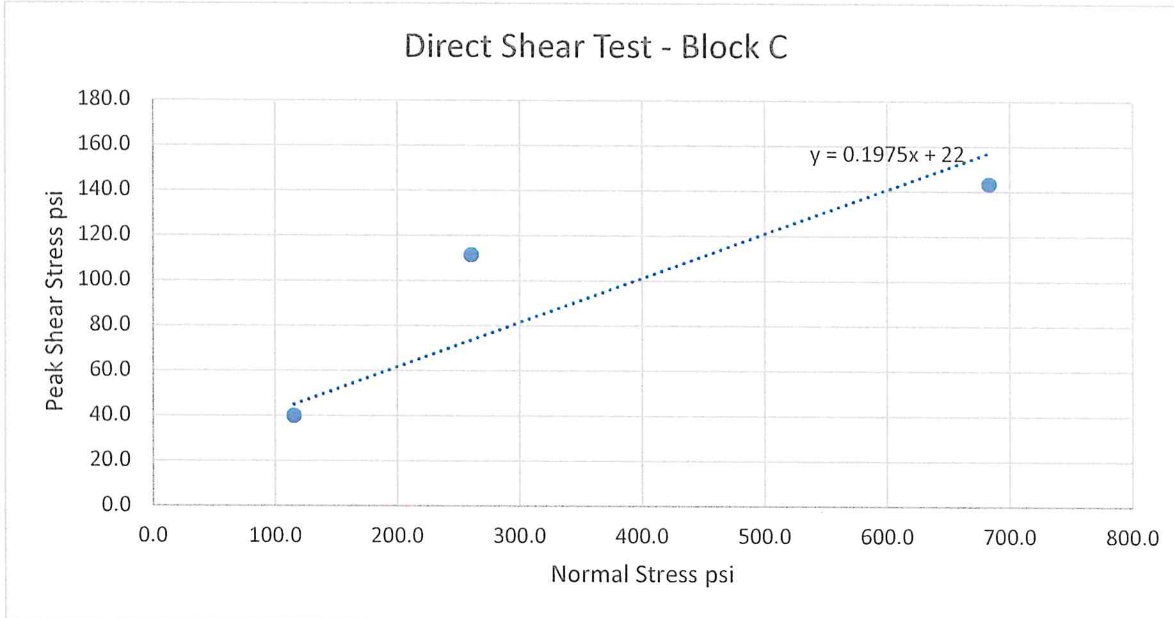
DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.002	112.5	35.83
0.005	225	71.62
0.006	337.5	107.43
0.014	450.6	143.43
0.018	488.6	155.53
0.035	522.4	166.29
0.048	589.3	187.58
0.06	610.2	194.23
0.07	863.5	274.86
0.12	450	143.24
0.16	450	143.24
0.25	450	143.24


DIRECT SHEAR ASTM D-5607
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SAMPLE C-3
300.00
250.00
200.00
150.00
100.00
50.00
0.00
SHEAR STRESS PSI
0.000
0.050
0.100
0.150
0.200
0.250
SHEAR DISPLACEMENT (in)

Block C	
Normal Stress psi	Peak Shear Stress psi
115.4	66.5
260.5	176.6
683.0	275.0



Block C	
Normal Stress psi	Residual Shear Stress psi
115.4	39.8
260.5	111.4
683.0	143.3



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Project No.:	**2021-146**	**Date:**	**10/4/2021**
Project Name:	**CARGILL LANSING**		

