thoroughbred solar

Attachment C

Geotechnical Report

Exhibit 12 – Site Assessment Report

CONSULTING GROUP

Report of Preliminary Geotechnical Investigation

Proposed 50 MWac Hart Solar Power Plant
Rowletts, Hart County, Kentucky

Latitude 37.24278° N
Longitude 85.92142° W

Prepared for:

Leeward Renewable Energy Development, LLC
6688 N. Central Expressway, Suite 500
Dallas, Texas 75206

G2 Project No. 213841
October 6, 2022

CONSULTING GROUP

October 6, 2022

Mr. Eric Thornbrew, P.E. – Senior Civil Engineer
Leeward Renewable Energy Development, LLC
6688 N. Central Expressway, Suite 500
Dallas, Texas 75206

Re: Report of Preliminary Geotechnical Investigation
Proposed Eiffel 200 MWac Solar Power Plant
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Dear Mr. Thornbrew:

We have completed the Preliminary Geotechnical Investigation for the proposed Hart Solar Power Plant to be constructed in Hart County, Kentucky. This report presents the results of our observations, on-site testing and analyses, and our preliminary recommendations for site preparation, foundation design, and construction considerations as they relate to the geotechnical conditions beneath the site.

We appreciate the opportunity to be of service to Leeward Renewable Energy Development, LLC and look forward to discussing the recommendations presented. In the meantime, if you have any questions regarding the report or any other matter pertaining to the project, please call us.

Sincerely,

G2 Consulting Group, LLC

Jeffrey D. Crow
Project Engineer

David L. Wanlass
Project Manager

STATE OF KENTUCKY
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ATTACHMENTS

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1.0 PROJECT DESCRIPTION

The project site, identified as the Hart Solar Project, is located directly west of the town of Rowletts, and approximately 3-1/2 miles north of downtown Horse Cave. The site is identified on the attached Geotechnical Test Location Plan, Plate No. 2, in relation to the surrounding area. The overall site is approximately 500 acres in area, of which generally half is currently designated for possible construction of arrays of photovoltaic (PV) solar panels. The solar panels and tracker tilt style frames will likely be supported on galvanized steel W6x9 driven piles extending approximately 5 to 8 feet below grade. We understand the proposed substation will be situated within the southern area of the site and along the existing transmission line that traverses north to south through the eastern portion of the project site.

We understand auxiliary systems and structures may include power conversion enclosures, transformers and overhead power transmission lines; however, these have not yet been identified or laid out. Most of the other structures are typically supported on shallow spread footing foundations or mat foundations. The power transmission monopoles are supported on drilled cast-in-place concrete pier foundations. The development is also anticipated to include underground utilities, site surface drainage features, gravel surfaced site roads and access roads.

Final design grades were not available at the time of this report; however, proposed site grades are expected to be similar to existing grades ranging from Elevation 557 feet to 742 feet. We anticipate earthwork will include minor grade cuts and fill placement to correct grade disparities and to prepare structure pads, pavement subgrades and site drainage excavations. At the time of this report, no other specific project or structural information regarding the proposed development was available for review.

2.0 SCOPE OF SERVICES

The field operations, laboratory testing, and engineering report preparation were performed under the direction and supervision of a licensed professional engineer. Our services were performed according to generally accepted standards and procedures in the practice of geotechnical engineering. Our scope of services for this project was as follows:

1. We installed a total of six (6) steel W6x9 test piles to embedment depths ranging between 4-1/2 and 8 feet below existing ground surface within the proposed solar array fields. Two (2) test piles were installed at each of the test locations PLT-1 through PLT-3.

2. We excavated a total of seven (7) test pits, PLT-1 through PLT-3 and TP-4 through TP-7, to depths ranging between 3-1/2 and 8 feet below the existing ground surface.

3. We drilled a total of one (1) soil boring. Soil boring B-1 was performed near the proposed on-site substation and extended to depth of 26 feet. Coring of the bedrock was performed to a depth of 8 feet below the bedrock contact.

4. We performed in-situ soil electrical resistivity testing at a total of one (1) test location.

5. We performed laboratory thermal resistivity testing on three (3) five-point sets of remolded soil samples obtained from three (3) of the test pit excavations.

6. We performed laboratory soil chemical corrosivity testing, including soil soluble sulfate content, soluble sulfide content, soluble chloride content, pH, “soil box” electrical resistivity, and oxidation-reduction (redox) potential on representative samples obtained from three (3) of the test pit excavations.

7. We performed laboratory geotechnical testing, including thermal resistivity, Standard Proctor compaction, California Bearing Ratio (CBR), Atterberg limits, unconfined compressive strength, natural moisture content, organic matter content determinations (loss-on-ignition), and visual engineering classification on representative samples obtained from the soil boring and test pit excavations.

8. We prepared this preliminary geotechnical engineering report. This report includes recommendations based on the encountered and tested geotechnical conditions at the site.

3.0 FIELD OPERATIONS

Leeward Renewable Energy Development, LLC (LRED) and G2 Consulting Group, LLC (G2) selected the number, depths and locations of the soil boring, test pits and test piles based on the features of the proposed development and site access conflicts. A G2 representative staked the proposed test locations in the field at the approximate locations indicated on the attached **Geotechnical Test Location Plan, Plate No. 2.**

3.1 Soil Boring/Rock Coring

Soil boring B-1 was performed within the proposed on-site substation area and extended to a depth of 26 feet. The soil boring was drilled by Tri-State Drilling, LLC using an all-terrain vehicle (ATV) mounted rotary drill rig under the guidance and direction of G2 personnel.

Continuous-flight, 4-1/4-inch inside diameter hollow-stem augers were used to advance the borehole to the approximate drilling refusal depth of 18 feet. Soil samples were obtained at intervals of 2-1/2 feet within the upper 10 feet and at intervals of 5 feet below that depth. These samples were obtained by the Standard Penetration Test method ASTM D1586, which involves driving a 2-inch diameter split-spoon sampler into the soil with a 140-pound weight falling 30 inches. The sampler is generally driven three successive 6-inch increments, with the number of blows for each increment recorded. The number of blows required to advance the sampler the last 12 inches is termed the Standard Penetration Resistance (N). The blow counts for each 6-inch increment and the resulting N-values are presented on the individual soil boring log.

A diamond tipped core barrel was used to extend the soil boring from an approximate depth of 18 feet to a final depth of 26 feet. Core samples were obtained for rock classification and rock quality determinations (RQD).

The soil and rock cores samples obtained during field operations were placed in sealed containers in the field and shipped to our laboratory for testing and classification. During the field operations, the drilling crew maintained a log of the encountered subsurface conditions, including changes in stratigraphy and observed groundwater levels. After completion of drilling operations, the borehole was backfilled with auger cuttings. The final soil boring log is based on the field log and laboratory soil classification and test results. The soil boring log is presented in **Appendix A, Figure No. 01.**

3.2 Test Pits

Test pits PLT-1 through PLT-3 were performed at the test pile areas and extended to depths ranging between 3-1/2 and 8 feet. Test pit TP-4 was performed within the proposed substation area and extended to a depth of 4 feet. Test pits TP-5 through TP-7 were performed within the proposed solar array areas and extended to depths ranging between 4 and 9 feet. The test pits were excavated using a Bobcat E85 compact excavator equipped with a 24-inch-wide bucket.

During excavation operations, a log of the encountered subsurface conditions was maintained for each location, including changes in stratigraphy and observed groundwater levels. G2 personnel entered each test pit for in-situ unconfined compressive strength testing to a maximum depth of 5 feet below existing grade by using a spring-loaded hand penetrometer device. The hand penetrometer estimates the unconfined compressive strength to a maximum of 4-1/2 tons per square foot (tsf) by measuring the resistance of the soil sample to the penetration of a calibrated spring-loaded cylinder. Additional hand penetrometer tests were performed on spoils excavated below a depth of 5 feet.

Bulk samples of the excavated soils were obtained and placed in sealed containers in the field for further laboratory testing and classification. After completion of the excavation operations, the test pits were backfilled with the excavated soils. No controlled compaction of the backfill was performed during backfilling operations. The final test pit logs are based on the field logs, laboratory test results and laboratory soil classification. The test pit logs are presented in **Appendix A, Figure Nos. 02 through 08.**

3.3 In-Situ Soil Electrical Resistivity Testing

In-situ soil electrical resistivity tests were performed at one (1) test area. The testing was performed following the Wenner four-pin test procedure (ASTM G57-06) using a Nilsson Model 400 resistivity meter with steel probes. The pins were set at a spacing of 2, 5, 10, 20, 30 and 50 feet. The results of the electrical resistivity tests are presented in **Appendix B, Figure No. 19.**

4.0 GEOTECHNICAL LABORATORY TESTING

Representative soil samples were subjected to geotechnical laboratory testing to determine soil parameters pertinent to site preparation and foundation and pavement design. An experienced geotechnical engineer classified the samples in general conformance with the Unified Soil Classification System (USCS). Laboratory testing included determinations based on the following standards:

Test Procedure	ASTM Procedure	Test Quantities	Sample Depths	Appendix A Figure Nos.
Natural Moisture Content	D2216	7	0 to 8 feet	01 – 08
Organic Matter Content	D2974	7	0 to 1 foot	01 - 08
Atterberg Limits	D4318	4	2 to 4-1/2 feet	10
Unconfined Compressive Strength	D2166	3	2-1/2 to 7-1/2 feet	11
Compressive Strength of Intact Rock	D7022	2	18 to 26 feet	01
Standard Proctor	D698	2	1 to 4 feet	12 – 13
California Bearing Ratio (CBR)	D1883	2	1 to 4 feet	14 – 15
Thermal Resistivity Dryout Curves	D5334	3	2 to 4 feet	16 – 18

Additional unconfined compressive strength tests were performed using a spring-loaded hand penetrometer device. The hand penetrometer estimates the unconfined compressive strength to a maximum of 4-1/2 tons per square foot (tsf) by measuring the resistance of the soil sample to the penetration of a calibrated spring-loaded cylinder.

5.0 SUBSURFACE CONDITIONS

5.1 Regional and Site Geology

According to the United States Geological Survey (USGS), the northern portion of the project site is located in the geologic region identified as Ste. Genevieve and St. Louis Limestone Formations. According to the Kentucky Emergency Management division (KYEM), the project site is also located in a geologic region identified as having a severe potential of forming karst-like features. Based on the surface features of the project site, there are several known, probable and possible sinkholes present throughout the area. According to the United States Department of Agriculture (USDA) soil survey, the near surface soils across the site consist predominantly of silt loam, silty clay loam and clay loam. These soils are identified as having very low to high permeability rates between 0.00 and 2.00 inches per hour.

The local climate is humid with an average of 52 inches of annual rainfall and 6 inches of annual snowfall. Temperatures generally range between 28 degrees in January and 89 degrees in July.

5.2 Site Seismicity

Based on the 2018 International Building Code, our familiarity with soil conditions in the area, and our engineering judgement, structures may be designed for seismic loading conditions on the basis of the following seismic coefficients and classifications. The Applied Technology Council (ATC) hazards tool (https://hazards.atcouncil.org/) was used for determination of seismic coefficients. If additional information is obtained from deeper soil borings or other geotechnical investigations, the Site Class assumed below shall be confirmed.

- Site Class C – Very Dense Soil and Soft Rock
- Maximum Considered Earthquake Spectral Response Acceleration
    - At short periods (S_S) = 0.226g
    - At one second period (S_1) = 0.118g
- Maximum Considered Earthquake Spectral Response Acceleration (adjusted for site class)
    - At short periods (S_{MS}) = 0.361g
    - At one second period (S_{M1}) = 0.275g
- Five Percent Damped Design Spectral Response Acceleration
    - At short periods (S_{DS}) = 0.241g
    - At one second period (S_{D1})= 0.184g

Loose granular soils and soils below a shallow groundwater table are generally more susceptible to liquefaction and seismic-induced settlement. In the event of an earthquake episode producing the maximum considered ground acceleration of 0.361g, there is very little to no potential for localized liquefaction to occur within the native stiff to very stiff lean and fat clay soils. Given that the site is also in an area with a low probability for seismic activity, we believe there is very little to no risk of liquefaction occurring at this site. No site remediation for seismic activity is recommended.

5.3 Soil and Rock Conditions

Approximately 3 to 16 inches, with an average thickness of 11 inches, of sandy clay tilled earth is present at the ground surface of each of the test pit locations (PLT-1 through PLT-3 and TP-4 through TP-7) and soil boring B-1. In general, the surface soils throughout most of the project area have been tilled for agricultural purposes. The resulting tilled earth is comprised of native soil that has been disturbed by these agricultural processes and includes varying quantities of organic matter. The tilled earth has moisture contents ranging from 18 to 44 percent and organic matter contents ranging from 3 to 7 percent.

Native lean to fat clay underlies the tilled earth of each test pit and boring location and extends to a depth of 18 feet within boring B-1 and to the explored depths of each test pit location. The native cohesive soils are stiff to very stiff in consistency with unconfined compressive strengths ranging from 3,000 to 7,750 psf. The native cohesive soils have natural moisture contents between 16 and 36 percent, liquid limits between 38 and 72 percent, and plasticity indexes between 18 and 44 percent. The native cohesive soils present at test pits PLT-1 and TP-4 have CBR values ranging from 3.2 to 6.2 at 95 percent compaction and associated CBR swell measurements ranging from 0.4 to 0.8 percent after 96 hours of inundation.

Faintly weathered limestone underlies the native cohesive soils at boring B-1 and extends to the explored depth. Native limestone fragments are also generally present within the cohesive soils of each test pit location. In addition, excavation refusal due to weathered limestone (apparent bedrock) was encountered at test pit locations PLT-2, PLT-3 and TP-4 through TP-7. The weathered limestone below a depth of 18 feet within boring B-01 is moderately strong in abrasion, with unconfined compressive strengths ranging from 8,490 to 11,090 pounds per square inch (psi), an approximate moisture content of less than 1 percent, and a dry density of 164 pounds per cubic foot (pcf). In addition, the limestone has Rock Quality Designations (RQD) ranging between 53 and 96 percent.

The Soil Boring Log and Test Pit Logs are presented in Appendix A, Figure Nos. 01 through 08. The stratification depths shown on the boring and test pit logs represent the soil conditions at the exploration locations. Variations may occur between exploration locations. Additionally, the stratigraphic lines represent the approximate boundary between soil types. The transition may be more gradual than what is shown. G2 has prepared the boring and test pit logs on the basis of laboratory classification and testing as well as field logs of the soils encountered. General Notes Terminology defining the nomenclature used on the boring and test pit logs and elsewhere in this report are presented in Appendix A, Figure No. 09.

5.4 Groundwater Conditions

No measurable groundwater was encountered during or upon completion of boring and test pit excavation operations. Fluctuations in groundwater levels should be anticipated due to seasonal variations and following periods of prolonged precipitation. It should be noted that groundwater observations made during drilling operations in predominantly cohesive soils are not necessarily indicative of the static groundwater level. This is due to the low permeability of such soils and the tendency of drilling operations to seal off the natural paths of groundwater flow.

5.5 Thermal Resistivity

Thermal resistivity of remolded soil samples within a range of moisture contents were performed on three (3) bulk soil samples obtained at depths between 2 and 4 feet below the ground surface. The results were used to plot the five-point Thermal Resistivity Dryout curves for each sample. To do this, a One-point Standard Proctor is performed on each bulk sample to determine the soil's maximum density at the sample's as-received moisture content. Three (3) sets of four (4) remolded samples (one set per bulk sample) were then prepared near the as-received moisture content and at a density equal to approximately 85 percent of the maximum dry density value. The thermal resistivity of one of the remolded samples from each sample set was determined near the as-received moisture content using a KD2 Pro Thermal Properties Analyzer in general conformance with the procedures described in the ASTM D5334 method of testing. Then, all four (4) remolded samples were placed in a 140˚F oven to dry until average moisture contents near 7 percent, 3 percent, 1 percent and 0 percent are achieved for each bulk sample. The thermal resistivity is determined at each of these average moisture contents. After testing individual samples, each sample was extruded and the moisture content of the sample surrounding the analyzer's sensor depth of 2 inches was determined for comparison to the average moisture content. A summary of the test results at a soil moisture content of 2 percent is presented below.

Test Procedure	ASTM	Laboratory Resistivity (˚C-cm/W)		Appendix A Figure Nos.
		Interpolated 2% Average Moisture Content	Interpolated 2% Moisture Content at Sensor	
Laboratory Thermal Resistivity Dry-Out Curves	D5334	Minimum: 219 Maximum: 283 Average: 257	Minimum: 231 Maximum: 330 Average: 283	16 – 18

5.6 Soil Corrosivity

5.6.1 Electrical Resistivity

In-situ soil electrical resistivity testing was performed at one test location. The in-situ testing was performed using a Nilsson Model 400 resistivity meter with steel probes. In addition, laboratory soil electrical resistivity testing was performed on bulk soil samples obtained from three (3) test locations on our behalf by Essential Corrosion Protection (Columbia, MD). A summary of the tested in-situ and laboratory test results is presented in the following table.

Test Procedure	ASTM	Test Depth or "a" spacing (feet)	Minimum Soil Resistivity (ohm-cm)	Maximum Soil Resistivity (ohm-cm)	Appendix B Figure No.
In-Situ ERT	G57	0 to 50	13,400	181,000	19
Laboratory ERT	G57	2 to 3	10,000	14,000	20

Based on the test results, the upper soils should generally be considered mildly corrosive based on the standard limits presented below.

Soil Corrosivity	Soil Resistivity (ohm-cm)
Extremely/Very Corrosive	Less than 1,000
Corrosive	1,000 to 5,000
Moderately Corrosive	5,000 to 10,000
Mildly Corrosive	Over 10,000

5.6.2 Laboratory Soil Chemical Corrosivity

Laboratory chemical tests were performed on three (3) bulk soil samples obtained from the sides of the corresponding test pit excavations between 2 and 3 feet below the ground surface. The testing was performed on our behalf by Essential Corrosion Protection (Columbia, MD). A summary of the test results is presented in the following table:

Test Procedure	ASTM	Minimum	Maximum	Soil corrosivity to buried metallic and concrete structures	Appendix B Figure No.
pH	G51	6.5	6.7	Negligible (Generally neutral)	20
Sulfates Content	D516	Less than 5 ppm	Less than 5 ppm	Negligible (less than 150 ppm) Moderate (between 150 and 1,500 ppm)	20
Chloride Content	D512	Less than 20 ppm	Less than 20 ppm	Negligible (Less than 150 ppm)	20
Oxidation-Reduction Potential	D1498	234 mV	260 mV	Slight (Between 200 and 400 mV)	20

6.0 EARTHWORK RECOMMENDATIONS

6.1 Site and Subgrade Preparation

Earthwork operations are expected to consist of removing the existing vegetation, cutting existing soils or placing engineered fill to achieve proposed site design grades and minimize any severe surface undulations within proposed solar panel areas, excavating for foundations and underground utilities, and preparing the subgrade for support of access and maintenance drives. G2 recommends all earthwork operations be performed in accordance with specifications that have been prepared by a Kentucky licensed professional engineer and be properly monitored in the field by qualified technical personnel under the direction of a licensed engineer.

At the beginning of the earthwork operations, all vegetation and their root mass should be grubbed from proposed construction areas and disposed of. Approximately 7 to 16 inches (average of 11 inches) of dark brown sandy clay tilled earth (topsoil) is present at the ground surface of each test pit location. The tilled earth has organic matter contents ranging between 3 to 7 percent. The table on the following page presents earthwork recommendations specific to the existing tilled earth.

Proposed Project Element	Tilled Earth (and Topsoil) Earthwork Recommendation
1) Buildings or auxiliary structures supported on shallow foundations	Remove tilled earth from within structure footprint.
2) Site access, perimeter and interior maintenance roads	Remove tilled earth if organic matter content exceeds 5 percent. Otherwise, tilled earth can remain-in-place for support of roads.
3) General site fill (non-engineered) placed to raise site grades within solar panel array areas	Remove tilled earth if organic matter content exceeds 5 percent, or if proposed general fill placed exceeds 1 foot thick. Otherwise, tilled earth can remain-in-place for support of general fill.

The native cohesive soils are highly prone to instability due to fluctuations in moisture content and will become very unstable during prolonged precipitation periods. As such, we recommend site grading operations be performed during extended periods with low precipitation. If grading operations are performed during or after recent precipitation events, it may be necessary to provide supplemental subgrade stabilization along construction traffic routes.

Once the proposed subgrade has been exposed, and prior to placement of any engineered fill and/or construction of pavement sections, the exposed subgrade in proposed pavement and auxiliary structure areas should be thoroughly proof-rolled using a heavy rubber-tired vehicle, such as a fully-loaded dump truck or front-end loader, and should be visually evaluated for instability and/or unsuitable conditions. Any remaining unstable or unsuitable areas should be densified with additional compaction or undercut and replaced with engineered fill.

6.2 Engineered Fill Soils and Placement

Where buildings or auxiliary structures supported on shallow spread footing foundations or mat foundations are planned, any fill soils placed beneath these structures shall consist of engineered fill. Where site access, perimeter and interior maintenance roads are planned, any fill soils placed beneath these roads shall also consist of engineered fill. Engineered fill should extend a distance laterally beyond the structure or road perimeter at least equal to twice the depth of the fill.

Imported engineered fill should consist of pre-approved environmentally clean soils, and should be free of organic matter, frozen soil clods, or other harmful material. Engineered fill should have a liquid limit less than 40 percent and a plasticity index of less than 12 percent.