DIRECT SHEAR TEST OF SOIL
ASTM D3080

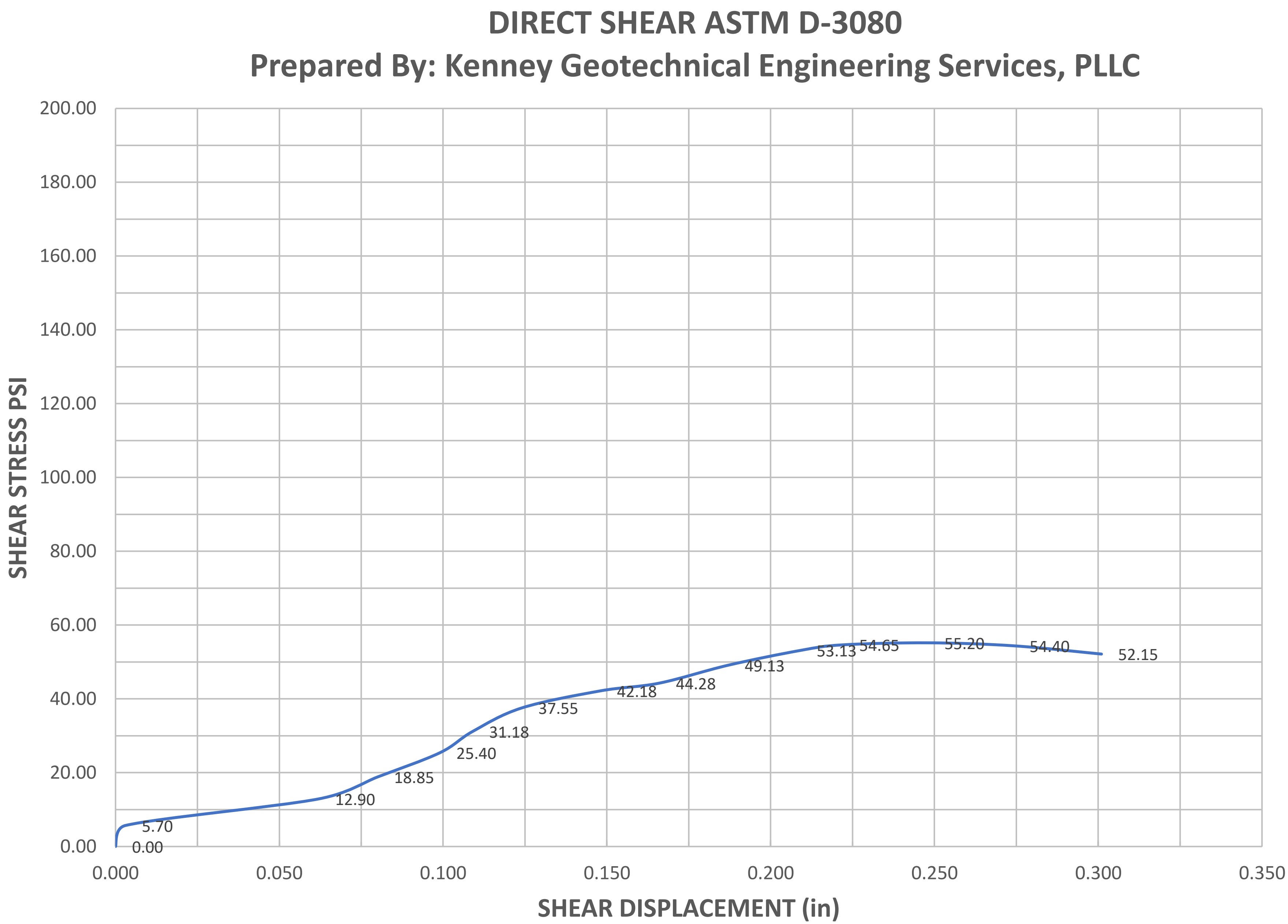
SAT. SAMPLE

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLES PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. SMALL BLOCK CUT FROM THREE DIFFERENT BLOCKS. SAMPLE SATURATED IN BRINE SOLUTION FOR 24 HOURS BEFORE TESTING.

AREA: 4 INCHES SQ. MOISTURE CONDITION: MOIST

NORMAL STRESS: 100 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.003	22.8	5.70
0.062	51.6	12.90
0.080	75.4	18.85
0.099	101.6	25.40
0.109	124.7	31.18
0.124	150.2	37.55
0.148	168.7	42.18
0.166	177.1	44.28
0.187	196.5	49.13
0.209	212.5	53.13
0.222	218.6	54.65
0.248	220.8	55.20
0.274	217.6	54.40
0.301	208.6	52.15



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Project No.:	2021-146	Date:	10/4/2021
Project Name:	CARGILL LANSING		