The on-site clay should not be used as engineered fill due to having a high potential for shrinkage or swelling with decreases or increases in moisture content (Liquid Limit = 38 to 72; Plasticity Index = 18 to 44, CBR Swell = 0.4 to 0.8 percent), particularly after they have been disturbed and recompacted. The following table presents a summary of the recommended general and engineered fill soil types.

Soil Type	Engineered fill under roads or structures supported on shallow foundations	General site fill (non-engineered) placed to raise site grades within solar panel array areas
Tilled Earth (and Topsoil)	no	no
Native Clay	no	yes
Imported Granular Soils	yes	yes

Engineered fill should be placed in uniform horizontal layers, not more than 9 inches in loose thickness. The engineered fill should be compacted to achieve a density of at least 95 percent of the maximum dry density as determined by the Standard Proctor compaction test (ASTM D698). We recommend the general fill be compacted to achieve a density of at least 90 percent of the maximum dry density as determined by the Standard Proctor compaction test (ASTM D698) in order to provide reasonable surface stability and erosion resistance.

Sheep-foot roller compaction equipment should be used for all compaction operations using cohesive soils. Any on-site cohesive fill soils should be compacted at moisture contents that are within 3 percent above the optimum moisture content. If imported non-cohesive granular fill soils are used as engineered fill or aggregate base material for roadways, the granular fill should be compacted at moisture contents that are within 2 percent above or below the optimum moisture content.

6.3 Permanent Fill Slopes and Temporary Excavation Slopes and Support

Provided the recommendations for site and subgrade preparation are adhered to as described herein, slope stability analyses indicate that permanent fill slopes, consisting of properly compacted engineered fill, may be designed at inclinations as steep as 2H:1V. Permanent fill slopes, consisting of general fill, may be designed at inclinations as steep as 3H:1V. Any fill soils placed on existing slopes should be continuously keyed into the existing slopes. We recommend key dimensions of at least 4 feet wide and no more than 1-1/2 feet deep. Further analyses may be required on a case-by-case basis for unique challenges at specific locations.

To achieve a uniformly compact surface on the face of the new fill slopes, the slopes should be overfilled and trimmed back. Fill slopes should be protected against erosion as soon as practical after construction. Erosion protection may consist of vegetation, composite erosion mats, top-of-slope swales or other drainage methods that direct water away from the top and toe of the slope.

For open cut temporary excavations where space is available, above the groundwater table and where personnel will enter the excavations, temporary unsurcharged slopes may be sloped back to a maximum depth of 5 feet without shoring at 3/4 units horizontal to 1 unit vertical (3/4H:1V) within the existing stiff to very stiff cohesive soils and limestone bedrock. Where groundwater seepage from excavation cuts is observed, the slopes will need to be flattened sufficiently to achieve stability, but in no case left steeper than 3H:1V at the seepage level. The tops of the slopes should be barricaded to prevent vehicles and storage loads within 5 feet of the tops of the slopes. If materials are stored or equipment is operated near an excavation, shoring and slopes must be designed to resist the additional lateral pressure due to the surcharge loads. Berms are recommended along the tops of slopes to prevent runoff water from entering the excavations and eroding the slope faces.

Where sloped excavations are not possible, shoring may be required to support vertical cuts that extend below a depth of 5 feet and where personnel will enter the excavations. For design of multi-level braced or tied-back shoring, we recommend the use of a rectangular distribution of lateral earth pressure. It may be assumed that the retained soils with a level surface behind the braced shoring will exert a lateral pressure equal to 24H in pounds per square foot, where H is the height of the shoring in feet. It may be assumed that the retained soils with a level surface behind cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot (pcf) for soils above the water level. If construction traffic or material storage is allowed within 10 feet of the vertical excavation, a uniform vertical pressure of 360 pounds per square foot should be added at the ground surface when determining the design lateral loads.

All excavations should be safely sheeted, shored, sloped, or braced in accordance with local or federal OSHA requirements. If material is stored or equipment is operated near an excavation, stronger shoring must be used to resist the extra pressure due to the superimposed loads and should be evaluated by an experienced professional engineer registered in the State of Kentucky. Care should always be exercised when excavating near existing roadways or utilities to avoid undermining them. In no case should excavations extend below the level of adjacent existing structures unless underpinning is planned.

7.0 PERMANENT ROADWAY RECOMMENDATIONS

We understand that permanent access and internal maintenance roads will be aggregate surfaced. It is expected that the most severe traffic conditions will occur during the construction phase, including heavy construction equipment and construction material delivery vehicles. We anticipate the construction traffic loading conditions will range from 7,000 to 18,000 equivalent 18-kip single-axle loads (ESALs). After construction, site traffic is expected to consist mostly of lighter-duty service trucks; however, occasional traffic from emergency vehicles, including emergency fire apparatuses weighing up to 75,000 pounds, may occur periodically.

7.1 Roadways on Native Subgrade

In accordance with AASHTO pavement design criteria for low volume aggregate-surfaced roads, we have assumed an Allowable Serviceability Loss of 2.5 (ΔPSI), and an Elastic Modulus of Aggregate Base (E_{BS}) of 35,000 psi. The tested California Bearing Ratio (CBR) values of the native soils range from 3.2 to 6.2 at 95 percent compaction. Based on these results, we recommend an effective CBR value of 3.2 for use in pavement design. A CBR value of 3.2 is approximately equivalent to a Resilient Modulus (M_R or E_R) of 4,800 psi (M_R psi = 1500 CBR, Heukelom & Klamp, 1962). The design charts (AASHTO 1993, II-74 and AASHTO 1993, II-75) with resolved traffic capacity for design aggregate thicknesses of 6 through 12 inches are presented in **Appendix F, Figure Nos. 85 and 86**. The table below presents the allowable traffic capacities for the varying aggregate thicknesses and associated allowable rut depths.

Permanent Aggregate Surfaced Roads bearing on Native Subgrade			
Aggregate	**Thickness**	**Allowable 18-kip ESALs**	**Allowable Rut Depth (inches)**
KTC Dense Graded Aggregate (DGB) or Crushed Stone Base (CSB)	6 inches	6,000	1.4
	7 inches	9,000	1.4
	8 inches	12,000	1.4
	9 inches	18,000	1.4
	10 inches	22,000	1.5
	11 inches	32,000	1.6
	12 inches	47,000	1.7

Periodic access by emergency fire apparatuses weighing up to 75,000 pounds may be supported on roads consisting of at minimum 10 inches of KTC aggregate base placed on properly prepared subgrade.

Where subgrade disturbance or rutting is experienced during construction, we recommend subgrade stabilization be performed prior to final aggregate placement. Subgrade stabilization shall consist of either lime treatment or geogrid reinforcement. If lime treatment is performed, the resulting treated subgrade soils must achieve an unconfined compressive strength increase of at least 50 psi above the natural in-situ subgrade soils. If geogrid reinforcement is used, the stabilization shall consist of placing a layer of triaxial geogrid over the exposed subgrade, and a minimum 9-inch thick layer of 1x3 crushed concrete or gravel over the geogrid. The geogrid shall consist of Tensar TriAx TRX160, or approved equal. The crushed 1x3 should be compacted to a stable and unyielding condition using a minimum 15-ton roller compactor.

7.2 Roadways on Lime Treated Subgrade

The native subgrade soils may be lime treated for support of permanent access and internal maintenance roads. The optimum lime content for producing lime stabilized subgrade soil shall be determined by performing a soil-lime mix design in accordance with the following test procedures:

- ASTM C977 Quicklime and Hydrated Lime for Soil Stabilization
- ASTM D6276 Using pH to Estimate the Soil-Lime Proportion Requirement
- ASTM D5102 Compressive Strength of Compacted Soil-Lime Mixtures

The existing tilled earth (topsoil) is not suitable for blending with lime treated soils due to the presence of organic matter and must be completely undercut. Based on the high plasticity of the underlying native non-organic clay soils, we recommend an initial estimated optimum lime content of 5 percent be used for evaluation. The unconfined compressive strength of the lime-treated soil should be designed for a minimum of 250 psi at 28 days.

To achieve optimal results for lime stabilization, the optimum lime content should be mixed homogeneously with the upper 12 inches of the subgrade soil and achieve a moisture content near optimum moisture content. The treated soil should be compacted to achieve a density of at least 95 percent of the maximum dry density as determined by the Standard Proctor compaction test (ASTM D698). The treated soil should cure for no less than 48 hours before construction traffic is allowed. The surface of the soil-lime mixture should be kept moist throughout the cure time. Additional curing time will be required when the ambient air temperature is 40˚ F and below.

In accordance with AASHTO pavement design criteria for low volume aggregate-surfaced roads, we have assumed an Allowable Serviceability Loss of 2.5 (ΔPSI) and an Elastic Modulus of Aggregate Base (E_{BS}) of 35,000 psi. Properly prepared lime treated subgrade soils can be assigned a Resilient Modulus (M_R) of 10,000 psi. Based on our analyses, periodic access by emergency fire apparatuses weighing up to 75,000 pounds may be supported on roads consisting of a minimum of 5 inches of KTC aggregate base placed on properly prepared lime treated subgrade.

8.0 SHALLOW CONCRETE FOUNDATIONS

Structure foundations should not bear on or within the existing tilled earth (and topsoil), and engineered fill should not be placed over existing tilled earth. In addition, the underlying native cohesive soils have a high potential for shrinkage or swelling with decreases or increases in moisture content, particularly after they have been disturbed (soil bond structure broken down) and recompacted. Structure foundations and floor slabs are not recommended for support directly on engineered fill prepared from on-site cohesive soils.

Based on the assumed soil and climate conditions, the **undisturbed** non-organic native cohesive soils are generally conducive to support of shallow foundation types, such as shallow spread footing or mat foundations for auxiliary systems and structures, provided some risk of differential soil expansion and/or settlement can be tolerated. It is critical to understand that once the native cohesive soils have been disturbed by excavation or construction traffic, the native soils are no longer suitable for reuse or re-compaction as engineered fill beneath foundations; therefore, every attempt should be made to excavate foundations neat and place foundation concrete and flowable fill (if used) as soon as practical to prevent such disturbance.

The undisturbed native clay soils will generally provide suitable support for embedded shallow driven pile or drilled pier foundations that support solar array panels or structure foundations; however, some minor loss of capacity should be expected if the surrounding native clay is allowed to shrink or swell during moisture fluctuations. The likelihood of significant soil moisture fluctuations occurring is considered relatively low in this region.

8.1 Mat Foundation Capacity

Mat foundations bearing on undisturbed native non-organic soils or imported granular engineered fill can be designed using a modulus of subgrade reaction (k_1). We recommended the following average modulus of subgrade reaction (k_1) values be used for determining the allowable subgrade modulus values (k_s) for actual mat foundation dimensions using the relationship presented on the following page, where B equals the least mat foundation width.

Average Modulus of Subgrade Reaction values for Mat Foundations (k_1)	
Soil Type	**K_1 (pci)**
Native Stiff to Very Stiff Clay	55
Native Weathered Limestone (Bedrock)	180
Imported Granular Engineered Fill	100
Allowable Subgrade Modulus Values (k_s) (pci):	$k_s = k_1[(B+1)/2B]^2$

8.2 Spread Footing Foundation Capacity

Spread footing foundations bearing on undisturbed non-organic native soils or imported engineered fill may be designed based on the net allowable soil bearing pressures presented below.

Allowable Soil Bearing Pressures for Spread Footing Foundations	
Soil Type	**$q_{allowable}$ (psf)**
Native Stiff to Very Stiff Clay	3,000
Native Weathered Limestone (Bedrock)	5,000
Imported Granular Engineered Fill	3,000

8.3 Foundation Dimensions

All spread footing and mat foundations should bear within the recommended soils described above, but should also bear at a minimum depth of 2 feet below the final adjacent grade for frost protection. If native soils are undercut and replaced with granular engineered fill, the undercut should extend laterally beyond the foundation perimeter a minimum distance equal to the undercut depth. If granular fill is used to backfill the undercut up to the minimum foundation bearing depth of 2 feet, supplemental drainage of the granular backfill must be provided to prevent pooling of water within the granular fill.

Continuous wall or strip footing foundations should be at least 16 inches in width and isolated column spread footing or mat foundations should be at least 30 inches in their least dimension. We recommend all foundations be suitably reinforced to minimize the effects of differential settlements associated with local variations in subgrade conditions.

8.4 Settlement

If the recommendations outlined in this report are adhered to, total and differential settlement of mat foundations bearing on undisturbed native non-organic stiff to very stiff cohesive soils, weathered bedrock, or granular engineered fill should be less than 1-1/2 inches and 3/4 inch, respectively.

If the recommendations outlined in this report are adhered to, total settlements of individual spread footing foundations and differential settlement between adjacent foundations bearing on the aforementioned soil types should be less than 1 inch and 1/2 inch, respectively.

8.5 Lateral Earth Pressures

Lateral loads on shallow spread footing and mat foundations may be resisted by the combined passive resistance of the adjacent soils and the soil frictional resistance beneath the foundations. The allowable passive resistance of undisturbed native soils or engineered fill may be modeled as a triangular load distribution equal to the pressure developed by a fluid with a density and maximum pressure as presented below:

Allowable Soil Passive Resistance		
Soil Type	**Equivalent Fluid (pcf)**	**Maximum Pressure (psf)**
Native Stiff to Very Stiff Clay	300	3,000
Native Weathered Limestone (Bedrock)	300	5,000
Imported Granular Engineered Fill	300	3,000

An allowable frictional resistance factor of 0.4 may be used along the bottoms of shallow spread footing or mat foundations. A one-third increase in the passive resistance values may be used for temporary wind or seismic loads. Tension loads on spread footing foundations may be resisted by the foundation concrete weight plus the weight of the soil backfill placed over the spread footing foundation.

9.0 DRILLED CONCRETE PIER FOUNDATIONS

Lean to fat clay is present at the ground surface of soil boring B-1 and extends to a depth of 18 feet. Faintly weathered limestone underlies the native clay and extends to the explored depth. The contractor should be prepared to use a rock auger and/or core barrel to excavate through the intermittent limestone layers (if encountered) and underlying bedrock. We recommend drilled piers extend through the native cohesive soil and penetrate at least 1 foot into the limestone bedrock. The actual minimum bedrock embedment should be evaluated by the structural engineer.

No measurable groundwater was encountered during drilling operations. Any drilled piers extending near or below the groundwater table (if encountered) should be constructed with the use of drilling slurry in order to provide a stable working bottom. Once drilling is completed to the design depth, reinforcing steel should be set and concrete placed by tremie method until a positive head of concrete has been established within the casing. We recommend using a concrete mix design with a slump of 5 to 7 inches for free fall placement to reduce the potential for concrete arching and provide a workable material. We recommend using a temporary form, such as a Sonotube®, to form the top portion of the drilled pier. The use of this top form is a beneficial aid to the correct placement and orientation of the anchor bolts.

We recommend any proposed power distribution monopoles at the substation be supported on drilled cast-in-place concrete pier foundations. We anticipate the drilled piers could have shaft diameters ranging between 1-1/2 and 3 feet. Adjacent piers should be spaced at least 3 pier diameters on center to prevent group interaction and bearing capacity reduction. Adjacent piers at different levels should be designed and constructed so the least lateral distance between them is equivalent to or more than the difference in their bearing levels. The upper 3 feet of soil below ground surface should be ignored when determining pier frictional uplift and lateral capacities to account for the effects of seasonal moisture variations and resultant soil shrinkage and swelling, disturbance during construction, and cyclic lateral loading.

9.1 Soil Parameters for Vertical Capacity

The soil parameters presented on the following page should be used for determining ultimate (nominal unfactored) downward and upward capacities of drilled concrete pier foundations:

Boring B-1		Drilled Pier Ultimate Soil/Rock Parameters				
Depth (feet)	Elevation Profile (feet)	Soil Type	Angle of Internal Friction	Soil or Rock Cohesion (psf)	Drilled Concrete Pier Adhesion (psf)	Effective Unit Weight (pcf)
0 to 3	624 to 621	Ignore due to potential moisture fluctuation and disturbance				115
3 to 18	621 to 607	Fat Clay	---	3,000	1,125	120
18 to 26	607 to 599	Limestone Bedrock	---	6,000	1,400	165

Compressive axial loads of drilled piers embed at least 1 foot into native bedrock are resisted by the end bearing at the base of the pier. Tensile axial loads are resisted by skin friction along the pier and the weight of the pier. We recommend using the following ultimate frictional resistance and the ultimate end bearing capacities presented below.

Depth (feet)	Vertical Ultimate Skin Friction and End Bearing Capacities			
	Soil Type	Ultimate Upward Skin Friction (psf)	Ultimate Downward Skin Friction (psf)	Ultimate End Bearing (psf)
0 to 3	Ignore due to potential moisture fluctuation and disturbance			
3 to 18	Fat Clay	1,125	---	---
3 to 26	Limestone Bedrock	1,400	---	120,000

The following tables present ultimate (nominal unfactored) downward and upward capacities for four possible drilled concrete pier diameters:

Pier Depth	Ultimate Downward Capacity (kips) (Downward End Bearing)			
	1.5-foot Diam.	2-foot Diam.	2.5-foot Diam.	3-foot Diam.
19 feet	212	377	589	848
20 feet	212	377	589	848
21 feet	212	377	589	848
26 feet	212	377	589	848

Pier Depth	Ultimate Upward Capacity (kips) (Upward Skin Friction + Pier Weight)			
	1.5-foot Diam.	2-foot Diam.	2.5-foot Diam.	3-foot Diam.
19 feet	91	123	157	192
20 feet	98	133	169	206
21 feet	104	142	181	220
26 feet	139	188	239	292

We recommend a minimum factor of safety for 3 for determining both allowable end bearing and allowable skin friction capacities. The pier weight for upward capacity does not require a factor of safety. The recommended capacities may be increased by a factor of 1/3 when considering temporary wind and seismic load conditions.

The ultimate capacities of other pier sizes not shown above may be determined based on the proportional surface areas of the pier sides and end. The presented capacities are based on the strength of the soils and weight of the concrete pier; the actual pier capacities may be limited to lower values based on the pier section properties. Total settlement of structures supported on pier foundations that extend to at least a depth of 18 feet below the existing ground surface and embedded at least 1 foot into the limestone bedrock will be less than 3/4 inch. Differential settlement will be less than 1/2 inch.

9.2 LPile v2019 Soil Parameters for Lateral Capacity

Lateral loads on drilled pier foundations may be resisted by the adjacent soils and by the section properties of the drilled pier. The lateral capacity of a drilled pier pile may be determined by performing LPILE analyses using the following soil parameters:

Boring B-1		LPILE v2019 Input Parameters for Undrained Conditions						
Layer Number	Depth (feet)	Soil Type	Cohesion (psf)	Strain factor ε_{50} or k_{rm}	Uniaxial Compressive Strength (psi)	Initial Modulus of Rock Mass (psi)	RQD (%)	Effective Unit Weight (pcf)
1	0 to 3	Stiff Clay w/o FW	1,500	---	---	---	---	115
2	3 to 18	Stiff Clay w/o FW	3,000	0.0060	---	---	---	120
3	18 to 26	Weak Rock	---	0.0005	8,490	50,000	53	165

10.0 SOLAR ARRAY DRIVEN PILE FOUNDATIONS

10.1 Pile Installation

10.1.1 Materials

G2 obtained new, 10-foot long non-galvanized steel W6x9 test piles for the six (6) test piles. A 1-1/4-inch diameter hole, centered 2-3/4 inches below the top of the pile, was precut through each beam web to provide a connection point for the pile tension test apparatus. The assumed W6x9 pile properties are presented below.

Property	W6x9
Depth – d	5.90 inches
Flange Width – b_f	3.94 inches
Flange Thickness – t_f	0.215 inches
Web Thickness – t_w	0.170 inches
Moment of Inertia – I_x	16.4 in^4
Section Area – A	2.68 in^2
Young's Modulus – E_s	29 $x10^6$ psi
Yield Stress – F_y	50 ksi
Hot dip Galvanization	0 mils

10.1.2 Procedure

The GPS coordinates for each test area was determined using hand-held GPS (Garmin® eTrex) in conjunction with Google Earth® software. A Bobcat E85 compact excavator fitted with a Furukawa FRD KF6 hydraulic impact hammer, having an energy class of 1,000 ft-lbs, an operating weight of 1,120 pounds and an adjustable maximum impact rate of 1,500 bpm, was used to drive each test pile to the final test embedment depth. The G2 field staff used the maximum impact rate setting after maximum push depths were encountered. A proprietary drive head was used with the impact hammer to maintain pile head seating and alignment. During driving operations, pile plumbness was monitored and adjusted as needed.

At initiation of pile installation, the weight of the drive hammer and the hydraulic force of the excavator arm were used to push the piling into the ground between 3 and 4 feet below grade before starting the vibratory hammer. During installation, the relative drivability per location was recorded as a function of the continuous drive time versus depth of penetration. Actual continuous drive times for each test pile ranged from 10 to 40 seconds. The GPS coordinates, pile embedment depths, and observed drivability rates for each test location are presented in Appendix D, Figure No. 38.

The web and flanges of the top and bottom of each pile were examined for damage that may have resulted during the installation process. The damage to the tops of the piles can be characterized as minor to no deformation of the web and flanges, and is primarily attributed to adjustments to the alignment of the driving helmet during driving operations. The damage to the bottoms of the piles can be characterized as minor to severe deformation of the web and flanges, and is primarily attributed to the driving operations into the underlying weathered limestone bedrock. Photographic documentation of the top and bottom of each pile is presented in Appendix C, Figure Nos. 35 through 37.

10.1.3 Drivability Considerations

We anticipate solar array foundations may be installed using a 700 ft-lb energy class (Vermeer PD-10 or equivalent) hydraulic impact hammer. Weathered bedrock is present throughout the entire development area at depths ranging between 3-1/2 and 8 feet. We anticipate the Vermeer PD-10 or equivalent hammer will experience driving refusal of W6x9 steel piles at relatively short penetrations (6 to 12 inches) into the weathered bedrock. Where piles must extend more than 6 inches into the bedrock, the contractor should predrill 4-inch diameter relief holes extending to the required pile embedment depths.

Due to the varying bedrock quality, distribution and depth, additional pile load testing using 4-inch diameter predrilled relief holes should be performed prior to final design of pile foundations. In addition, further delineation of the bedrock interface should be performed prior to final design of pile foundations. Geophysical surveys, such as seismic refraction or shear-wave velocity methods may be performed within solar array areas to gather cross-section profiles and associated rippability data of the soil and bedrock stratification beneath the site.

10.2 Axial Capacity

10.2.1 Tension Pile Load Test Procedure

Axial uplift (tension) pile load tests were performed in general conformance with the procedures described in the ASTM D3689 method of testing for Deep Foundations under Static Axial Tensile Load. The load tests were performed within 1 to 3 hours after the piles were installed. The complete results of the pile load tests are presented in Appendix D, Figure Nos. 40 through 45.

A Bobcat E85 compact excavator, with an operating weight of 18,977 pounds, was used as a tensile reaction against the test pile load. An Enerpac hydraulic load jack, with a rated capacity of 20 tons, was used to apply the tensile load to the top of the test pile. A Crosby Bluelink Dynamometer wireless pressure-to-load transducer, with a 14,300-pound capacity and an accuracy of 0.2 percent, was fitted between the test pile and load jack. The resulting jack loads during the load test were transmitted wirelessly and displayed on a hand-held computer.

Two (2) Starrett manual dial gauges, with a resolution of 0.001 inches, were mounted to opposing sides of the pile web using magnetic bases. Two (2) 10-foot long steel L-channel reference beams were supported above grade and adjacent to opposing sides of the test pile. The dial gauges were extended to a vertical position over and in contact with the reference beams.

Pile Tension Load Test Setup

The proposed load sequence was recommended by G2. Each pile was incrementally loaded to the design load of 2,000 pounds. Once this load was reached, the pile was unloaded and each pile was loaded until tension load failure was experienced (greater than 0.25 inches of deflection). The piles were then unloaded and reloaded to determine the load at which 1 inch deflection occurs. After 1 inch of deflection was achieved, the piles were again unloaded, the gauges were reset to zero, and the piles were reloaded until an additional deflection of 1/2 inch to evaluate residual pile capacity. Incremental load hold times were generally maintained for approximately 1 minute. A summary of the as-tested tension loads measured at the indicated deflections is presented in **Appendix D, Figure No. 39.**

10.2.2 Shrink/Swell Potential

An approximate average of 11 inches of sandy clay tilled earth with organic matter contents greater than 3 percent is generally present at the ground surface of each soil boring and test pit location. The tilled earth is underlain by native lean to fat clay soils with a high potential for shrinkage or swelling with decreases or increases in moisture content. Groundwater was not encountered during or upon completion of test pit excavation operations. Fine roots from overlying vegetation extend into the native clay to depths of 3 and 4 feet below the ground surface of test pits PLT-2 and TP-5.

The regional climate in the vicinity of Mammoth Cave, KY is considered to be humid with an approximate Thornthwaite Moisture Index (TMI) of 60. Based on an evaluation using the Foundation Performance Association (FPA) method for estimating the depth of the moisture active zone, the upper 5-1/2 feet of the subsurface soils is considered susceptible to periodic moisture fluctuations.

Based on an evaluation using the Texas Department of Transportation method (TxDOT Designation Tx-124-E) for estimating Potential Vertical Rise (PVR), an unloaded surface structure has a PVR of approximately 1-inch if the native undisturbed fat clay to a depth of 5-1/2 feet were allowed to transition from a "dry" condition to a "wet" condition. The estimated PVR value indicates the possible vertical movement of the ground surface relative to existing grade over time.

In combined consideration of the climatic conditions, observed soil consistency (generally stiff to very stiff), soil impermeability, as-tested natural moisture contents, observed depth of root growth, and observed groundwater depth, the effects of seasonal fluctuations in soil moisture are anticipated to be moderate. Based on these combined conditions, we estimate an effective active zone of moisture fluctuation extending to an approximate depth of 2-1/2 feet for use in foundation analyses. The effective active zone represents the depth to which the moisture content of the near-surface clay is reasonably expected to fluctuate seasonally.

During periods of seasonal drying, it is expected that the upper tilled earth (topsoil) and native clay may shrink away from contact with the sides of the piling. Since any gaps that might develop adjacent to a driven pile could result in loss of frictional skin resistance on the sides of the steel piles, any axial capacity within the upper 4 feet of embedment should not be included in the axial capacity design.

During periods of seasonal wetting, it is expected that the upper clay may swell. Any swelling within the clay will manifest as ground surface rise. The rising ground will, in turn, impose frictional tension forces on the sides of the steel piles. For tension capacity design of driven steel piles, it is recommended that an ultimate frictional skin tension (negative) force of 500 psf be assumed as applied to the boxed perimeter of the pile section. The ultimate negative skin friction value is derived based on the estimated adhesion relative to the undrained shear strength of these upper soils during periods of elevated moisture contents.

10.2.3 Adfreeze

The frost depth in the Cave City, KY area is approximately 24 inches. Lightly loaded PV array pile foundations may be susceptible to the effects of frost penetration that may occur within the near-surface soils. In the nearby Mammoth Cave, KY area, the mean annual air temperature is 57°F, and the air freezing index with a 25-year return period is 378. The average annual precipitation is 52 inches, and the average annual snowfall is 6 inches. Based on these conditions, it is recommended that an effective frost depth of 24 inches be assumed in design of PV array pile foundations.

The near-surface clay soils are identified within the U.S. Army Corps of Engineers (USACE) as frost group F3, which is indicative of soil with a high degree of susceptibility to frost penetration. Therefore, adfreeze shear stress may develop along the upper embedded portion of sides of driven piles that could cause upward tension on the piles. For tension capacity design of driven steel piles, it is recommended that an ultimate adfreeze (negative) force of 1,000 psf be assumed for the upper 24 inches of tilled earth (and topsoil) and native clay as applied to the boxed perimeter of the pile section.

10.2.3 Driven Test Pile Axial Capacity

Given the varied strength of the native clay throughout the site, we recommend the use of two (2) sets of parameters (Capacity Area Nos. 1 and 2) for use in axial compression and tension design of driven steel piles. The design parameters are intended to represent the observed ultimate capacities in the vicinity of the indicated test pile locations in order to simplify design of pile foundations. The capacities depicted on the following page are based on the field test results that were performed in a relatively short timeframe.

Capacity Area No.	Test Locations	Ultimate Axial Capacity Soil Parameters			
		Depth (feet)	Ultimate Skin Friction (psf) (1)	Ultimate End Bearing (psf) (2)	Effective Unit Weight (pcf)
1	PLT-1, PLT-3 and TP-7	0 to 2-1/2	0	---	120
		2-1/2 to 5	440	9,000	125
		5+	740	15,000	125
2	PLT-2, TP-5 and TP-6	0 to 2-1/2	0	---	120
		2-1/2 to 5	80	7,500	125
		5+	710	15,000	125

1. Ultimate skin friction assuming driven steel pile foundations applied to the boxed perimeter of the pile section.
2. Ultimate end bearing based on the boxed area of the W-section pile tip.

Given the relatively high confidence in the data obtained by direct pile tension load tests, a relatively low factor of safety of 1.5 may be used in determining allowable design skin friction values. Given the ultimate end bearing parameters were based on indirect field tests, a factor of safety of 3.0 may be used for determination of allowable end bearing.

The minimum pile embedment depth should consider the negative tension loads due to soil swell and adfreeze as applied to the boxed perimeter of pile section. The recommended tension forces presented below do not need to be evaluated as an additional tension load when evaluating transient wind or earthquake loads, but should be considered independently as a static load to be overcome by pile embedment below the effective active zone depth. No multiplier or factor of safety should be applied to the ultimate negative skin friction values presented below:

Type of Static Tension on Pile	Depth (feet)	Ultimate Negative Skin Friction (psf) (1)
Shrink/Swell	0 to 2-1/2	-500 psf
Adfreeze	0 to 2	-1,000 psf

1. Ultimate negative skin friction assuming driven steel pile foundations applied to the boxed perimeter of the pile section.

10.3 Lateral Capacity

10.3.1 Lateral Pile Load Test Procedure

Lateral pile load tests were performed in general conformance with the procedures described in the ASTM D3966 method of testing for Deep Foundations under Lateral Load. The load tests were performed within 1 to 3 hours after the piles were installed. The complete results of the pile load tests are presented in **Appendix D, Figure Nos. 40 through 45.**

A Bobcat E85 compact excavator, with an operating weight of 18,977 pounds, was used as a lateral reaction against the test pile load. An Enerpac hydraulic load jack, with a rated capacity of 20 tons, was used to apply the lateral load to the side of the test pile. A Crosby Bluelink Dynamometer wireless pressure to load transducer, with a 14,300-pound capacity and an accuracy of 0.2 percent, was fitted between the test pile and load jack. A top of the beam clamp was centered approximately 6 inches above grade and used to connect the load transducer to the test pile. The resulting jack loads during the load test were transmitted wirelessly and displayed on a hand-held computer.

Two (2) Starrett manual dial gauges, with a resolution of 0.001 inches, were mounted using magnetic bases to the same side of the pile web above the beam clamp and approximately 6 inches above the ground surface. Two (2) 10-foot long steel L-channel reference beams were supported above grade and perpendicular to the load direction. The dial gauges were extended to a horizontal position parallel to the load jack and in contact with the side of the reference beam.

Pile Lateral Load Test Setup

The proposed load sequence was recommended by G2. Each pile was cyclically loaded to 3,000 pounds. Incremental load hold times were maintained for approximately 1 minute. Each pile then was unloaded and reloaded until lateral load failure was experienced. Finally, the piles were unloaded and reloaded to determine the load at which 1 inch of additional deflection occurs after failure of the pile had already been experienced to evaluate residual pile capacity. A summary of the as-tested lateral loads measured at the indicated deflections is presented in **Appendix D, Figure Nos. 39**.

10.3.2 Lateral Capacity

Based on the pile lateral load tests, LPILE analyses were performed using LPILE (version 2019.11.03) to "reverse model" the observed deflections at the applied lateral loads. The "stiff clay without free water" model was assumed for lateral capacity analyses based on the observed soil conditions at each test location. The LPILE analyses were performed using a cyclic loading frequency of 2 cycles with the load applied 6 inches above the ground surface. The appropriate soil parameters, including soil effective unit weight, soil modulus (k), and soil cohesion were adjusted until 1/2 inch of deflection was approximated 6 inches above the ground surface at the actual applied lateral load observed in the field for 1/2 inch of deflection 6 inches above the ground surface. The model parameters were then iteratively refined until the calculated 1/4-inch and 1-inch deflections 6 inches above the ground surface were approximated at the actual test loads. The resulting modeled soil parameters based on the actual deflections and load conditions are presented in **Appendix E, Figure No. 46.**

To determine the appropriateness of the selected design parameters, the modeled LPILE soil parameters were used to calculate the modeled deflections for concurrence comparison to the actual deflections. For test piles with embedment depths of 5 feet or less, the modeled deflection of the actual applied lateral load is significantly higher than the 1-inch of deflection observed in the field. The resulting poor concurrence is primarily attributed to bedrock disturbance during actual pile driving operations which the software conservatively misinterprets the actual bedrock as stiff clay. We recommend additional pile load testing using 4-inch diameter predrilled relief holes be performed prior to final design of pile foundations to further refine the modeled LPILE soil parameters. A summary of the modeled deflections and actual deflections for each of the lateral load tests are presented in Appendix E, Figure No. 47.

Given the varied strength of the native clay throughout the site, we recommend the use of two (2) sets of composite LPILE soil parameters (Capacity Area Nos. 1 and 2) for use in design evaluation of the lateral capacity of driven steel piles. We recommend the use of the following composite LPILE soil parameters:

Capacity Area No.	Test Locations	LPILE Soil Parameters				
		Depth (feet)	LPILE Soil Type	Cohesion (psf)	ε50	Unit Weight (pcf)
1	PLT-1, PLT-3 and TP-7	0 to 1	Stiff Clay w/o Free Water	550	0.0150	110
		1+	Stiff Clay w/o Free Water	2,800	0.0064	125
2	PLT-2, TP-5 and TP-6	0 to 1	Stiff Clay w/o Free Water	500	0.0200	110
		1 to 3	Stiff Clay w/o Free Water	1,100	0.0100	125
		3+	Stiff Clay w/o Free Water	3,600	0.0050	125

LPILE analyses using the soil parameters presented above were again performed by applying loads (ranging from 1,000 to 8,000 lbs) at 6 inches, 48 inches, and 72 inches above the ground surface to W6x9 driven piles with embedment depths ranging between 5 and 10 feet. No axial load was applied when performing the LPILE lateral capacity analyses. The LPILE analyses were performed with an assumed cyclic loading frequency of 1,000 cycles. A summary of the computed LPILE lateral capacities is presented in Appendix E, Figure No. 48.

The load-deflection curves for the computed LPILE lateral capacities presented are based on the use of W6x9 steel piles that do not include any calculated factor of safety reduction. Since the capacities were derived from direct static load tests and are limited by the indicated deflection criteria, no additional factor of safety needs to be applied in determining the design lateral capacities of the piles. The computed LPILE load vs. deflection curves are presented in Appendix E, Figure Nos. 49 through 84.

11.0 CORROSIVITY CONSIDERATIONS

11.1 Below-Grade Corrosion of Steel Piles

The electrical resistivity and chemical properties of the native soils are generally considered to be the primary factors in underground corrosion of metal. The electrical resistivities of the saturated soils at depths between 2 and 3 feet are generally greater than 10,000 ohm-cm. The native soils have sulfate contents less than 5 ppm and chloride contents less than 20 ppm. In combined consideration of the electrical resistivity and chemical properties of the native soils, the underground corrosion rates may be estimated for early planning purposes using the AASHTO metal loss model as specified within the NCHRP Report 675 "LRFD Metal Loss and Service Life Strength Reduction Factors for Metal-Reinforced Systems, Transportation Research Board, 2011".

The metal loss rate due to corrosion is initially higher during the first 2 years and decreases over time as a corrosion by-product film is gradually formed on the metal surface. Based on our analysis for early planning, we recommend using the estimated metal loss rates of zinc and steel presented in the table on the following page.

AASHTO (2009) Metal Loss Model	
Zinc Loss Rate	0 to 2 years: 0.59 mils/year (15 μm/year) per side 2+ years: 0.16 mils/year (4 μm/year) per side
Steel Loss Rate	0.47 mils/year (12 μm/year) per side
Estimated Total Loss of Metal upon 35 years (1)	Zinc: Completely consumed in 16 years Steel: 9.0 mils (228 μm) per side

(1) Based on steel W6x9 pile with a minimum zinc galvanization thickness of 3.4 mils (86 μm)

All driven piles should have a corrosion allowance (CA), consisting of zinc galvanization and sacrificial steel, to maintain the integrity of the nominal steel section needed to resist the applied load for the design service life. ASTM A123 recommends a minimum zinc galvanization thickness greater than 3.4 mils (86 μm) be utilized for structural steel members with maximum thicknesses between 187 and 250 mils. The estimated metal loss of different steel sections can be calculated using the provided metal loss rates as applied to each side of the steel section.

No factor of safety or localized pitting considerations were applied in the calculation of any of the values presented above. In-situ tests, such as “NACE RP 0502 D3.4.7- In-situ Soil LPR (Linear Polarization Resistance)” and “NACE RP 0502- D3.4.5 & D3.4.6 or Elog-I” using bare and galvanized zinc coated steel piles may be performed to better define the magnitude and delineation of underground corrosion potential across the site.

11.2 Above-Grade (Atmospheric) Corrosion of Steel Piles

We determined the corrosion rates of metal from atmospheric exposure in general accordance with the International Standards ISO 9223 and 9224. The “Time of Wetness”, average chloride deposition rate and atmospheric concentration of sulfur dioxide are generally considered to be the primary factors in above-grade corrosion of metal.

The “Time of Wetness” (TOW) is defined as the hours per year of surface wetness, and is influenced by dew, rainfall, melting snow and high humidity levels. The local climate is humid with an average of 52 inches of annual rainfall and 6 inches of annual snowfall. The annual mean temperature is 57 degrees and generally ranges between 28 degrees in January and 89 degrees in July. The annual average relative humidity at the project site is approximately 60 percent. Based on the local climate conditions, the project site can be classified as TOW level r_4, where the time of wetness ranges between 2,500 hours and 5,500 hours per year.

The primary sources of sulfur dioxide include fossil fuel industrial plants and combustion from vehicles. The project site is located in a rural environment directly west of the town of Rowletts and approximately 3-1/2 miles north of downtown Horse Cave, Kentucky. The closest industrial power plant emitting sulfur dioxide is the 1,465MW coal-fired Mill Creek Generating Station located approximately 50 miles north from the project location. Based on the project site location and its distance away from sources of sulfur dioxide, we anticipate sulfur dioxide levels will not exceed 15μm/m^3. Therefore, the site can be classified as sulfur dioxide level P_0.

Atmospheric chlorides can be deposited on the ground surface from precipitation and wind-blown dry deposition. The primary source of atmospheric chlorides are marine environments and de-icing of roads. According to the National Atmospheric Deposition Program (NADP), the estimated total deposition of chlorides near the site in 2016 was less than 3 mg/m^2 per day. Therefore, the site can be classified as chloride level S_0.

Based on the time of wetness, average chloride deposition rate and atmospheric concentration of sulfur dioxide, the project site can be classified as corrosive category C2, which is indicative of low atmospheric corrosivity to metal. The estimated corrosion rates for metal from atmospheric exposure are presented in the following table.

	Corrosion Loss Rate during years 1 through 10	Corrosion Loss Rate during years 11 through 35	Total Loss of Metal per side at 35 years
Zinc Loss Rate	0.020 mils/year (0.5 μm/year)	0.016 mils/year (0.4μm/year)	0.591 mils (15 μm)
Steel Loss Rate	0.327 mils/year (8.3 μm/year)	0.193 mils/year (4.9 μm/year)	Not affected by corrosion due to minimum zinc thickness

(1) Based on steel W6x9 Pile with a minimum zinc galvanization thickness of 3.4 mils (85μm).

ASTM A123 recommends a minimum zinc galvanization thickness greater than 3.4 mils (85μm) be utilized for structural steel members with thicknesses greater than 187 mils. Therefore, the steel should not be susceptible to atmospheric corrosion for the design life of 35 years.

11.3 Sulfate Attack Potential on Concrete Foundations

Concrete in contact with sulfate-bearing soils is susceptible to sulfate attack. The effects of sulfate attack include concrete cracking, expansion, loss of bond between cement and aggregate, and an overall loss of concrete strength. The American Concrete Institute ACI 318 building code (Table 4.3.1) has provided recommendations to mitigate sulfate attack including the use of specific types of concrete. The sulfate content of the on-site soils is less than 5 ppm. Therefore, there should be a negligible sulfate attack potential on concrete that may be in contact with on-site soils.

12.0 SPECIAL INSPECTION CONSIDERATIONS

Any testing or verification inspection required by the building officials or the project drawings and specification should be performed by an independent testing firm in accordance with Chapter 17 of the 2018 Kentucky building code and section 1704 of the 2018 International Building Code. We recommend the following special inspections and minimum verification and testing frequencies presented below.