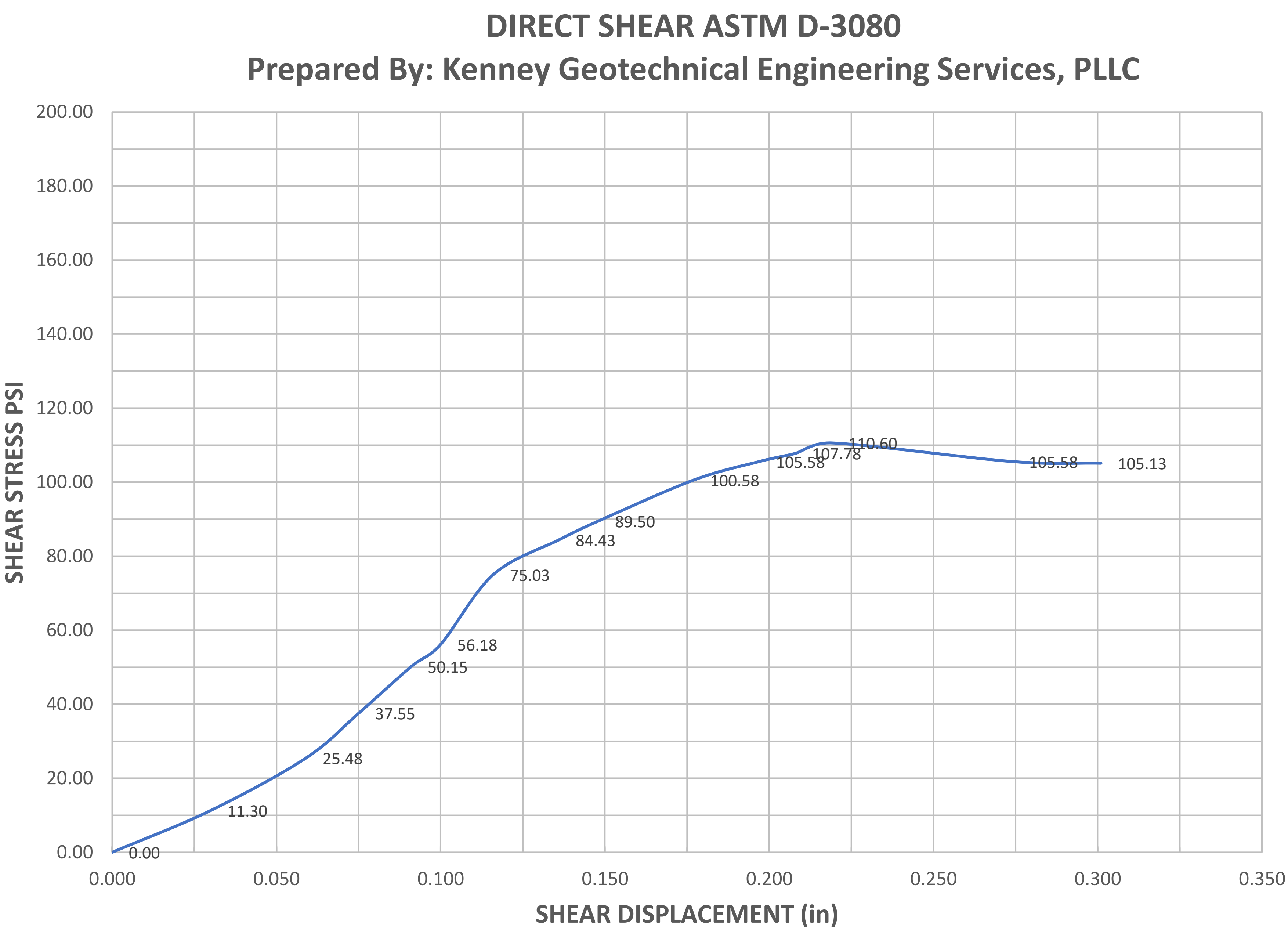
DIRECT SHEAR TEST OF SOIL
ASTM D3080
SAT. SAMPLE

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLES PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. SMALL BLOCK CUT FROM THREE DIFFERENT BLOCKS. SAMPLE SATURATED IN BRINE SOLUTION FOR 24 HOURS BEFORE TESTING.

AREA: 4 INCHES SQ. MOISTURE CONDITION: MOIST

NORMAL STRESS: 200 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.030	45.2	11.30
0.059	101.9	25.48
0.075	150.2	37.55
0.091	200.6	50.15
0.100	224.7	56.18
0.116	300.1	75.03
0.136	337.7	84.43
0.148	358	89.50
0.177	402.3	100.58
0.197	422.3	105.58
0.208	431.1	107.78
0.219	442.4	110.60
0.274	422.3	105.58
0.301	420.5	105.13


DIRECT SHEAR ASTM D-3080
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SHEAR STRESS PSI
SHEAR DISPLACEMENT (in)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.00
11.30
25.48
37.55
50.15
56.18
75.03
84.43
89.50
100.58
105.58
107.78
110.60
105.58
105.13