Special Inspection Type	Suggested Minimum Verification and Testing Frequency
Structural Steel (Production Piles)	Verification of every pile and load testing of one interior and one exterior production pile per inverter block
Structural Steel (Plates and Angles)	Every field weld in accordance with AWS D1.1
Structural Steel (Anchor Bolts)	Every structure
Foundation Subgrade	Every foundation
Earthwork	One density test per lift
Cast-in-Place Concrete and Reinforcing Steel	Every pad and/or foundation

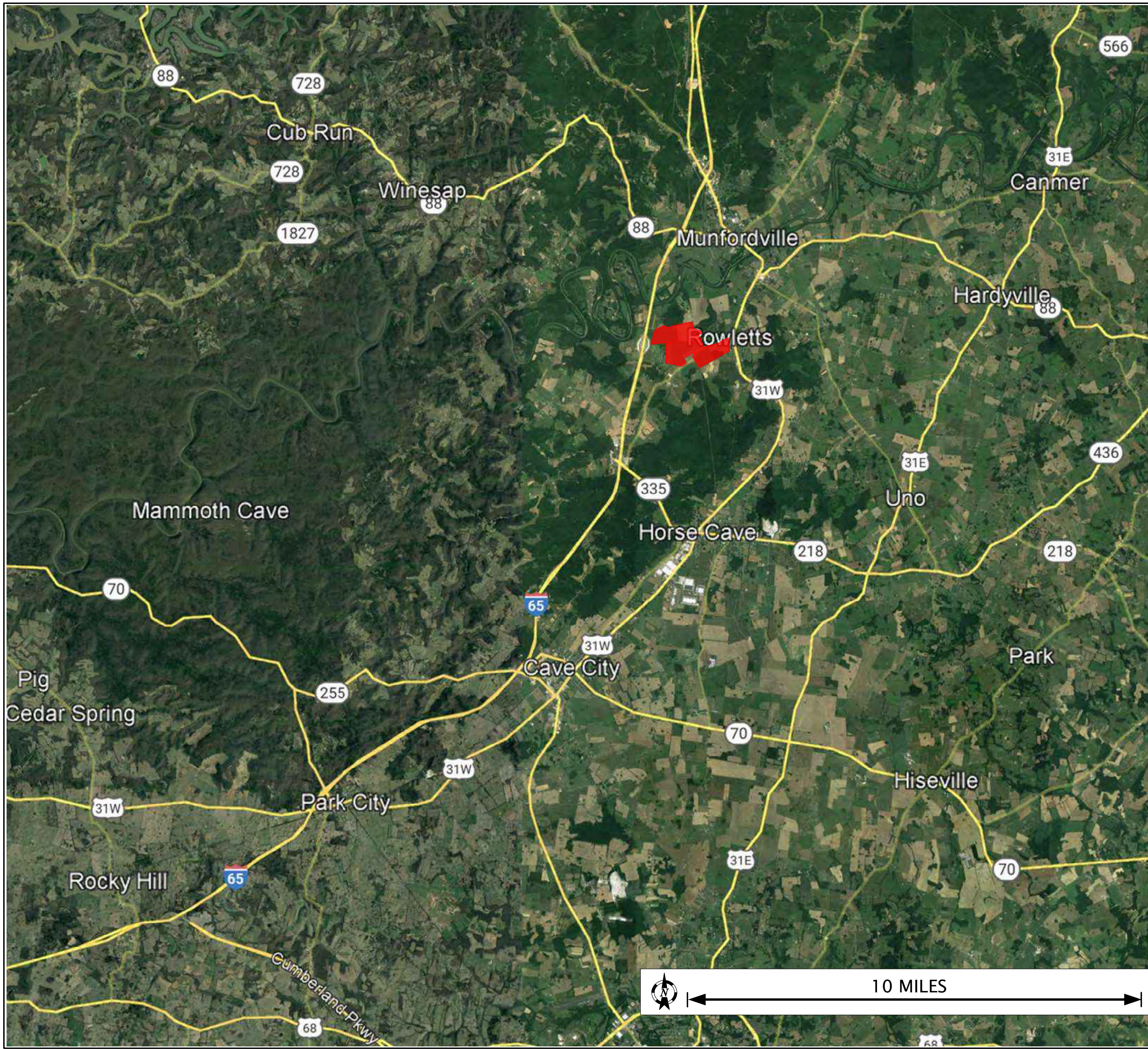
13.0 GENERAL COMMENTS

G2 has formulated the evaluations and recommendations presented in this report relative to site preparation and foundations on the basis of data provided to them relating to the location, type, and grade for the proposed site. Any significant change in this data should be brought to G2's attention for review and evaluation with respect to the prevailing subsurface conditions.

The scope of the present investigation was limited to evaluation of subsurface conditions for the support of the proposed structures and other related aspects of the development. No environmental or hydrogeological testing or analyses were included in the scope of this investigation. If changes occur in the design, location, or concept of the project, the conclusions and recommendations contained in this report are not valid unless G2 Consulting Group, LLC reviews the changes. G2 Consulting Group, LLC will then confirm the recommendations presented herein or make changes in writing.

G2 has based the analyses and recommendations submitted in this report upon the data from the soil borings, test pits and pile load tests performed at the approximate locations shown on the **Geotechnical Test Location Plan, Plate No. 2.** This report does not reflect variations that may occur between the actual test locations and the actual structure locations. The nature and extent of any such variations may not become clear until the time of construction. If significant variations then become evident, it may be necessary for G2 to re-evaluate the report recommendations.

Soil conditions at the site could vary from those generalized on the basis of tests performed at specific locations. It is, therefore, recommended that G2 Consulting Group, LLC be retained to provide geotechnical engineering services during the site preparation, excavation, and foundation construction phases of the proposed project. This is to observe compliance with the design concepts, specifications, and recommendations. Also, this allows design changes to be made in the event that subsurface conditions differ from those anticipated prior to the start of construction.



CONSULTING GROUP

PROJECT NO: 213481

PROJECT NAME: HART SOLAR

LOCATION: HART COUNTY, KY

DRAWN BY: JDC

DATE: 12-13-2021

LEGEND

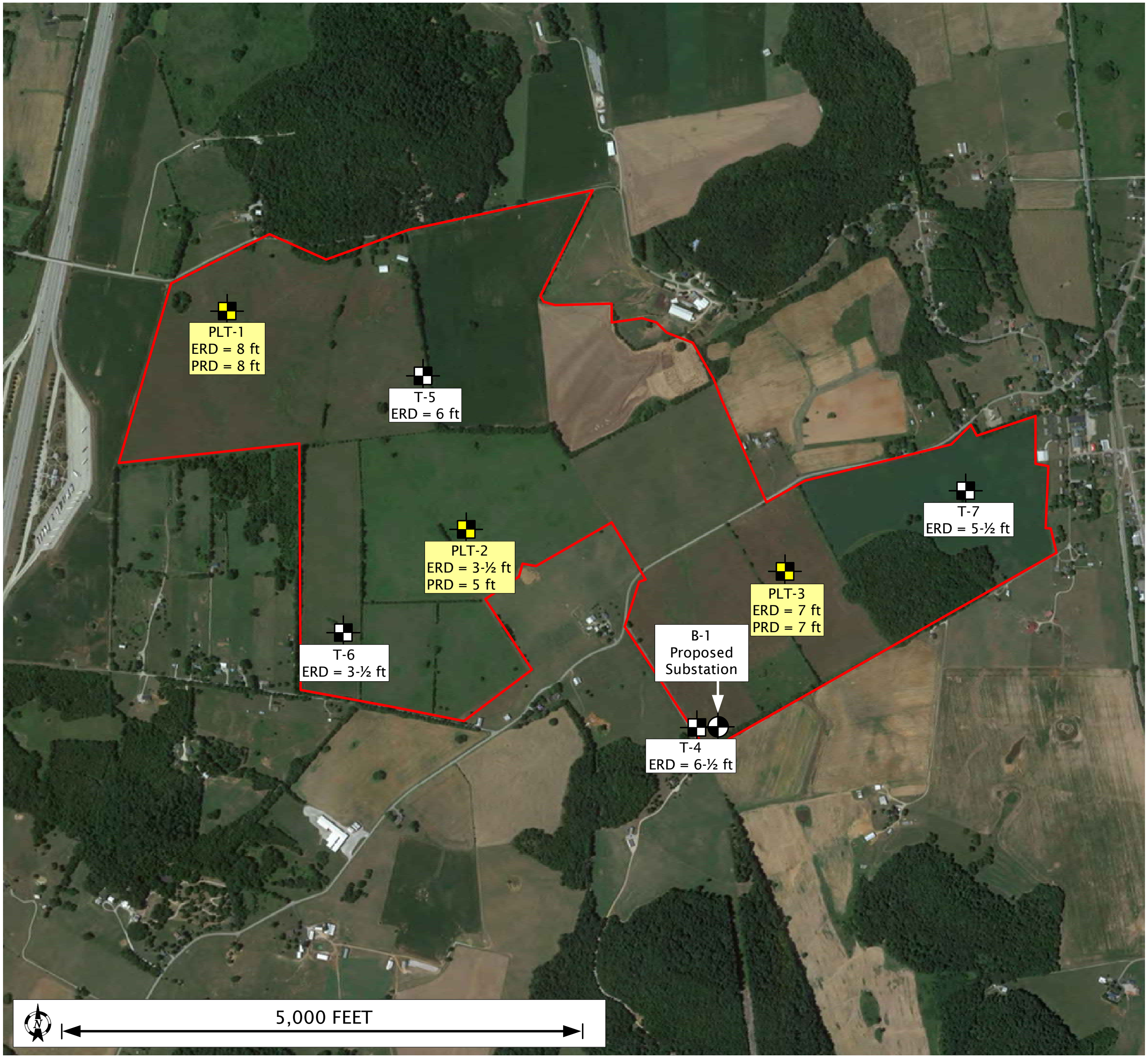
Project Boundary

SHEET TITLE

SITE VICINITY MAP

SHEET NO.

PLATE NO. 1



G2 CONSULTING GROUP

PROJECT NO: 213481

PROJECT NAME: HART SOLAR

LOCATION: HART COUNTY, KY

DRAWN BY: KAC

DATE: 10-06-2022

LEGEND

Soil boring performed by Central Star Drilling on January 29, 2022

Test pits performed by G2 Consulting Group, LLC on December 7 and 8, 2021

Pile load testing performed by G2 Consulting Group, LLC on December 7 and 8, 2021

Project Boundary

PRD = Pile Refusal Depth

ERD = Excavation Refusal Depth

SHEET TITLE

GEOTECHNICAL TEST LOCATION PLAN

SHEET NO.

PLATE NO. 2

APPENDIX A

Geotechnical Test Data

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.23206° Longitude: -85.90553°

Soil Boring No. B-1

G2 CONSULTING GROUP

SUBSURFACE PROFILE / SOIL SAMPLE DATA

GROUND SURFACE ELEVATION: 625.0 ft ±

ELEV. (ft)	Description	Depth to base (ft)	DEPTH (ft)	SAMPLE TYPE-NO.	BLOWS/ 6-INCHES	STD. PEN. RESISTANCE (N)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONF. COMP. STR. (PSF)
	Tilled Earth: Dark Brown Sandy Clay with trace gravel (3 inches)	0.3							
	Medium Brown Lean Clay with trace sand and gravel	3.0		S-1	3 4 3	7	16.6		
620.0	Very Stiff Reddish Brown Fat Clay with trace silt and sand	17.7	5	S-2	4 4 7	11	31.0	87	5,910
				S-3	4 5 6	11	30.0	95	6,820
615.0			10	S-4	5 3 3	6	26.4	100	7,750
610.0			15	S-5	3 2 2	4	27.1		
605.0	Moderately Strong Gray Faintly Weathered Limestone Recovery = 97% RQD = 53% 17-3/4 to 20-3/4 feet	20.7	20	RC-1 3 feet			0.4		8,490 (psi)
600.0	Moderately Strong Gray Faintly Weathered Limestone Recovery: 100% RQD: 96% 20-3/4 to 25-3/4 feet	25.7	25	RC-2 5 feet			0.4		11,090 (psi)
595.0	End of Boring @ 25.7 ft		30						

Total Depth: 25.7 ft
Drilling Date: January 29, 2022
Inspector:
Contractor: Tri-State Drilling, LLC
Driller: Billy

Drilling Method: 4-1/4 inch inside diameter hollow stem augers to 18 feet. Diamond-tipped core barrel to 26 feet.

Water Level Observation:
Dry during and upon completion

Excavation Backfilling Procedure:
Excavated soil

Figure No. 1

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.24278° Longitude: -85.92142°

Test Pit No. PLT-1

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA				
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 582.0 ft ±	DEPTH (ft)	SAMPLE TYPE/NO.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	PERCENT COMPACTION	UNCOF. COMP. ST. (PSF)
		Tilled Earth: Dark Brown Sandy Clay with trace gravel (10 inches) (Organic Content = 4.8%) 0.8		BS-1		33.6		
		Stiff Brown Lean Clay with trace sand and gravel, occasional sand seams (LL = 38; PI = 18) 4.5		BS-2		24.1		3000*
577.0		Very Stiff Reddish Brown Sandy Fat Clay with trace sand and gravel 8.0	5	BS-3		28.2		6000*
572.0		End of Test Pit @ 8 ft, Refusal	10					

Total Depth: 8 ft
Excavation Date: December 8, 2021
Inspector: C. SaintCyr
Contractor: G2 Consulting Group, LLC
Operator: J. Puscas

Excavation Equipment:
Bobcat E85 Excavator
24-inch bucket

Water Level Observation:
Dry during and upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Excavated soil

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.23715° Longitude: -85.91367°

Test Pit No. PLT-2

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA				
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 591.0 ft ±	DEPTH (ft)	SAMPLE TYPE/NO.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	PERCENT COMPACTION	UNCOF. COMP. ST. (PSF)
		Tilled Earth: Dark Brown Sandy Clay with trace gravel (16 inches) (Organic Content = 3.8%)	1.3	BS-1		26.1		
		Stiff Brown Lean Clay with trace sand and gravel, frequent fine roots	3.0	BS-2		22.8		3000*
		Stiff Brown Lean Clay with trace sand and gravel, frequent limestone fragments	3.5	BS-3		22.3		3000*
		End of Test Pit @ 3.5 ft, Refusal						
586.0			5					
581.0			10					

Total Depth: 3.5 ft
Excavation Date: December 8, 2021
Inspector: C. SaintCyr
Contractor: G2 Consulting Group, LLC
Operator: J. Puscas

Excavation Equipment:
Bobcat E85 Excavator
24-inch bucket

Water Level Observation:
Dry during and upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Excavated soil

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.23608° Longitude: -85.90336°

Test Pit No. PLT-3

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA				
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 618.0 ft ±	DEPTH (ft)	SAMPLE TYPE/NO.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	PERCENT COMPACTION	UNCOF. COMP. ST. (PSF)
		Tilled Earth: Dark Brown Sandy Clay with trace gravel (12 inches) (Organic Content = 3.0%)	1.0	BS-1		21.9		
		Very Stiff Reddish Brown Fat Clay with trace sand and gravel (LL = 55; PI = 32)	4.0	BS-2		26.7		4000*
613.0		Very Stiff Reddish Brown Fat Clay with trace sand and gravel, frequent limestone fragments	5	BS-3		33.1		4500*
			7.0	BS-4		35.0		6500*
		End of Test Pit @ 7 ft, Refusal						
608.0			10					

Total Depth: 7 ft
Excavation Date: December 8, 2021
Inspector: C. SaintCyr
Contractor: G2 Consulting Group, LLC
Operator: J. Puscas

Excavation Equipment:
Bobcat E85 Excavator
24-inch bucket

Water Level Observation:
Dry during and upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Excavated soil

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.23208° Longitude: -85.90618°

Test Pit No. TP-4

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA				
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 629.0 ft ±	DEPTH (ft)	SAMPLE TYPE/NO.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	PERCENT COMPACTION	UNCOF. COMP. ST. (PSF)
		Tilled Earth: Dark Brown Sandy Clay with trace gravel (12 inches) (Organic Content = 6.7%)	1.0	BS-1		43.3		
		Very Stiff Reddish Brown Fat Clay with trace sand and gravel, occasional sand seams	2.5	BS-2		29.1		6000*
624.0		Very Stiff Reddish Brown Fat Clay with trace sand and gravel	5 / 6.0	BS-3		35.2		7000*
		Very Stiff Reddish Brown Fat Clay with trace sand and gravel, frequent limestone fragments	6.5	BS-4		21.0		6000*
619.0		End of Test Pit @ 6.5 ft, Refusal	10					

Total Depth: 6.5 ft
Excavation Date: December 8, 2021
Inspector: C. SaintCyr
Contractor: G2 Consulting Group, LLC
Operator: J. Puscas

Excavation Equipment:
Bobcat E85 Excavator
24-inch bucket

Water Level Observation:
Dry during and upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Excavated soil

Figure No. 5

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.24106° Longitude: -85.91504°

Test Pit No. TP-5

G2 CONSULTING GROUP

ELEV. (ft)	PRO-FILE	SUBSURFACE PROFILE — GROUND SURFACE ELEVATION: 586.0 ft ±	DEPTH (ft)	SAMPLE TYPE/NO.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	PERCENT COMPACTION	UNCOF. COMP. ST. (PSF)
		Tilled Earth: Dark Brown Sandy Clay with trace gravel (7 inches) (Organic Content = 3.5%) 0.6		BS-1		24.8		
		Stiff Reddish Brown Fat Clay with trace sand and gravel, frequent fine roots 4.0		BS-2		21.3		3000*
581.0		Very Stiff Reddish Brown Fat Clay with trace sand and gravel, frequent limestone fragments	5	BS-3		29.6		4500*
		6.0		BS-4		20.5		6000*
		End of Test Pit @ 6 ft, Refusal						
576.0			10					

Total Depth: 6 ft
Excavation Date: December 8, 2021
Inspector: C. SaintCyr
Contractor: G2 Consulting Group, LLC
Operator: J. Puscas

Excavation Equipment:
Bobcat E85 Excavator
24-inch bucket

Water Level Observation:
Dry during and upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Excavated soil

Figure No. 6

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.23451° Longitude: -85.91766°

Test Pit No. TP-6

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA				
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 584.0 ft ±	DEPTH (ft)	SAMPLE TYPE/NO.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	PERCENT COMPACTION	UNCOF. COMP. ST. (PSF)
		Tilled Earth: Dark Brown Sandy Clay with trace gravel (11 inches) (Organic Content = 3.2%)	0.9	BS-1		18.8		
		Very Stiff Reddish Brown Fat Clay with trace sand and gravel, frequent limestone fragments (LL = 72; PI = 44)		BS-2		30.3		4000*
			3.5	BS-3		30.9		4500*
		End of Test Pit @ 3.5 ft, Refusal						
579.0			5					
574.0			10					

Total Depth: 3.5 ft
Excavation Date: December 8, 2021
Inspector: C. SaintCyr
Contractor: G2 Consulting Group, LLC
Operator: J. Puscas

Excavation Equipment:
Bobcat E85 Excavator
24-inch bucket

Water Level Observation:
Dry during and upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Excavated soil

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No. 213841

Latitude: 37.23808° Longitude: -85.89747°

Test Pit No. TP-7

G2 CONSULTING GROUP

SUBSURFACE PROFILE				SOIL SAMPLE DATA				
ELEV. (ft)	PRO-FILE	GROUND SURFACE ELEVATION: 657.0 ft ±	DEPTH (ft)	SAMPLE TYPE/NO.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	PERCENT COMPACTION	UNCOF. COMP. ST. (PSF)
		Tilled Earth: Dark Brown Sandy Clay with trace gravel (11 inches) (Organic Content = 7.0%) 0.9		BS-1		30.3		
		Very Stiff Reddish Brown Fat Clay with trace sand and gravel (LL = 52; PI = 33) 3.0		BS-2		23.5		4500*
652.0		Very Stiff Reddish Brown Fat Clay with trace sand and gravel, frequent limestone fragments 5.5	5	BS-3		28.2		7500*
647.0		End of Test Pit @ 5.5 ft, Refusal	10					

Total Depth: 5.5 ft
Excavation Date: December 8, 2021
Inspector: C. SaintCyr
Contractor: G2 Consulting Group, LLC
Operator: J. Puscas

Excavation Equipment:
Bobcat E85 Excavator
24-inch bucket

Water Level Observation:
Dry during and upon completion

Notes:
* Calibrated Hand Penetrometer

Excavation Backfilling Procedure:
Excavated soil

GENERAL NOTES TERMINOLOGY

Unless otherwise noted, all terms herein refer to the Standard Definitions presented in ASTM 653.

PARTICLE SIZE

Boulders		- greater than 12 inches
Cobbles		- 3 inches to 12 inches
Gravel	- Coarse	- 3/4 inches to 3 inches
	- Fine	- No. 4 to 3/4 inches
Sand	- Coarse	- No. 10 to No. 4
	- Medium	- No. 40 to No. 10
	- Fine	- No. 200 to No. 40
Silt		- 0.005mm to 0.074mm
Clay		- Less than 0.005mm

CLASSIFICATION

The major soil constituent is the principal noun, i.e. clay, silt, sand, gravel. The second major soil constituent and other minor constituents are reported as follows:

Second Major Constituent (percent by weight)	Minor Constituent (percent by weight)
Trace - 1 to 12%	Trace - 1 to 12%
Adjective - 12 to 35%	Little - 12 to 23%
And - over 35%	Some - 23 to 33%

COHESIVE SOILS

If clay content is sufficient so that clay dominates soil properties, clay becomes the principal noun with the other major soil constituent as modifier, i.e. sandy clay. Other minor soil constituents may be included in accordance with the classification breakdown for cohesionless soils, i.e. silty clay, trace sand, little gravel.

Consistency	Unconfined Compressive Strength (psf)	Approximate Range of (N)
Very Soft	Below 500	0 - 2
Soft	500 - 1,000	3 - 4
Medium	1,000 - 2,000	5 - 8
Stiff	2,000 - 4,000	9 - 15
Very Stiff	4,000 - 8,000	16 - 30
Hard	8,000 - 16,000	31 - 50
Very Hard	Over 16,000	Over 50

Consistency of cohesive soils is based upon an evaluation of the observed resistance to deformation under load and not upon the Standard Penetration Resistance (N).

COHESIONLESS SOILS

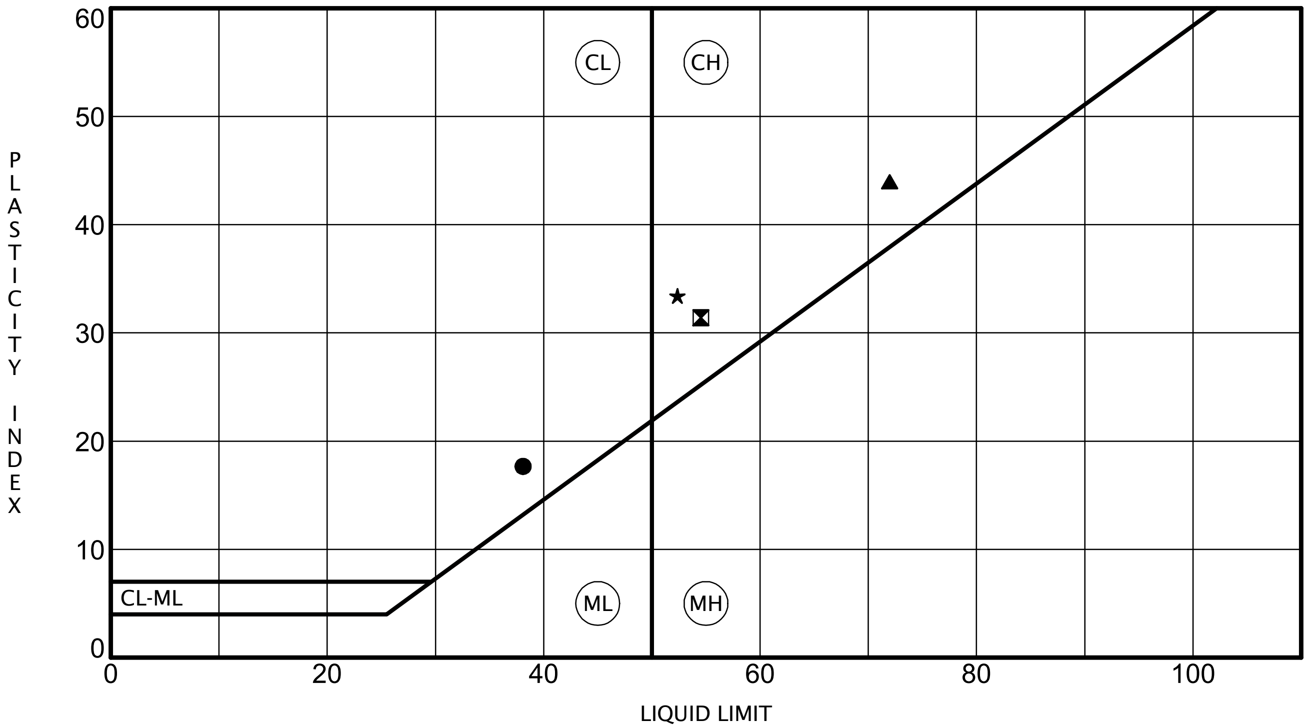
Density Classification	Relative Density %	Approximate Range of (N)
Very Loose	0 - 15	0 - 4
Loose	16 - 35	5 - 10
Medium Compact	36 - 65	11 - 30
Compact	66 - 85	31 - 50
Very Compact	86 - 100	Over 50

Relative Density of cohesionless soils is based upon the evaluation of the Standard Penetration Resistance (N), modified as required for depth effects, sampling effects, etc.

SAMPLE DESIGNATIONS

AS - Auger Sample – Cuttings directly from auger flight
BS - Bottle or Bag Samples
S - Split Spoon Sample - ASTM D 1586
LS - Liner Sample with liner insert 3 inches in length
ST - Shelby Tube sample - 3 inch diameter unless otherwise noted
PS - Piston Sample - 3 inch diameter unless otherwise noted
RC - Rock Core - NX core unless otherwise noted

STANDARD PENETRATION TEST (ASTM D 1586) - A 2.0 inch outside-diameter, 1-3/8 inch inside-diameter split barrel sampler is driven into undisturbed soil by means of a 140-pound weight falling freely through a vertical distance of 30 inches. The sampler is normally driven three successive 6-inch increments. The total number of blows required for the final 12 inches of penetration is the Standard Penetration Resistance (N).



	Specimen Identification		LL	PL	PI	Fines	M %	Classification
●	TP-1	BS-2	38	20	18		24	Brown Lean Clay
⧗	TP-3	BS-2	55	23	32		27	Reddish Brown Fat Clay
▲	TP-6	BS-2	72	28	44		30	Reddish Brown Fat Clay
★	TP-7	BS-2	52	19	33		23	Reddish Brown Fat Clay

ATTERBERG LIMITS RESULTS

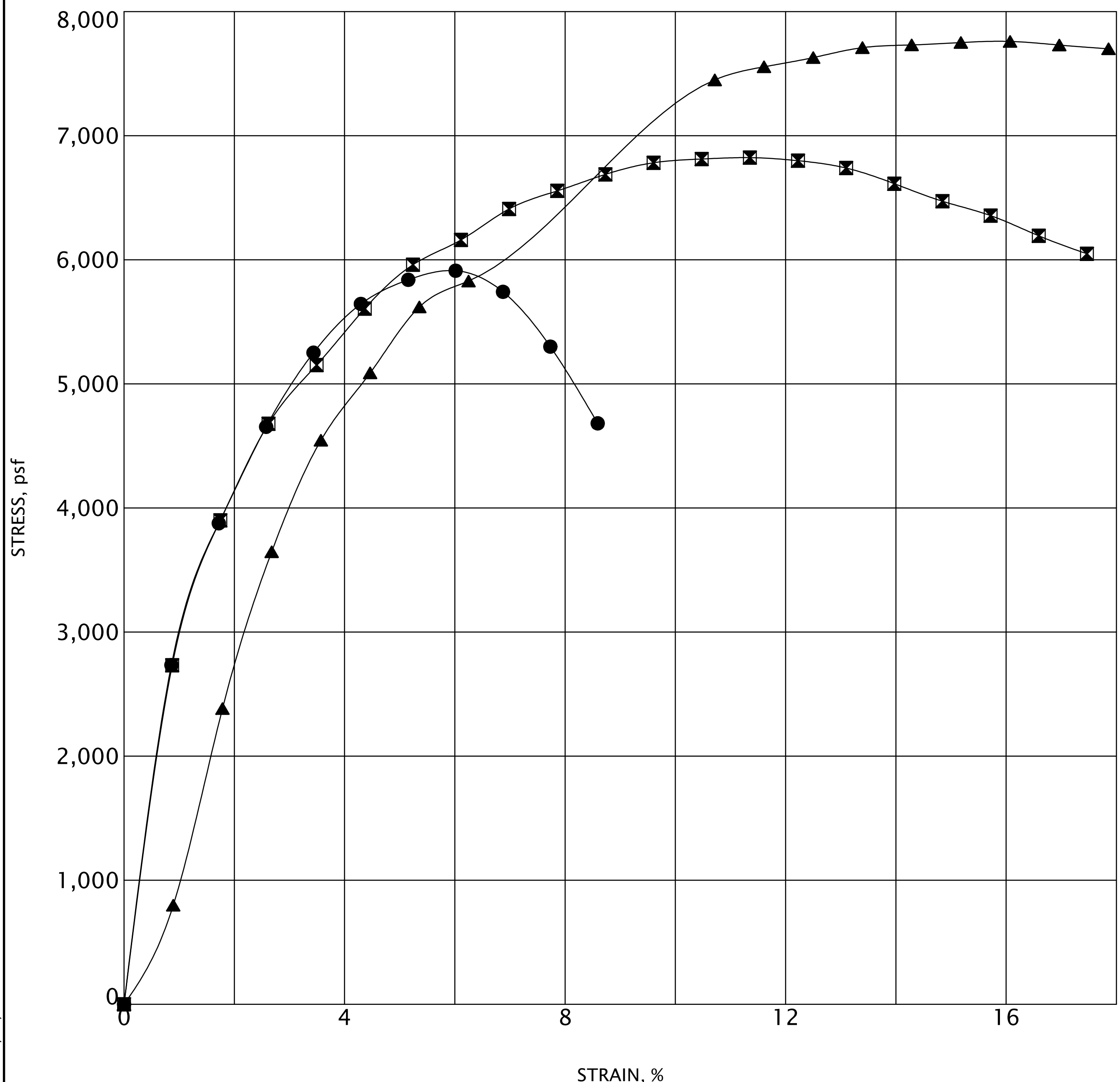
Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No.: 213841

Figure No. 10

G2 CONSULTING GROUP



	Specimen		Classification	MC%	γ_d	UC
●	B-1	S-2	Reddish Brown Fat Clay	31	87	5910
⊠	B-1	S-3	Reddish Brown Fat Clay	30	95	6820
▲	B-1	S-4	Reddish Brown Fat Clay	26	100	7750

G2 CONSULTING GROUP

UNCONFINED COMPRESSIVE STRENGTH TEST

Project Name: Proposed Hart 50 MWac Solar Power Plant

Project Location: Rowletts, Hart County, Kentucky

G2 Project No.: 213841

Figure No. 11

G2 Consulting Group

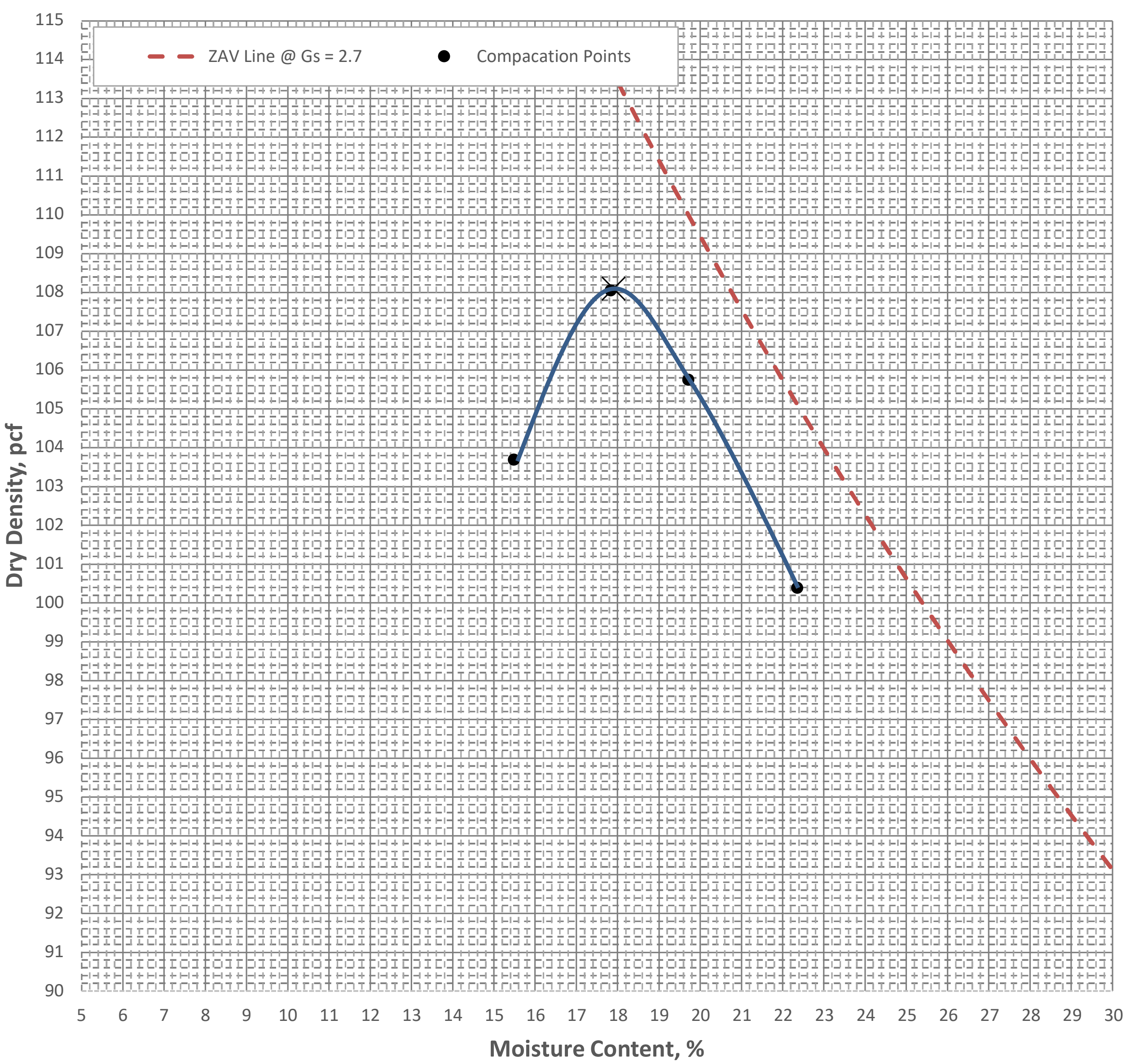
Moisture Density Curve

Project Name:	**Hart Solar Power Plant**	Date:	**12/19/2021**
Job Number:	**213841**	Method used:	**A**
Project Location:	**Rowletts, Hart County, KY**	As-received water content:	**24.2**
Sample Location:	**PLT-01**	Rammer type:	**Manual**
Sample No.:	**Composite**	Oversize correction used?	**No**
Depth of Sample:	**1 to 4 feet**	Specific gravity (estimated)	**2.7**
Soil Description:	**Brown Lean Clay**		
Technician:	**K. Crow**		

Proctor Type: **Standard Proctor, ASTM D698**

Standard maximum dry unit weight (pcf) **108.1**

Standard optimum water content (%) **17.9**



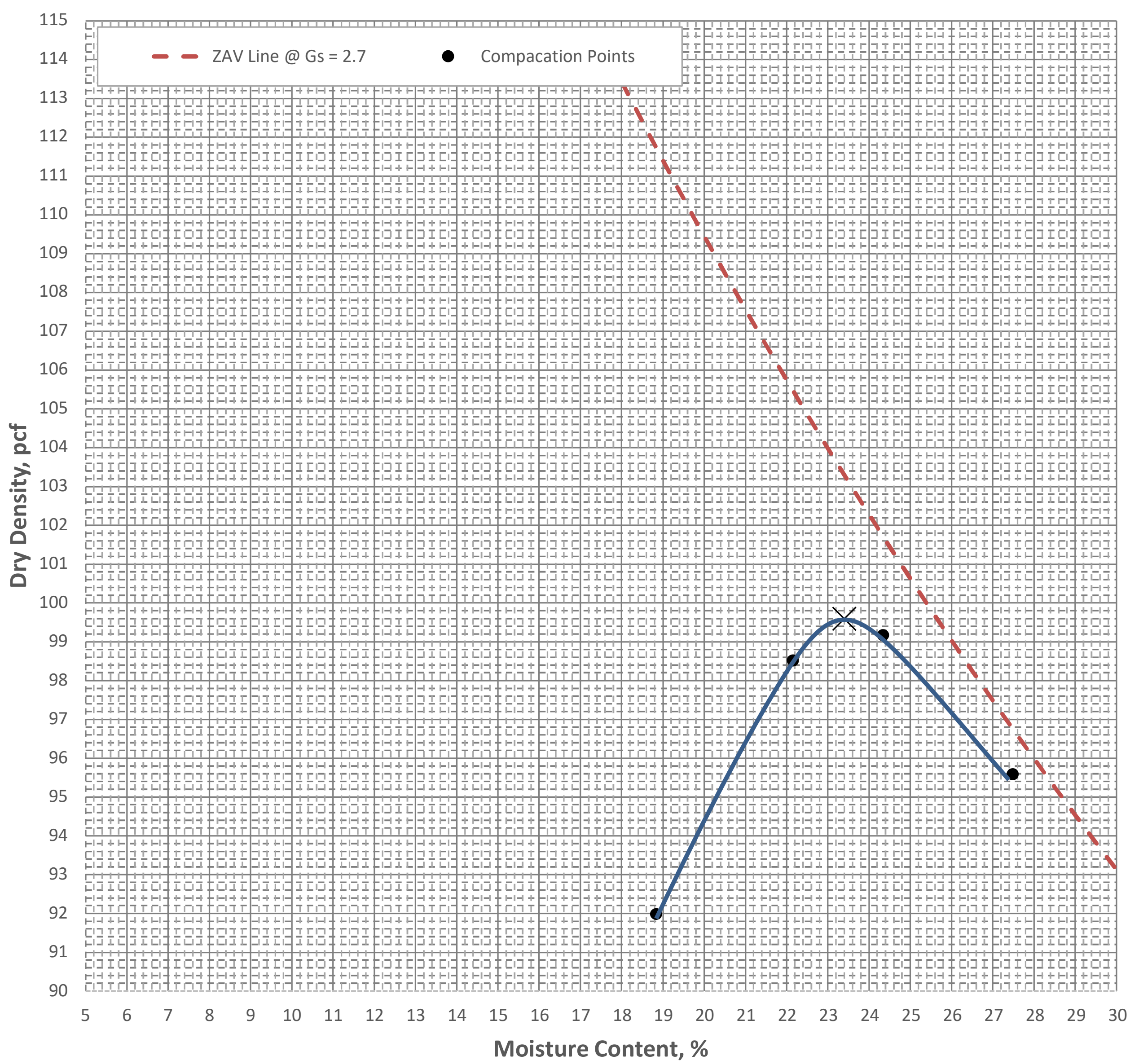
G2 Consulting Group **Moisture Density Curve**

Project Name:	**Hart Solar Power Plant**	Date:	**12/19/2021**
Job Number:	**213841**	Method used:	**A**
Project Location:	**Rowletts, Hart County, KY**	As-received water content:	**23.8**
Sample Location:	**TP-04**	Rammer type:	**Manual**
Sample No.:	**Composite**	Oversize correction used?	**No**
Depth of Sample:	**1 to 4 feet**	Specific gravity (estimated)	**2.7**
Soil Description:	**Reddish Brown Fat Clay**		
Technician:	**K. Crow**		

Proctor Type: **Standard Proctor, ASTM D698**

Standard maximum dry unit weight (pcf) **99.6**

Standard optimum water content (%) **23.4**



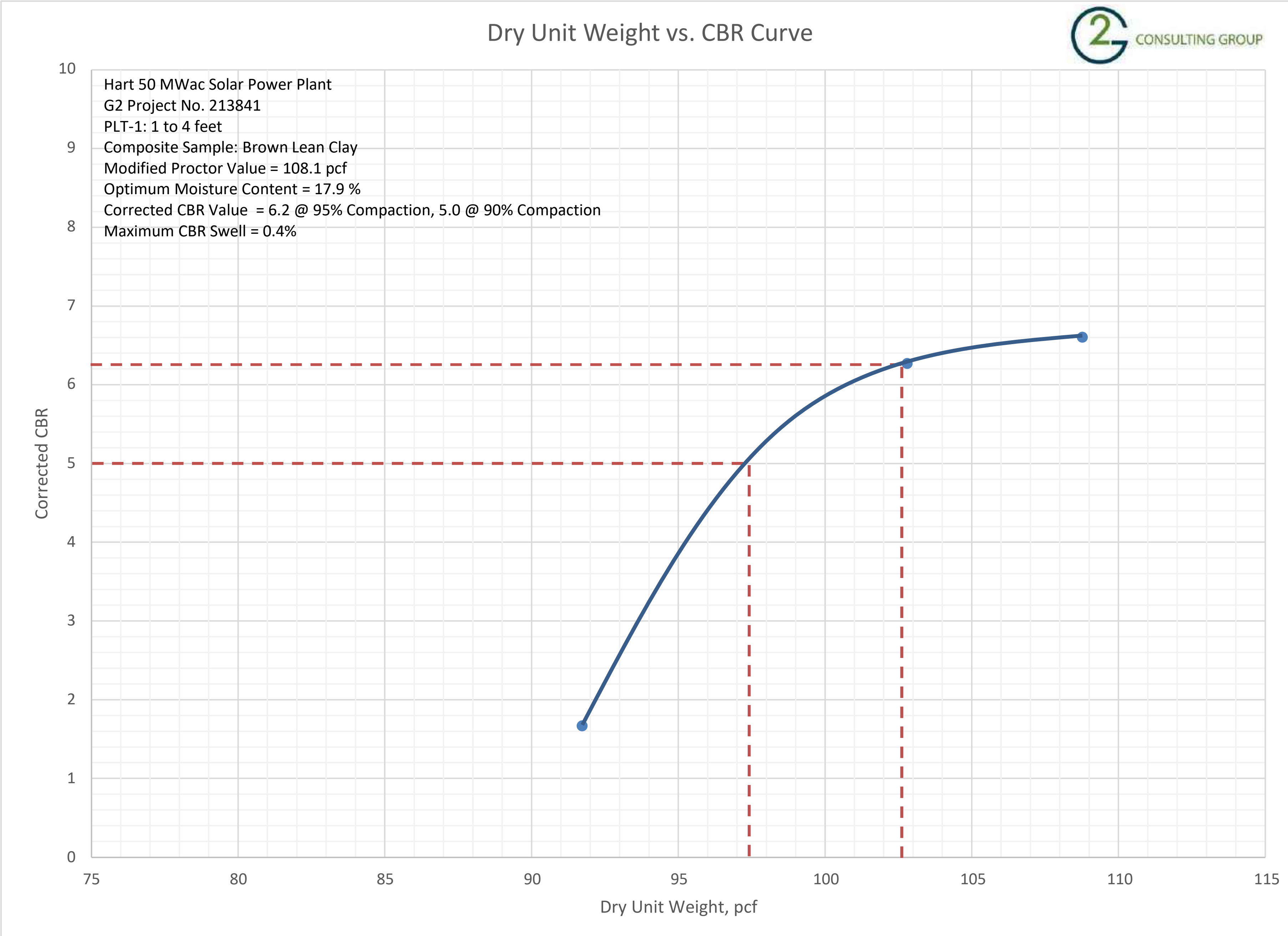
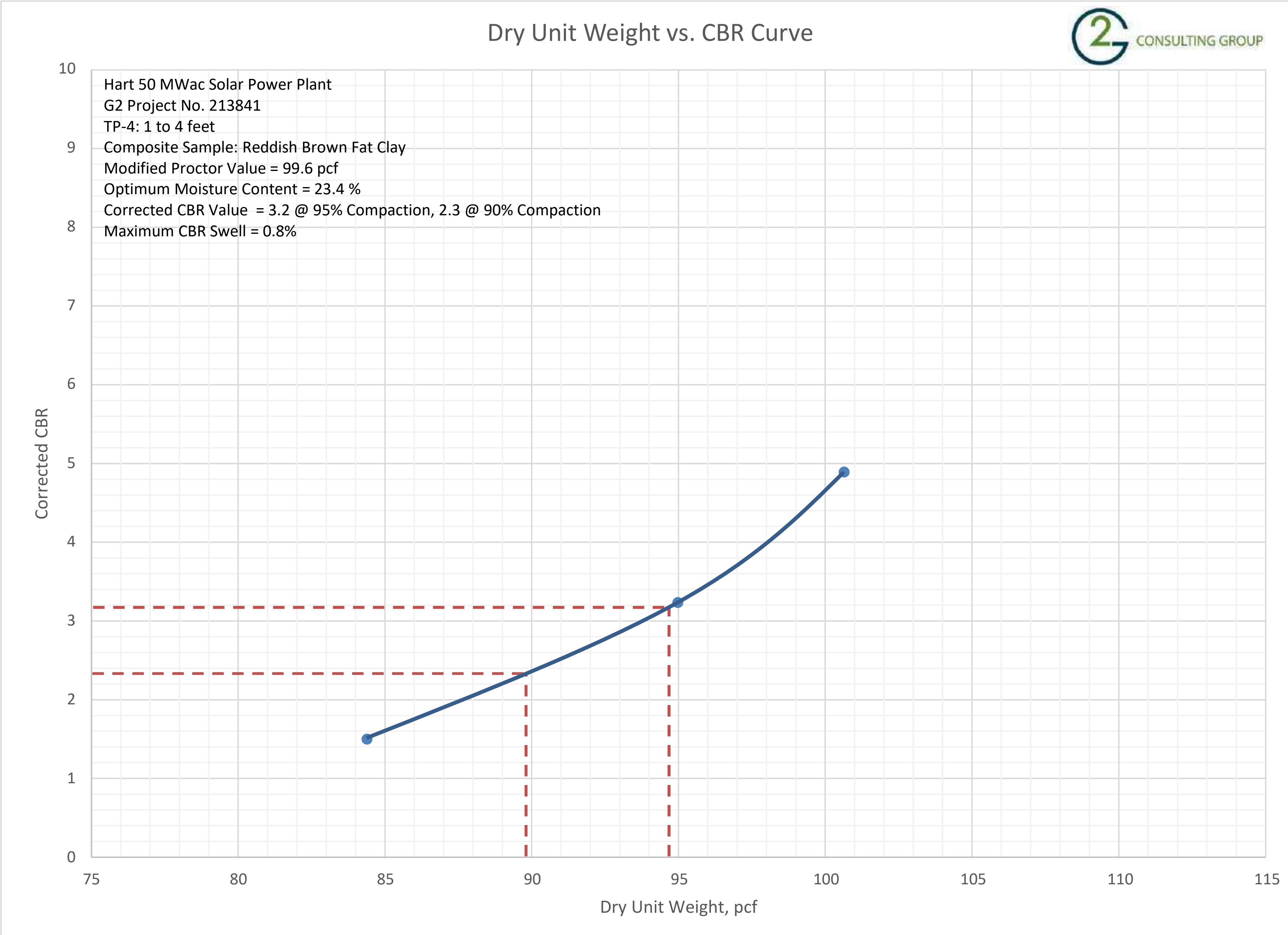