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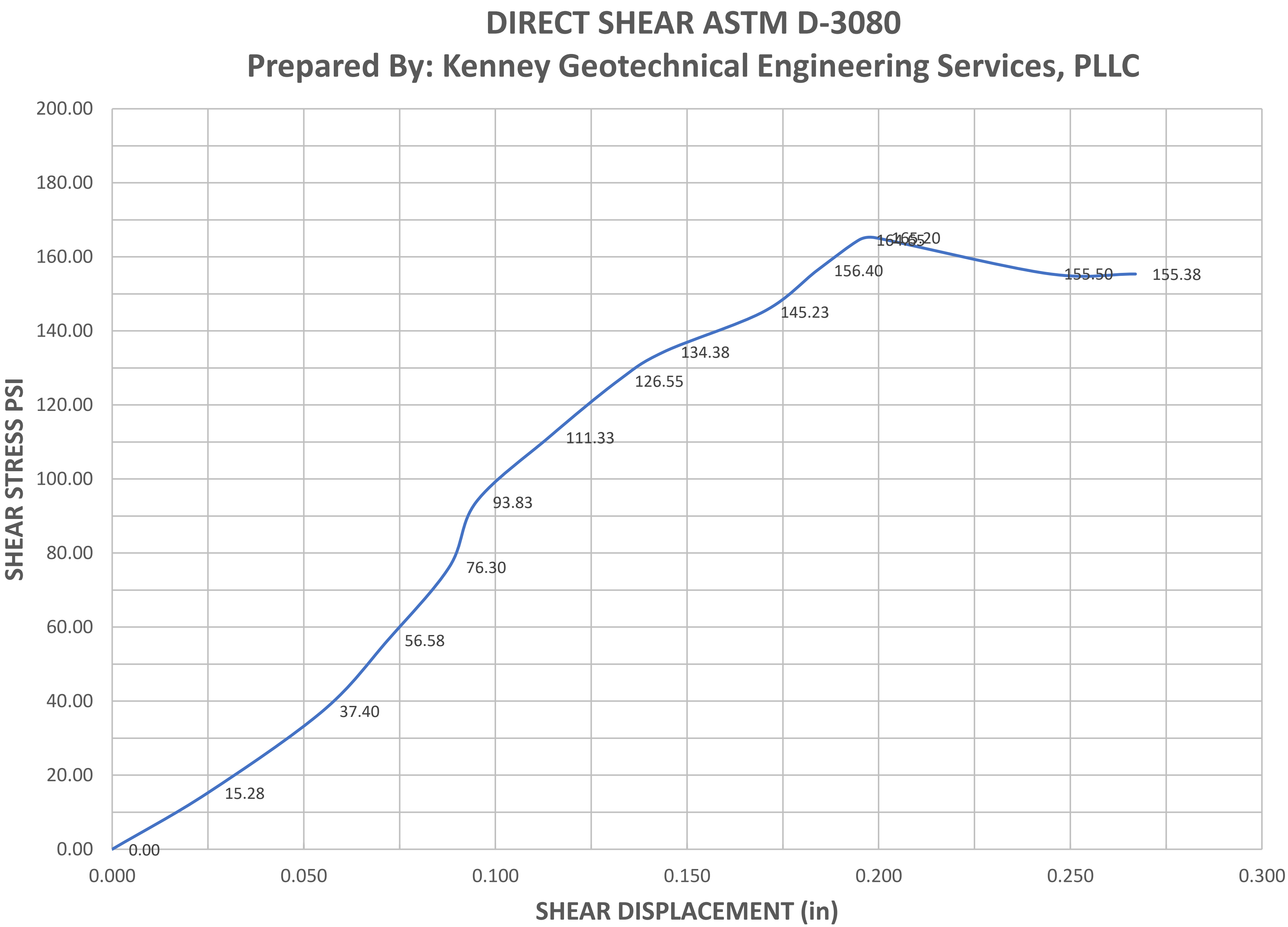
DIRECT SHEAR TEST OF SOIL
ASTM D3080
SAT. SAMPLE

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLES PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. SMALL BLOCK CUT FROM THREE DIFFERENT BLOCKS. SAMPLE SATURATED IN BRINE SOLUTION FOR 24 HOURS BEFORE TESTING.