Dry Unit Weight vs. CBR Curve
G2 CONSULTING GROUP
Hart 50 MWac Solar Power Plant
G2 Project No. 213841
PLT-1: 1 to 4 feet
Composite Sample: Brown Lean Clay
Modified Proctor Value = 108.1 pcf
Optimum Moisture Content = 17.9 %
Corrected CBR Value = 6.2 @ 95% Compaction, 5.0 @ 90% Compaction
Maximum CBR Swell = 0.4%
Corrected CBR
0
1
2
3
4
5
6
7
8
9
10
Dry Unit Weight, pcf
75
80
85
90
95
100
105
110
115

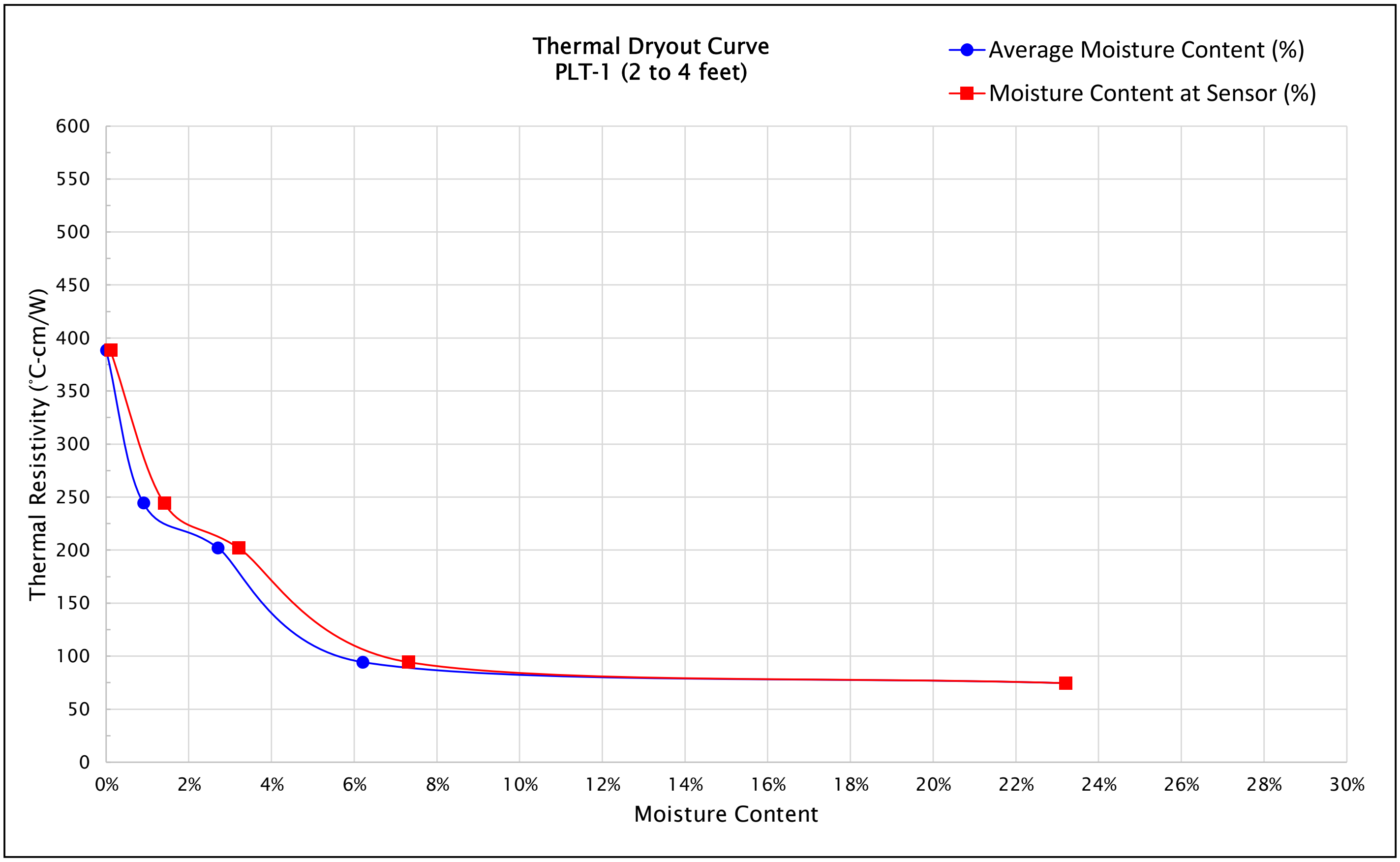
Figure No. 14



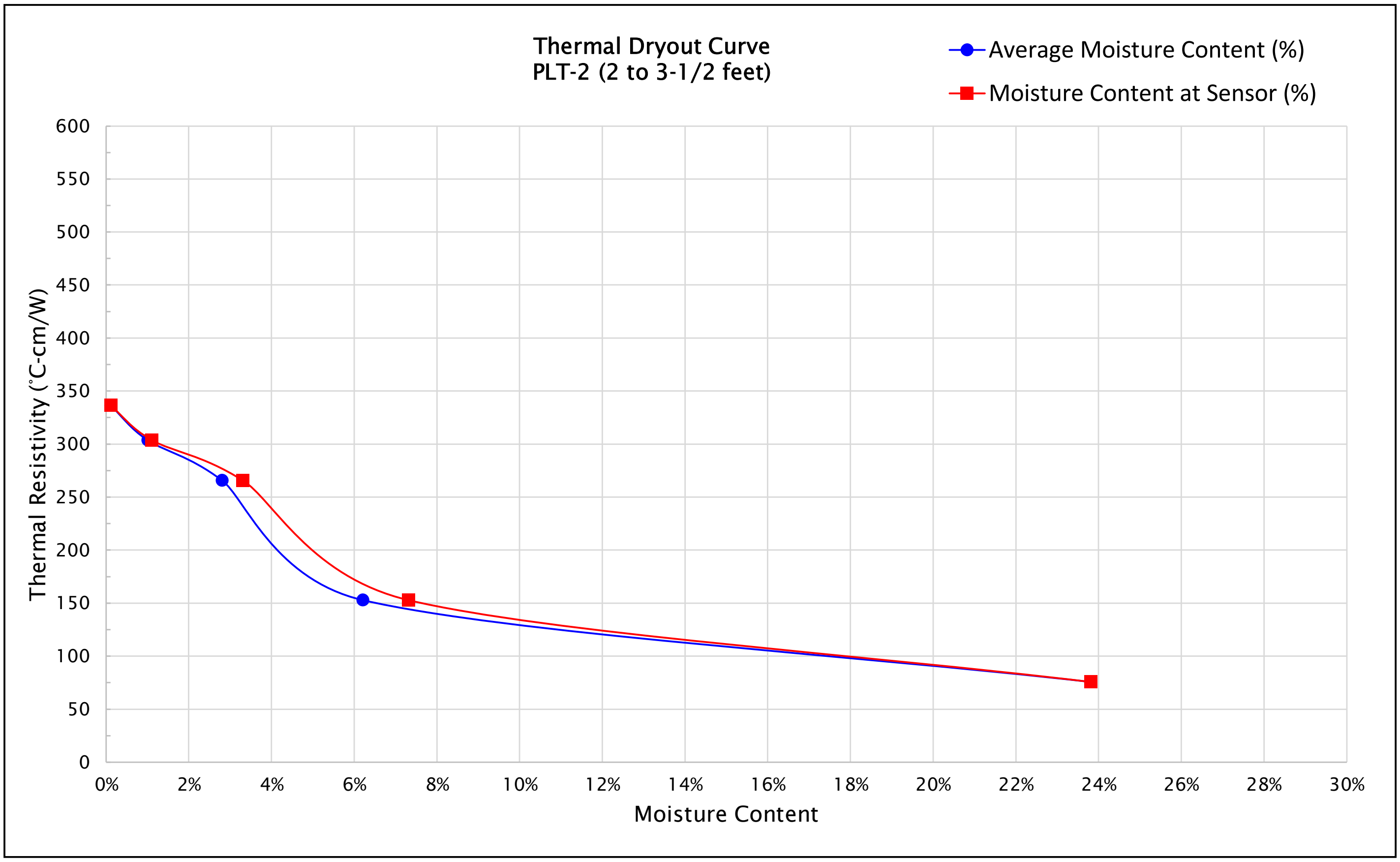
Dry Unit Weight vs. CBR Curve
CONSULTING GROUP
Hart 50 MWac Solar Power Plant
G2 Project No. 213841
TP-4: 1 to 4 feet
Composite Sample: Reddish Brown Fat Clay
Modified Proctor Value = 99.6 pcf
Optimum Moisture Content = 23.4 %
Corrected CBR Value = 3.2 @ 95% Compaction, 2.3 @ 90% Compaction
Maximum CBR Swell = 0.8%
Corrected CBR
Dry Unit Weight, pcf
0
1
2
3
4
5
6
7
8
9
10
75
80
85
90
95
100
105
110
115

Figure No. 15

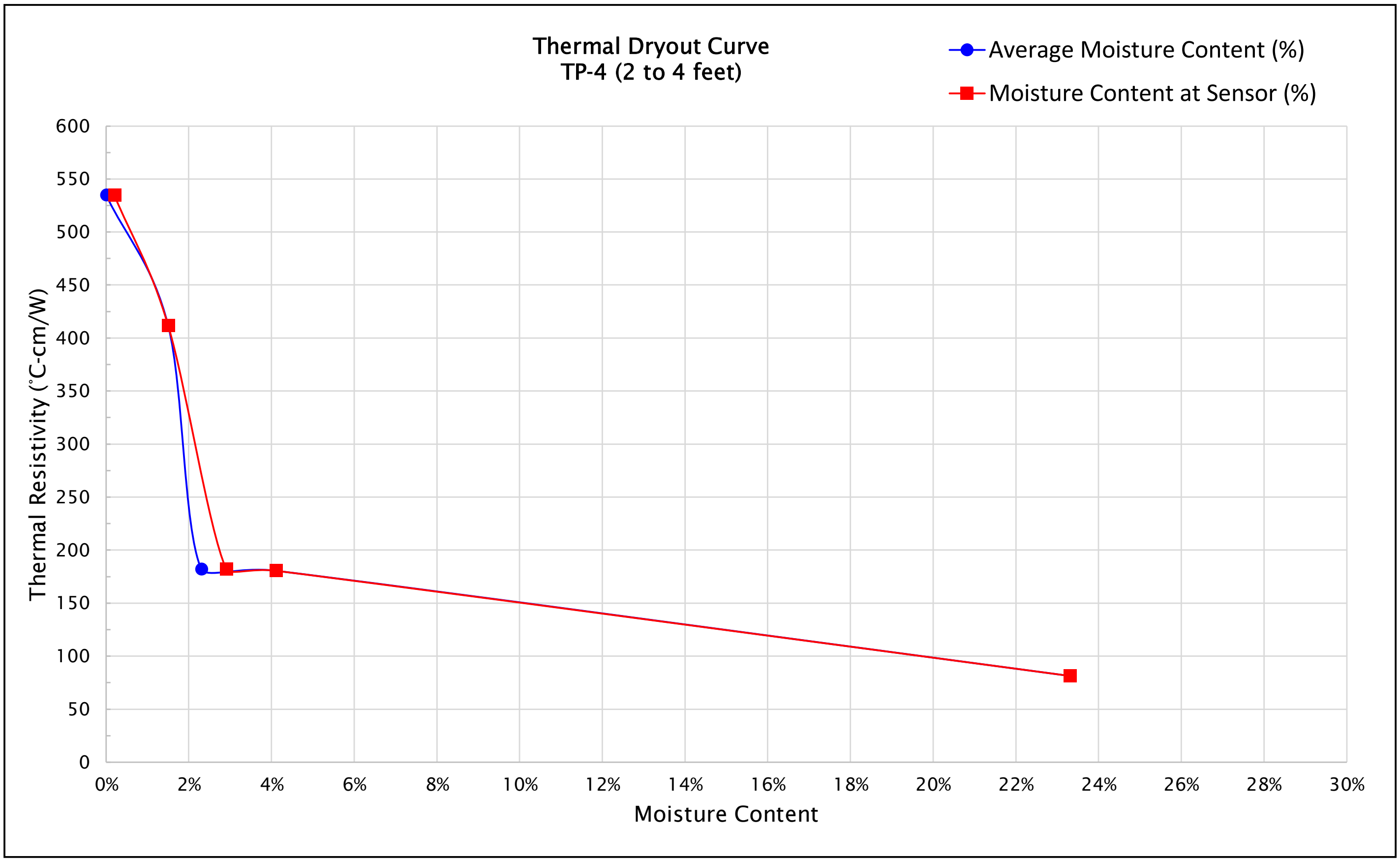
Thermal Resistivity Dryout Curves
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841



Remolded Target 85% Compaction of Standard Proctor (ASTM D698)						
Sample Description	**Dry Density (pcf)**	**Shrinkage (%)**	**Average Moisture Content (%)**	**Moisture Content at Sensor (%)**	**Thermal Resistivity (˚C-cm/W)**	**Temp (˚C)**
Location: PLT-1 **Sample Type:** Bulk sample **Sample Depth:** 2 to 4 feet below grade **Soil Type:** Brown Lean Clay	83.7	0.0%	23.2%	23.2%	75	19.8
	85.2	1.7%	6.2%	7.3%	95	20.8
<u>Thermal Resistivity Test Summary (˚C-cm/W)</u> 2% interpolated average moisture content: 219 2% interpolated moisture content at sensor: 231 2% interpolated at both average and sensor: 225 *Note: Linearly interpolated*	85.1	1.6%	2.7%	3.2%	202	20.9
	85.1	1.6%	0.9%	1.4%	245	21.6
	85.2	1.8%	0.0%	0.1%	389	21.7



Remolded Target 85% Compaction of Standard Proctor (ASTM D698)						
Sample Description	**Dry Density (pcf)**	**Shrinkage (%)**	**Average Moisture Content (%)**	**Moisture Content at Sensor (%)**	**Thermal Resistivity (°C-cm/W)**	**Temp (°C)**
Location: PLT-2 **Sample Type:** Bulk sample **Sample Depth:** 2 to 3-1/2 feet below grade **Soil Type:** Brown Lean Clay	82.8	0.0%	23.8%	23.8%	76	20.8
	84.0	1.4%	6.2%	7.3%	153	20.7
Thermal Resistivity Test Summary (°C-cm/W) 2% interpolated average moisture content: 283 2% interpolated moisture content at sensor: 288 2% interpolated at both average and sensor: 286 *Note: Linearly interpolated*	84.1	1.6%	2.8%	3.3%	266	21.4
	84.2	1.7%	1.0%	1.1%	304	21.6
	84.4	2.0%	0.1%	0.1%	337	21.3



Remolded Target 85% Compaction of Standard Proctor (ASTM D698)						
Sample Description	**Dry Density (pcf)**	**Shrinkage (%)**	**Average Moisture Content (%)**	**Moisture Content at Sensor (%)**	**Thermal Resistivity (˚C-cm/W)**	**Temp (˚C)**
Location: TP-4 **Sample Type:** Bulk sample **Sample Depth:** 2 to 4 feet below grade **Soil Type:** Reddish Brown Fat Clay	85.9	0.0%	23.3%	23.3%	82	21.5
	87.5	1.8%	4.1%	4.1%	181	20.9
Thermal Resistivity Test Summary (˚C-cm/W) 2% interpolated average moisture content: 268 2% interpolated moisture content at sensor: 330 2% interpolated at both average and sensor: 299 *Note: Linearly interpolated*	87.5	1.9%	2.3%	2.9%	182	20.5
	87.8	2.1%	1.5%	1.5%	412	21.4
	87.8	2.2%	0.0%	0.2%	535	21.2

APPENDIX B

Soil Corrosivity Test Data

In-situ Electrical Resistivity (ohm-cm)
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

			'a' spacing (feet)					
Location Number	**GPS Location**	**Direction**	**2**	**5**	**10**	**20**	**30**	**50**
PLT-2	Lat: 37.23715° Long: -85.26460°	N-S	13,400	22,000	38,300	72,700	109,000	181,000
		E-W	14,900	22,900	36,300	72,700	103,000	143,000
		Minimum	13,400	22,000	36,300	72,700	103,000	143,000
		Maximum	14,900	22,900	38,300	72,700	109,000	181,000
		Average	14,200	22,500	37,300	72,700	106,000	162,000

Note: In-situ soil electrical resistivity testing was performed following the Wenner four-pin test procedure (ASTM G57-06) using a Nilsson Model 400 resistivity meter with steel probes.

Figure No. 19

ESSENTIAL CORROSION PROTECTION

12/27/2021

G2 Consulting Group Laboratory Soil Sample Analysis Results

Project	Sample ID	As-Is Resistivity (ohm-cm)	"Wetted" Resistivity (ohm-cm)	Redox (mV)	pH	Chloride (ppm)	Sulfate (ppm)	Sulfides
213841	PLT-1	14,000	13,000	234	6.7	<20	<5	Not Present
	TP-2	13,000	13,000	260	6.6	<20	<5	Not Present
	TP-4	11,000	10,000	256	6.5	<20	<5	Not Present

6700 Alexander Bell Dr. Suite 200
Columbia, MD 21046
(443) 510-8955

Figure No. 20

Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

APPENDIX C

Photographic Documentation

Test Pit Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Test Pit ID: PLT-1
Test Pit Depth: 8 feet
Date: December 8, 2021
Field Engineers: Charles SaintCyr
Jack Puscas

Test Pit Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Test Pit ID: PLT-2
Test Pit Depth: 3-1/2 feet
Date: December 8, 2021
Field Engineers: Charles SaintCyr
Jack Puscas

Figure No. 22

Test Pit Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Test Pit ID:	PLT-3
Test Pit Depth:	4-1/2 feet
Date:	December 8, 2021
Field Engineers:	Charles SaintCyr Jack Puscas

Test Pit Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Test Pit ID:	TP-4
Test Pit Depth:	6-1/2 feet
Date:	December 8, 2021
Field Engineers:	Charles SaintCyr
	Jack Puscas

Figure No. 24

Test Pit Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Test Pit ID:	TP-5
Test Pit Depth:	7 feet
Date:	December 8, 2021
Field Engineers:	Charles SaintCyr Jack Puscas

Figure No. 25

Test Pit Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Test Pit ID:	TP-6
Test Pit Depth:	3-1/2 feet
Date:	December 8, 2021
Field Engineers:	Charles SaintCyr
	Jack Puscas

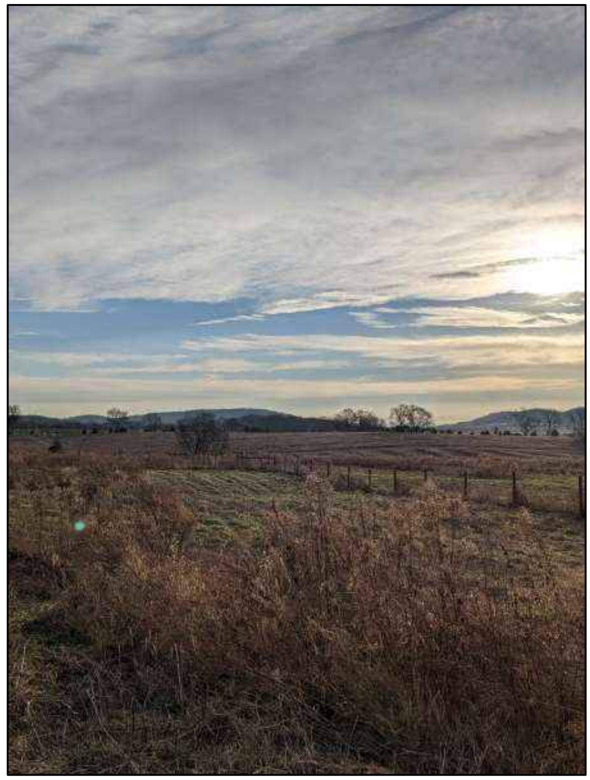
Figure No. 26

Test Pit Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

Test Pit ID:	TP-7
Test Pit Depth:	7 feet
Date:	December 8, 2021
Field Engineers:	Charles SaintCyr Jack Puscas

Figure No. 27

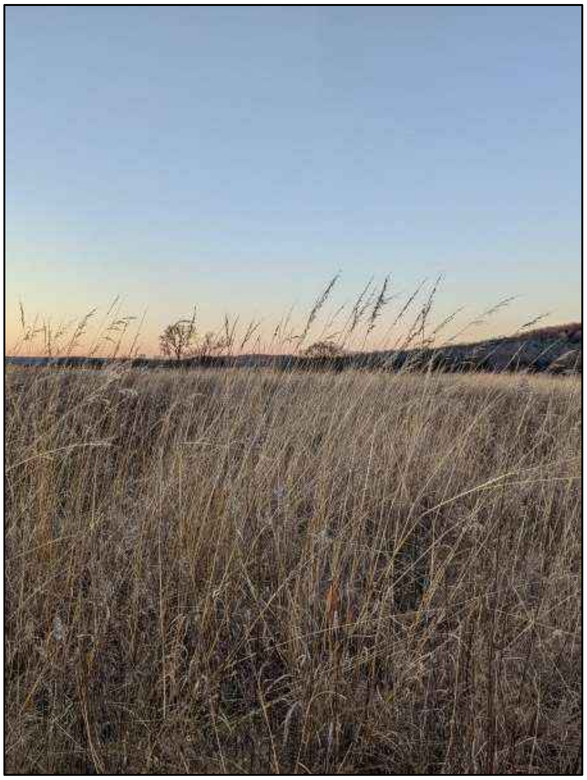

PLT-1 - Looking North


PLT-1 - Looking East

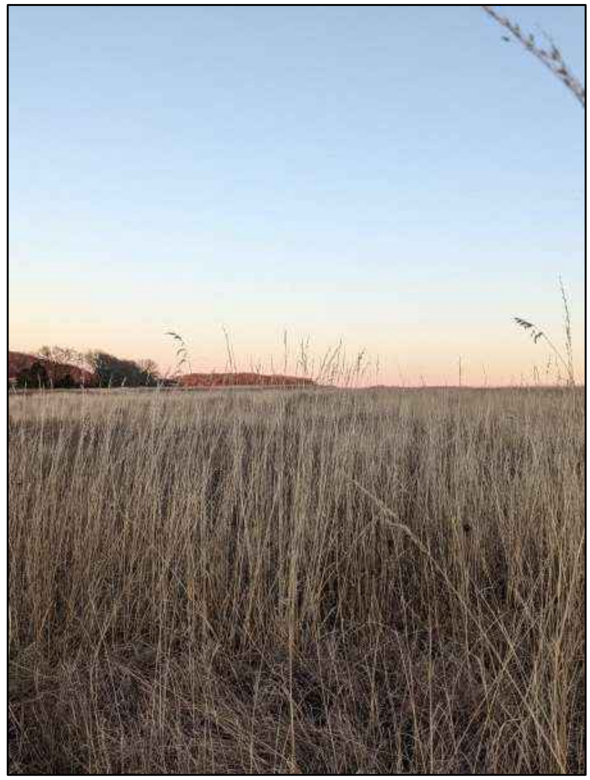

PLT-1 - Looking West


PLT-1 - Looking South

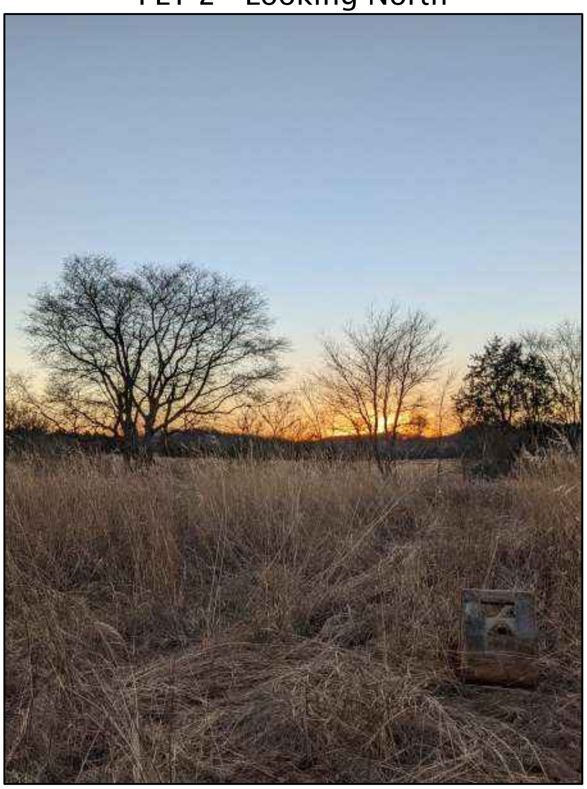
Figure No. 28



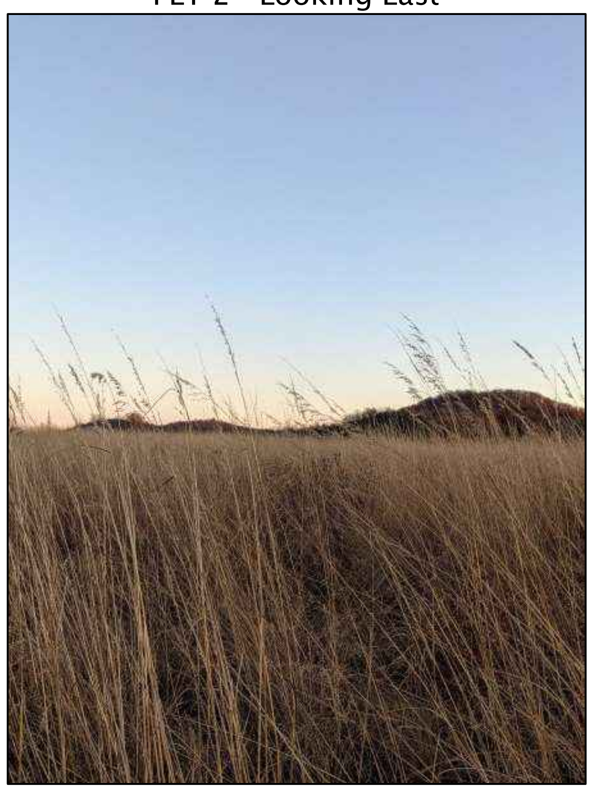
PLT-2 - Looking North



PLT-2 - Looking East



PLT-2 - Looking West



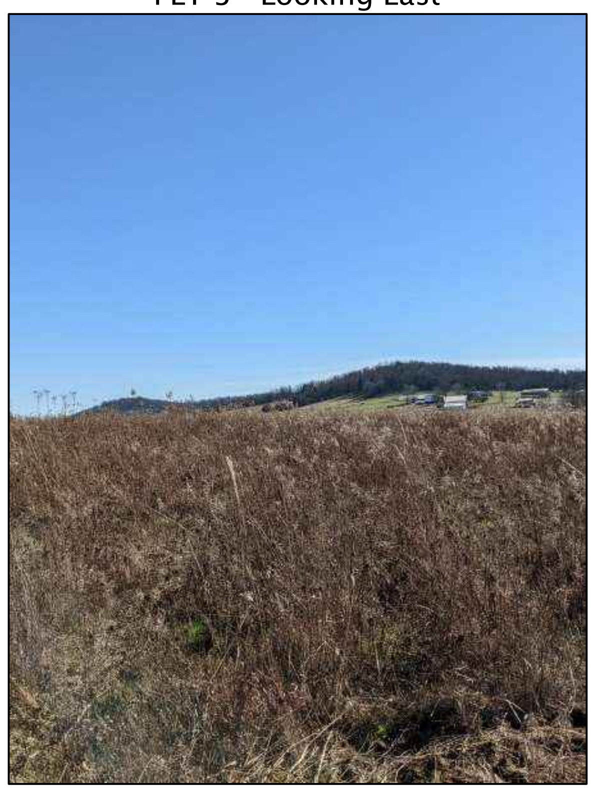
PLT-2 - Looking South

Figure No. 29

PLT-3 - Looking North

PLT-3 - Looking East

PLT-3 - Looking West



PLT-3 - Looking South

Figure No. 30


TP-4 - Looking North

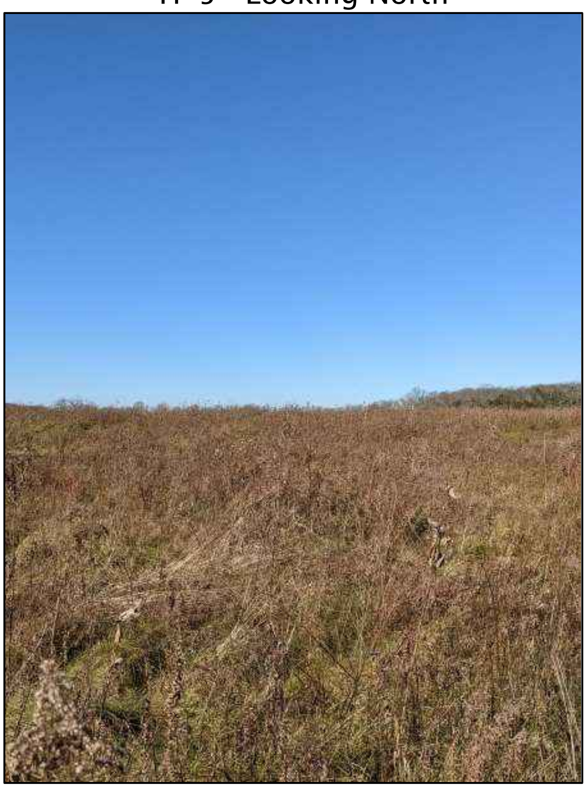

TP-4 - Looking East


TP-4 - Looking West


TP-4 - Looking South

TP-5 - Looking North

TP-5 - Looking East



TP-5 - Looking West

TP-5 - Looking South

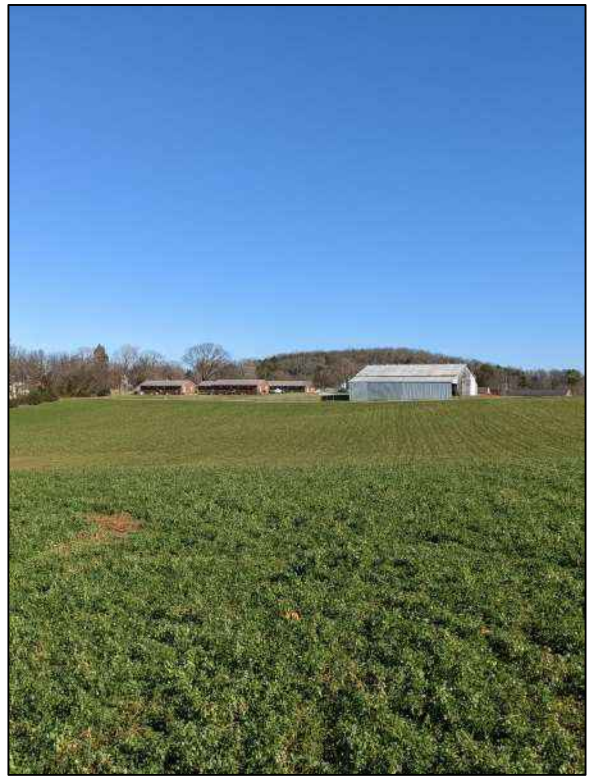
TP-6 - Looking North

TP-6 - Looking East

TP-6 - Looking West

TP-6 - Looking South

TP-7 - Looking North



TP-7 - Looking East

TP-7 - Looking West

TP-7 - Looking South

PLT-1A (Top)

PLT-1A (Bottom)

PLT-1B (Top)

PLT-1B (Bottom)

Figure No. 35

Pile Head Photographic Documentation
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

PLT-2A (Top)

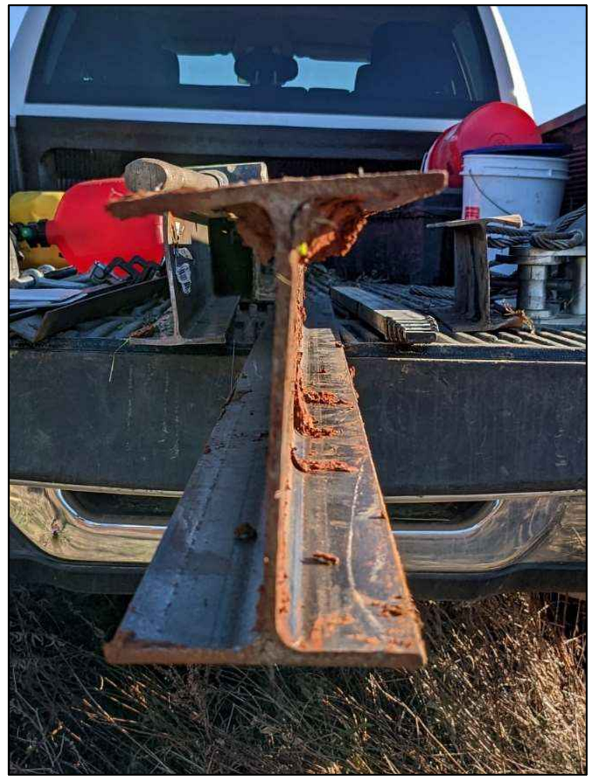
PLT-2A (Bottom)

PLT-2B (Top)

PLT-2B (Bottom)

Figure No. 36

PLT-3A (Top)



PLT-3A (Bottom)

PLT-3B (Top)

PLT-3B (Bottom)

Figure No. 37

APPENDIX D

Pile Load Test Data

Bobcat E85 excavator fitted with a Furukawa KF6 hydraulic impact hammer

The following table presents the final test pile locations and embedment depths. The GPS coordinates presented are based on handheld GPS (Garmin®) in conjunction with Google Earth® software. Photographic documentation of the top and bottom of each pile is presented in Appendix C, Figure Nos. 35 through 37.

Test No.	Embedment Depths	GPS Location (decimal degree)	Average Push Depth with vibratory setting off	Average Driving Rate after Push Depth w/ Hammer on High Vibratory Setting (inches of penetration per second)
PLT-1	5 feet (short) 8 feet (long)	Latitude: 37.24278˚ Longitude: -85.92142˚	4	2 to 4 in/sec to 7 feet 0.5 to 1 in/sec to 8 feet refusal at 8 feet
PLT-2	4-1/2 feet (short) 5 feet (long)	Latitude: 37.23715˚ Longitude: -85.26460˚	3-1/2	3 to 6 in/sec to 5 feet refusal at 5 feet
PLT-3	5 feet (short) 7 feet (long)	Latitude: 37.23608˚ Longitude: -85.90336˚	3	2 to 4 in/sec to 7 feet refusal at 7 feet

CONSULTING GROUP

As-Tested Tension and Lateral Load of Driven Piles
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

		As-Tested Tension Load (lbs)			
Pile No.	Pile Embedment Depth (feet)	Load @ 0.25-inch Deflection	Load @ 0.50-inch Deflection	Load @ 1.00-inch Deflection	Maximum Recorded Load During Initial Load Sequence
PLT-1A	5	3,620	4,180	5,180	5,200 @ 1.01 in.
PLT-1B	8	3,750	3,830	3,900	3,900 @ 1.02 in.
PLT-2A	4-1/2	570	680	750	750 @ 1.04 in.
PLT-2B	5	1,150	1,350	1,670	1,700 @ 1.05 in.
PLT-3A	5	4,520	4,820	5,080	5,100 @ 1.05 in
PLT-3B	7	6,960	7,030	7,090	7,100 @ 1.06 in.

Note: Tension Load Acceptance Criteria is assumed to be 0.25-inch deflection

		As-Tested Lateral Load (lbs)			
Pile No.	Pile Embedment Depth (feet)	Load @ 0.25-inch Deflection	Load @ 0.50-inch Deflection	Load @ 1.00-inch Deflection	Maximum Recorded Load During Initial Load Sequence
PLT-1A	5	2,440	4,050	5,920	6,000 @ 1.02 in.
PLT-1B	8	3,330	5,920	8,060	8,100 @ 1.01 in.
PLT-2A	4-1/2	1,440	2,000	3,310	3,400 @ 1.03 in.
PLT-2B	5	1,180	2,000	3,410	4,000 @ 1.10 in.
PLT-3A	5	2,420	4,030	5,420	5,500 @ 1.04 in.
PLT-3B	7	2,390	4,820	7,140	7,300 @ 1.04 in.

Note: Lateral Load Acceptance Criteria is assumed to be 0.50-inch deflection

CONSULTING GROUP

Pile Load Test Results

Test Location: PLT-1A
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 5 feet
GPS Coordinates: 37.24278˚, -85.92142˚

Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021

Tension Load Test						Lateral Load Test					
Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)	Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)
1	0	--	0.000	0.000	0.000	1	0	--	0.000	0.000	0.000
2	500	1	0.003	0.002	0.003	2	500	1	0.036	0.048	0.042
3	1,000	1	0.004	0.004	0.004	3	1,000	1	0.086	0.094	0.090
4	1,500	1	0.007	0.009	0.008	4	0	1	0.023	0.049	0.036
5	2,000	1	0.012	0.013	0.013	5	1,000	1	0.096	0.105	0.101
6	2,000	5	0.015	0.018	0.017	6	1,500	1	0.143	0.141	0.142
7	0	1	0.015	0.017	0.016	7	0	1	0.050	0.072	0.061
8	2,000	1	0.019	0.021	0.020	8	1,500	1	0.156	0.160	0.158
9	3,700	1	0.261	0.263	0.262	9	2,000	1	0.193	0.197	0.195
10	0	1	0.246	0.251	0.249	10	0	1	0.062	0.089	0.076
11	5,200	1	1.015	1.008	1.012	11	2,000	1	0.202	0.203	0.203
12	0	1	0.987	0.986	0.987	12	2,500	1	0.255	0.256	0.256
Reset Gauges to Zero						13	0	1	0.087	0.111	0.099
13	0	--	0	0	0	14	2,500	1	0.262	0.275	0.269
14	6300	1	0.521	0.511	0.516	15	3,000	1	0.311	0.331	0.321
15	0	1	0.485	0.476	0.481	16	0	1	0.098	0.125	0.112
						17	3,000	1	0.335	0.357	0.346
						18	4,100	1	0.490	0.525	0.508
						19	0	1	0.202	0.238	0.220
						20	6,000	1	0.992	1.050	1.021
						21	0	1	0.480	0.522	0.501

Figure No. 40A

Tension Pile Load Test Results

Test Location:	PLT-1A	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	5 feet	Test Date:	12/8/2021
GPS Coordinates:	37.24278˚, -85.92142˚		