AREA:	4	INCHES SQ.	MOISTURE CONDITION:	MOIST	
NORMAL STRESS:	300	PSI			

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.025	61.1	15.28
0.055	149.6	37.40
0.072	226.3	56.58
0.088	305.2	76.30
0.095	375.3	93.83
0.114	445.3	111.33
0.132	506.2	126.55
0.144	537.5	134.38
0.170	580.9	145.23
0.184	625.6	156.40
0.195	658.6	164.65
0.199	660.8	165.20
0.244	622	155.50
0.267	621.5	155.38



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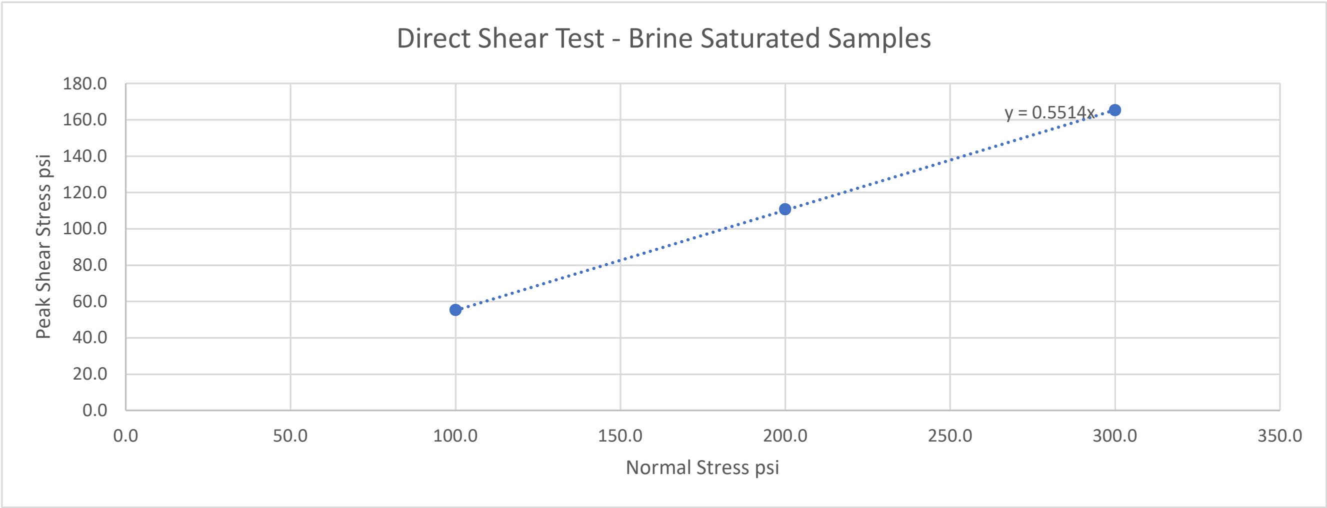
BRINE SAT.	
Normal Stress psi	Peak Shear Stress psi
100.0	55.2
200.0	110.6
300.0	165.2



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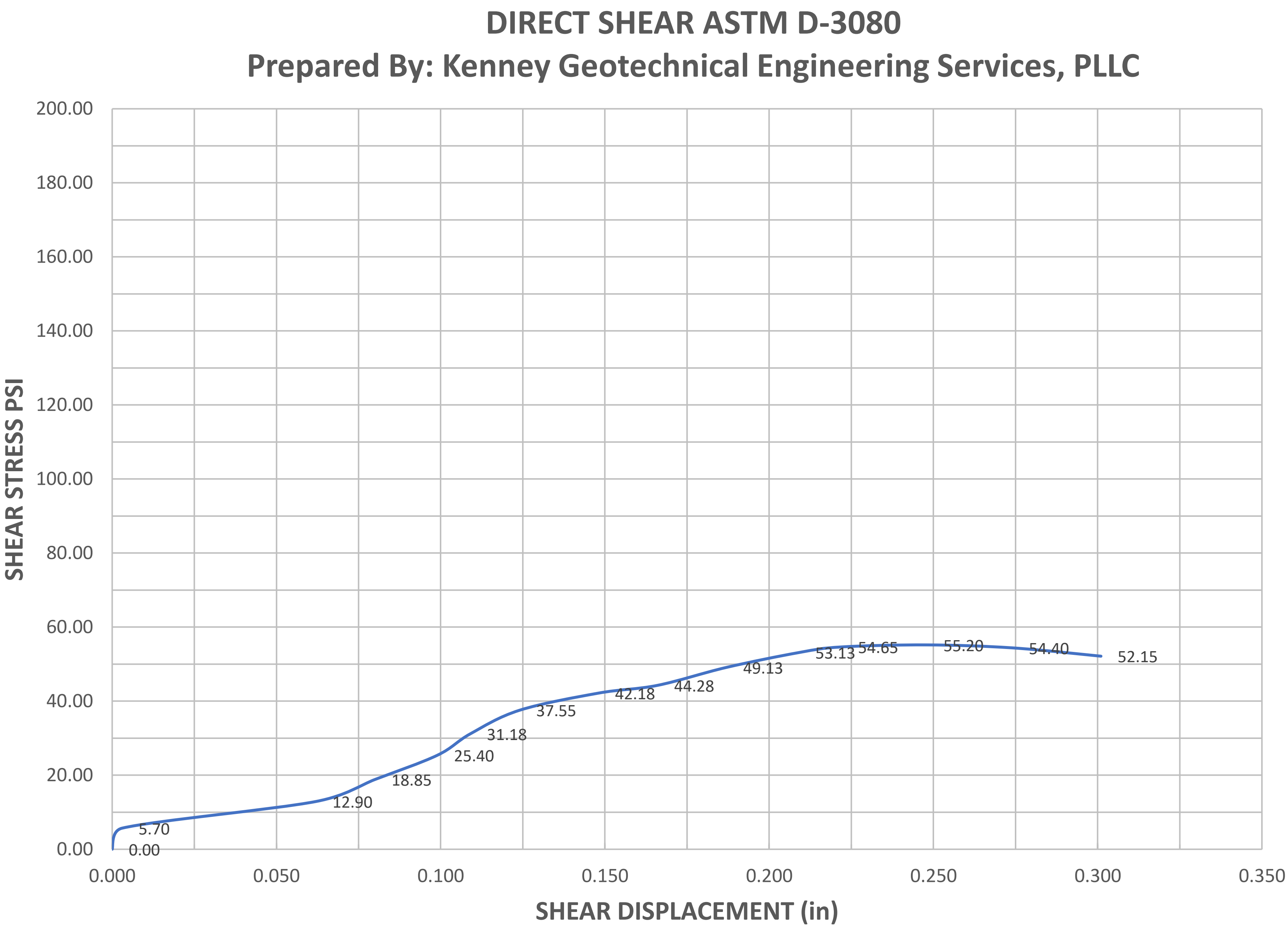
DIRECT SHEAR TEST OF SOIL
ASTM D3080
SAT. SAMPLE

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLES PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. SMALL BLOCK CUT FROM THREE DIFFERENT BLOCKS. SAMPLE SATURATED IN BRINE SOLUTION FOR 24 HOURS BEFORE TESTING.

AREA:	4	INCHES SQ.	MOISTURE CONDITION:	MOIST	
NORMAL STRESS:	100	PSI			

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.003	22.8	5.70
0.062	51.6	12.90
0.080	75.4	18.85
0.099	101.6	25.40
0.109	124.7	31.18
0.124	150.2	37.55
0.148	168.7	42.18
0.166	177.1	44.28
0.187	196.5	49.13
0.209	212.5	53.13
0.222	218.6	54.65
0.248	220.8	55.20
0.274	217.6	54.40
0.301	208.6	52.15


DIRECT SHEAR ASTM D-3080
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SHEAR STRESS PSI
SHEAR DISPLACEMENT (in)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.00
5.70
12.90
18.85
25.40
31.18
37.55
42.18
44.28
49.13
53.13
54.65
55.20
54.40
52.15

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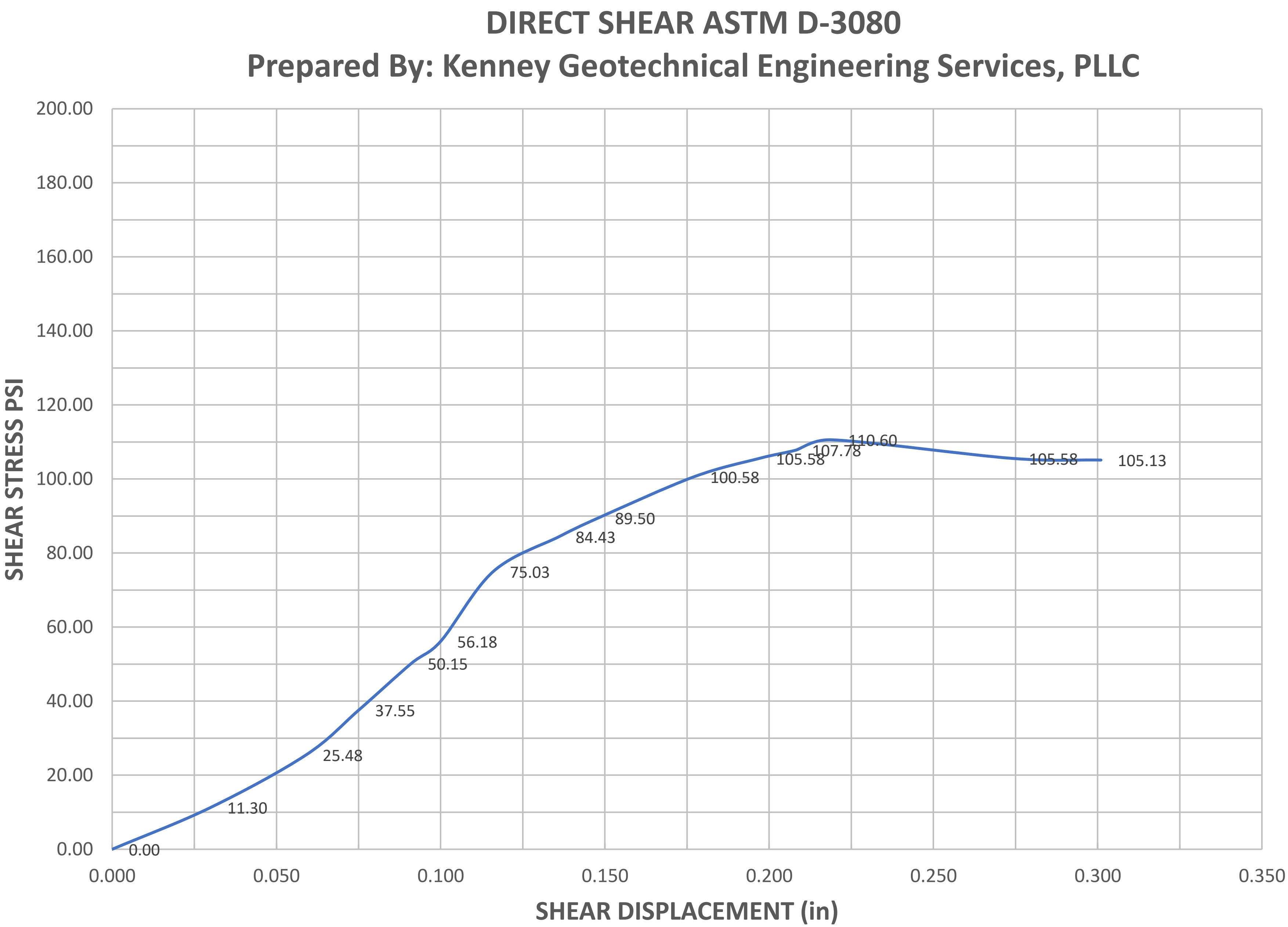
ASTM D3080

SAT. SAMPLE

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLES PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. SMALL BLOCK CUT FROM THREE DIFFERENT BLOCKS. SAMPLE SATURATED IN BRINE SOLUTION FOR 24 HOURS BEFORE TESTING.

AREA:	4	INCHES SQ.	MOISTURE CONDITION:	MOIST
NORMAL STRESS:	200	PSI		

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.030	45.2	11.30
0.059	101.9	25.48
0.075	150.2	37.55
0.091	200.6	50.15
0.100	224.7	56.18
0.116	300.1	75.03
0.136	337.7	84.43
0.148	358	89.50
0.177	402.3	100.58
0.197	422.3	105.58
0.208	431.1	107.78
0.219	442.4	110.60
0.274	422.3	105.58
0.301	420.5	105.13


DIRECT SHEAR ASTM D-3080
Prepared By: Kenney Geotechnical Engineering Services, PLLC
SHEAR STRESS PSI
SHEAR DISPLACEMENT (in)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.00
11.30
25.48
37.55
50.15
56.18
75.03
84.43
89.50
100.58
105.58
107.78
110.60
105.58
105.13

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DIRECT SHEAR TEST OF SOIL
ASTM D3080

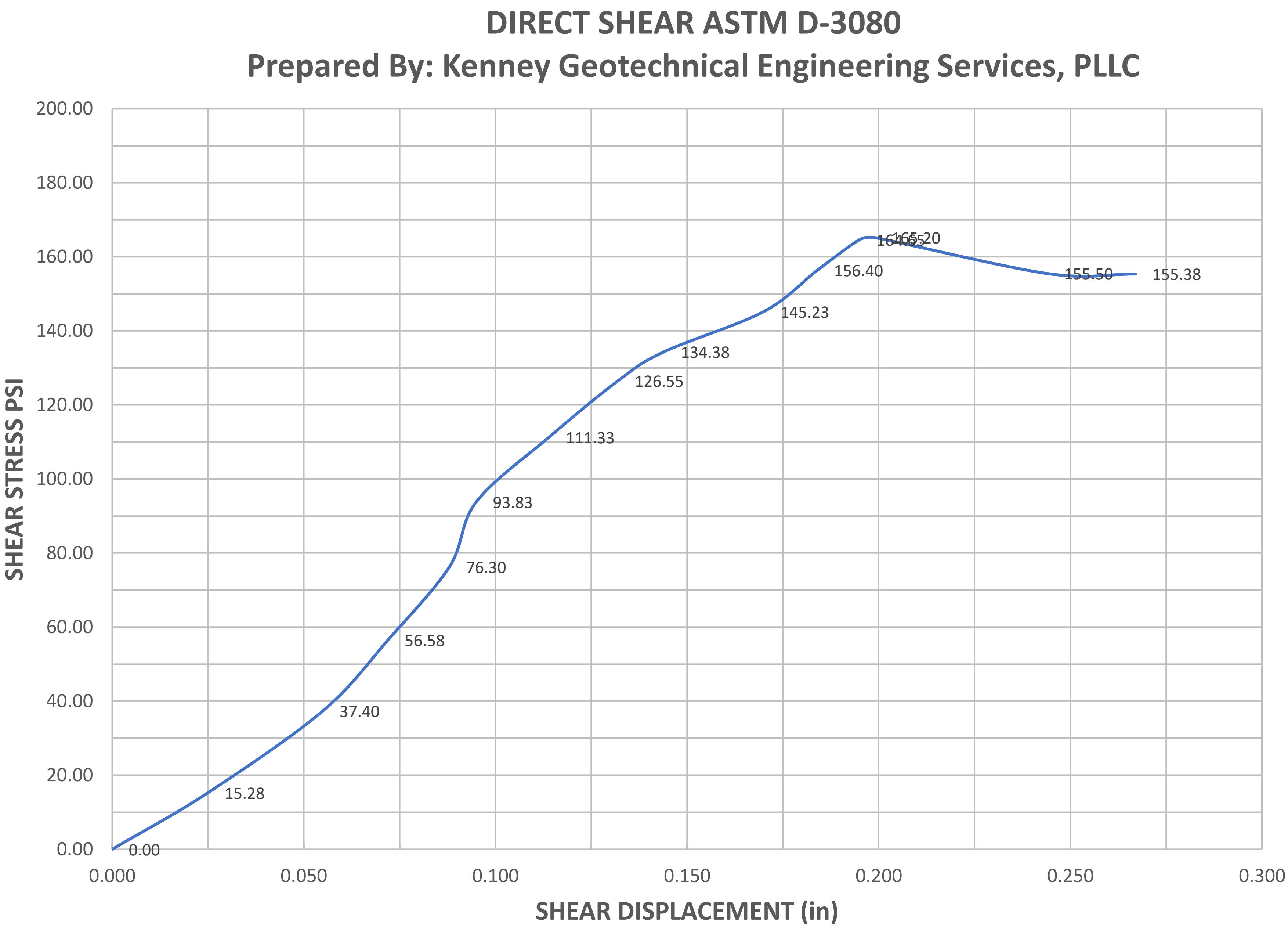
SAT. SAMPLE

NOTES: TESTING PERFORMED ON ROCK SALT BLOCK SAMPLES PROVIDED BY CARGILL. SAMPLE WRAPPED IN PLASTIC PRIOR TO TESTING. SMALL BLOCK CUT FROM THREE DIFFERENT BLOCKS. SAMPLE SATURATED IN BRINE SOLUTION FOR 24 HOURS BEFORE TESTING.

AREA:	4	INCHES SQ.	MOISTURE CONDITION:	MOIST

NORMAL STRESS: 300 PSI

DISPLACMENT (INCH)	LOAD LBS	STRESS PSI
0.000	0	0.00
0.025	61.1	15.28
0.055	149.6	37.40
0.072	226.3	56.58
0.088	305.2	76.30
0.095	375.3	93.83
0.114	445.3	111.33
0.132	506.2	126.55
0.144	537.5	134.38
0.170	580.9	145.23
0.184	625.6	156.40
0.195	658.6	164.65
0.199	660.8	165.20
0.244	622	155.50
0.267	621.5	155.38



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BRINE SAT.	
Normal Stress psi	Peak Shear Stress psi
100.0	55.2
200.0	110.6
300.0	165.2