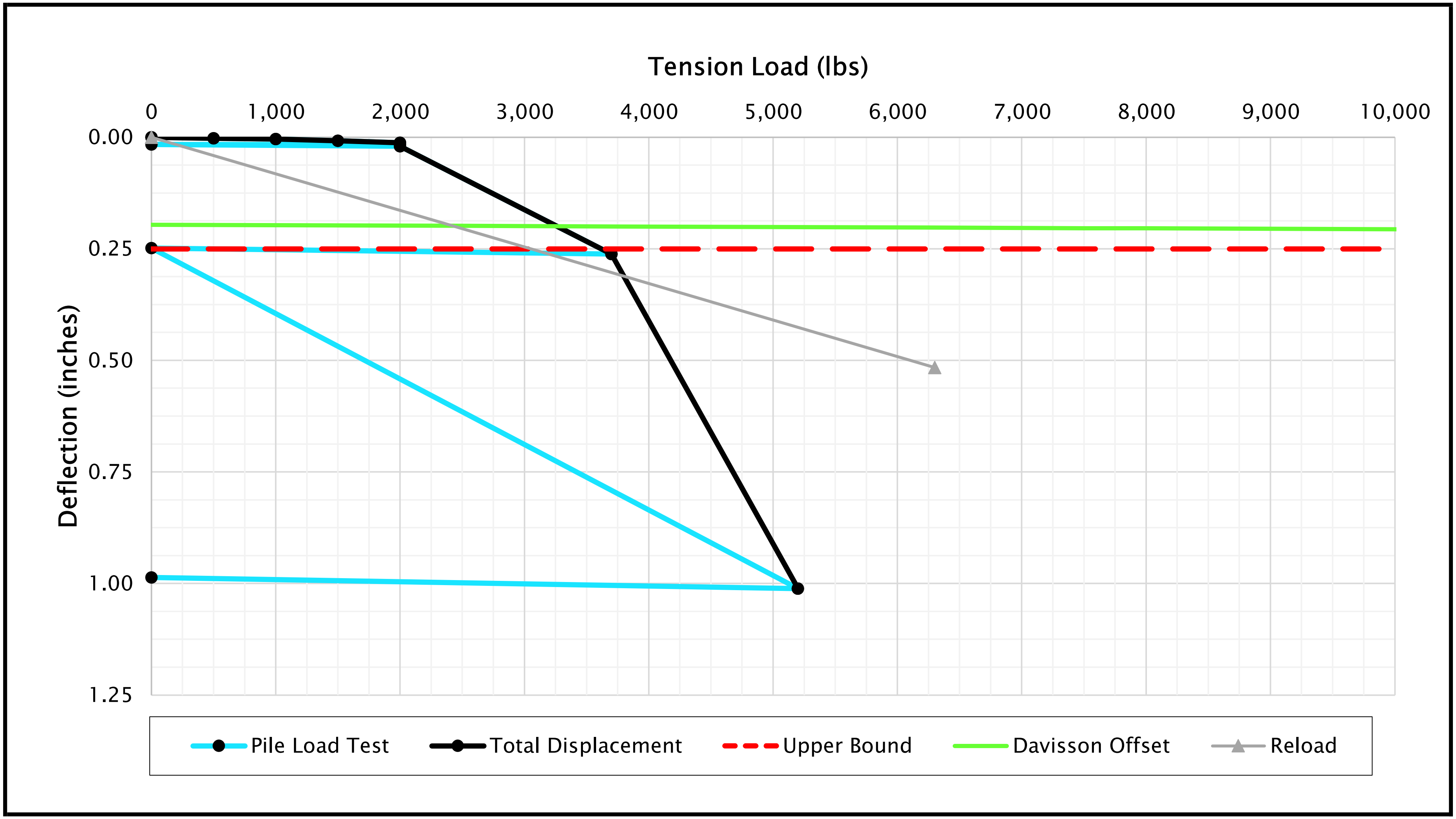


Figure No. 40B

CONSULTING GROUP

Lateral Pile Load Test Results

Test Location:	PLT-1A	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	5 feet	Test Date:	12/8/2021
GPS Coordinates:	37.24278˚, -85.92142˚		

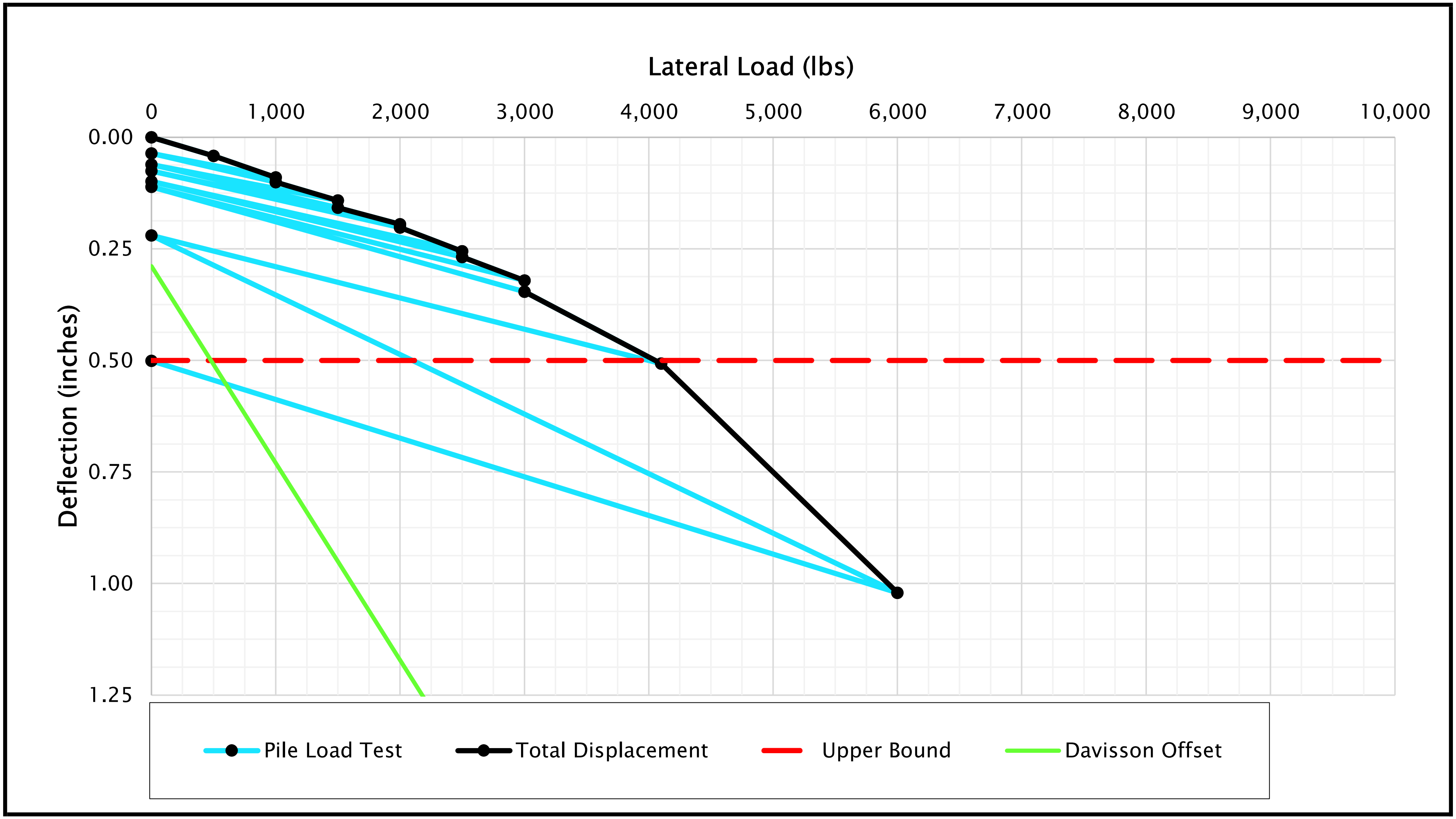


Figure No. 40C

CONSULTING GROUP

Pile Load Test Results

Test Location: PLT-1B
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 8 feet
GPS Coordinates: 37.24278˚, -85.92142˚

Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021

Tension Load Test						Lateral Load Test					
Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)	Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)
1	0	--	0.000	0.000	0.000	1	0	--	0.000	0.000	0.000
2	500	1	0.001	0.002	0.002	2	500	1	0.035	0.013	0.024
3	1,000	1	0.003	0.002	0.003	3	1,000	1	0.072	0.053	0.063
4	1,500	1	0.005	0.002	0.004	4	0	1	0.028	0.023	0.026
5	2,000	1	0.009	0.002	0.006	5	1,000	1	0.084	0.055	0.070
6	2,000	5	0.010	0.004	0.007	6	1,500	1	0.117	0.090	0.104
7	0	1	0.004	0.004	0.004	7	0	1	0.043	0.036	0.040
8	2,000	1	0.010	0.004	0.007	8	1,500	1	0.130	0.100	0.115
9	3,800	1	0.253	0.261	0.257	9	2,000	1	0.155	0.132	0.144
10	0	1	0.231	0.252	0.242	10	0	1	0.046	0.043	0.045
11	3,900	1	1.029	1.004	1.017	11	2,000	1	0.158	0.139	0.149
12	0	1	1.001	0.995	0.998	12	2,500	1	0.181	0.167	0.174
Reset Gauges to Zero						13	0	1	0.056	0.058	0.057
13	0	--	0	0	0	14	2,500	1	0.183	0.171	0.177
14	4000	1	0.504	0.505	0.505	15	3,000	1	0.220	0.211	0.216
15	0	1	0.469	0.458	0.464	16	0	1	0.071	0.081	0.076
						17	3,000	1	0.221	0.215	0.218
						18	6,200	1	0.480	0.574	0.527
						19	0	1	0.166	0.267	0.217
						20	8,100	1	0.990	1.032	1.011
						21	0	1	0.451	0.567	0.509

Figure No. 41A

Tension Pile Load Test Results

Test Location:	PLT-1B	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	8 feet	Test Date:	12/8/2021
GPS Coordinates:	37.24278˚, -85.92142˚		

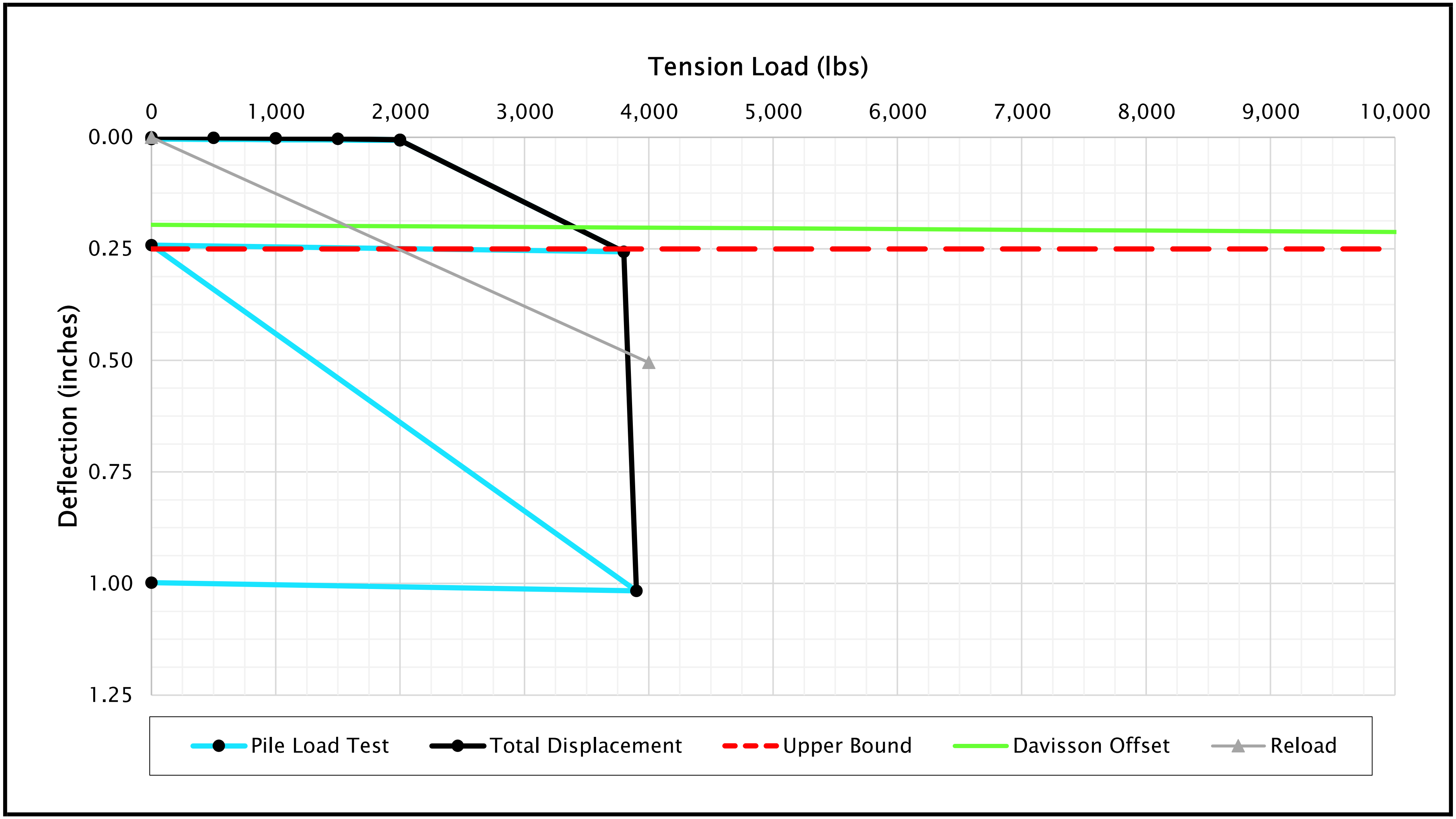


Figure No. 41B

Lateral Pile Load Test Results

Test Location:	PLT-1B	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	8 feet	Test Date:	12/8/2021
GPS Coordinates:	37.24278°, -85.92142°		

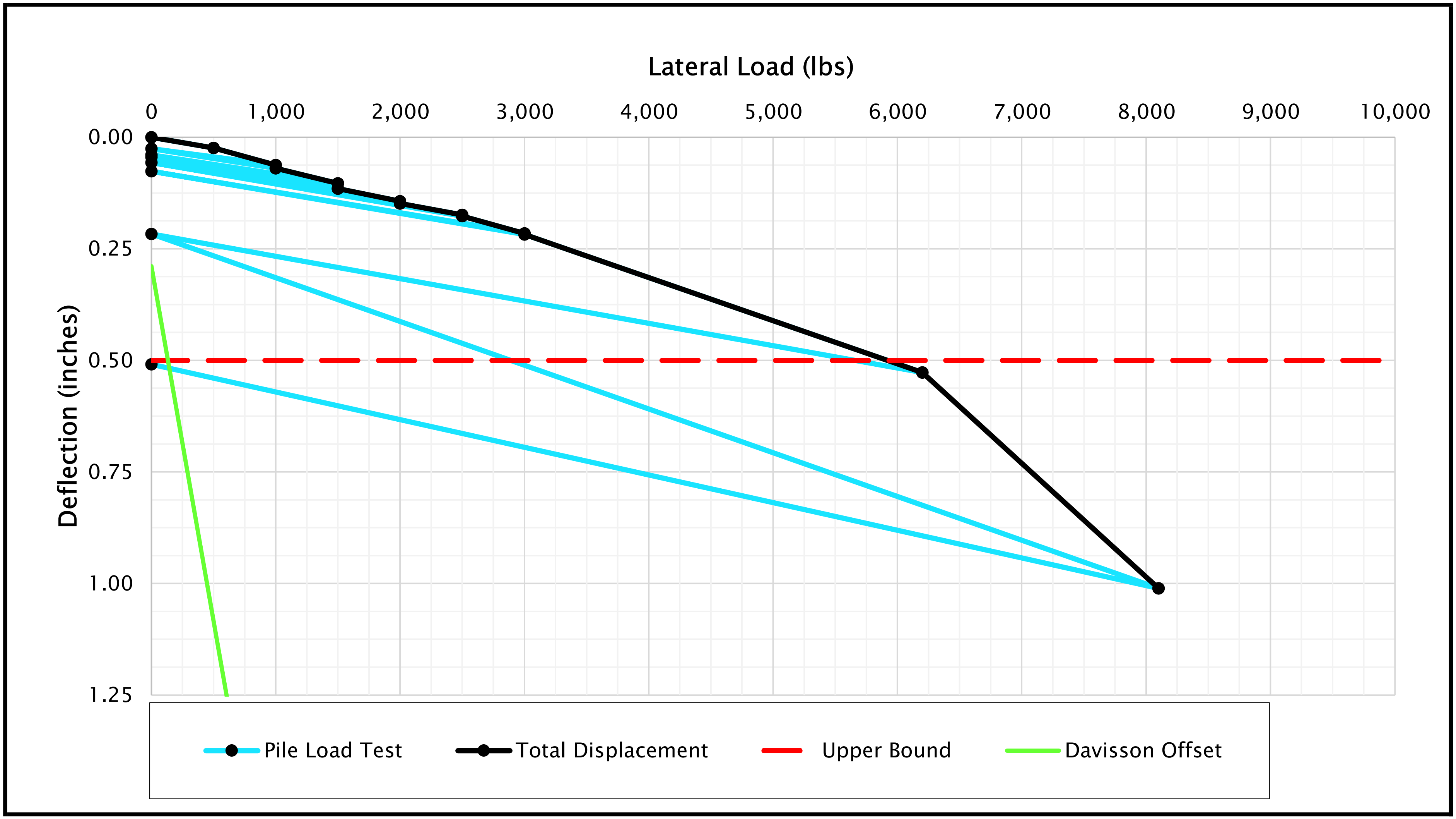


Figure No. 41C

Pile Load Test Results

Test Location: PLT-2A
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 4.5 feet
GPS Coordinates: 37.23715˚, -85.26460˚

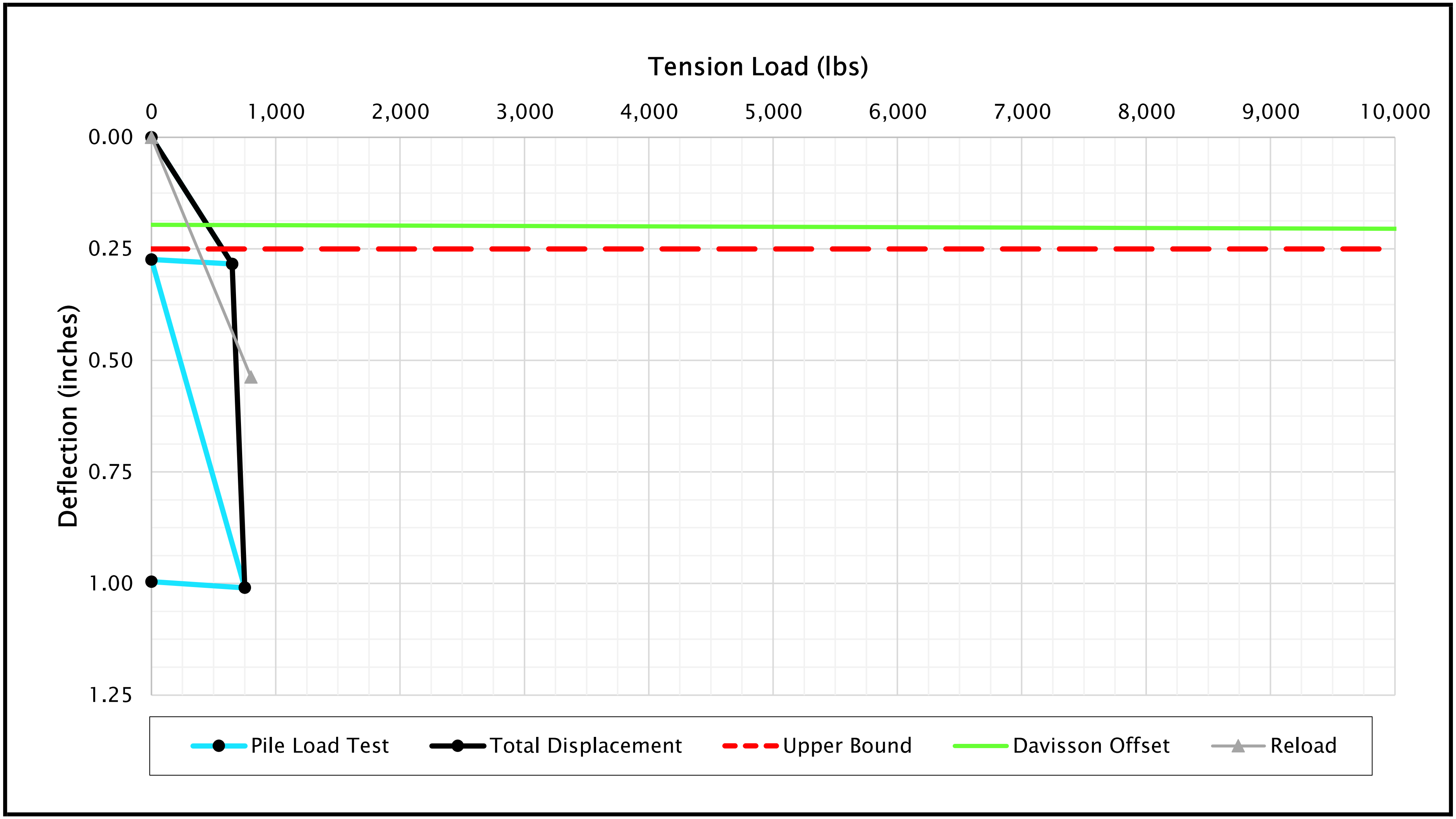
Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021

Tension Load Test						Lateral Load Test					
Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)	Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)
1	0	--	0.000	0.000	0.000	1	0	--	0.000	0.000	0.000
2	650	1	0.267	0.301	0.284	2	500	1	0.047	0.091	0.069
3	0	1	0.260	0.288	0.274	3	1,000	1	0.099	0.166	0.133
4	750	1	0.940	1.079	1.010	4	0	1	0.046	0.063	0.055
5	0	1	0.931	1.061	0.996	5	1,000	1	0.118	0.192	0.155
Reset Gauges to Zero						6	1,500	1	0.208	0.316	0.262
6	0	1	0.000	0.000	0.000	7	0	1	0.088	0.096	0.092
7	800	1	0.493	0.582	0.538	8	1,500	1	0.198	0.373	0.286
8	0	1	0.461	0.564	0.513	9	2,000	1	0.277	0.490	0.384
						10	0	1	0.158	0.232	0.195
						11	2,000	1	0.455	0.542	0.499
						12	2,100	1	0.511	0.593	0.552
						13	0	1	0.300	0.360	0.330
						14	3,400	1	0.972	1.092	1.032
						15	0	1	0.633	0.691	0.662

Figure No. 42A

Tension Pile Load Test Results

Test Location:	PLT-2A	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	4.5 feet	Test Date:	12/8/2021
GPS Coordinates:	37.23715˚, -85.26460˚		



Lateral Pile Load Test Results

Test Location:	PLT-2A	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	4.5 feet	Test Date:	12/8/2021
GPS Coordinates:	37.23715˚, -85.26460˚		

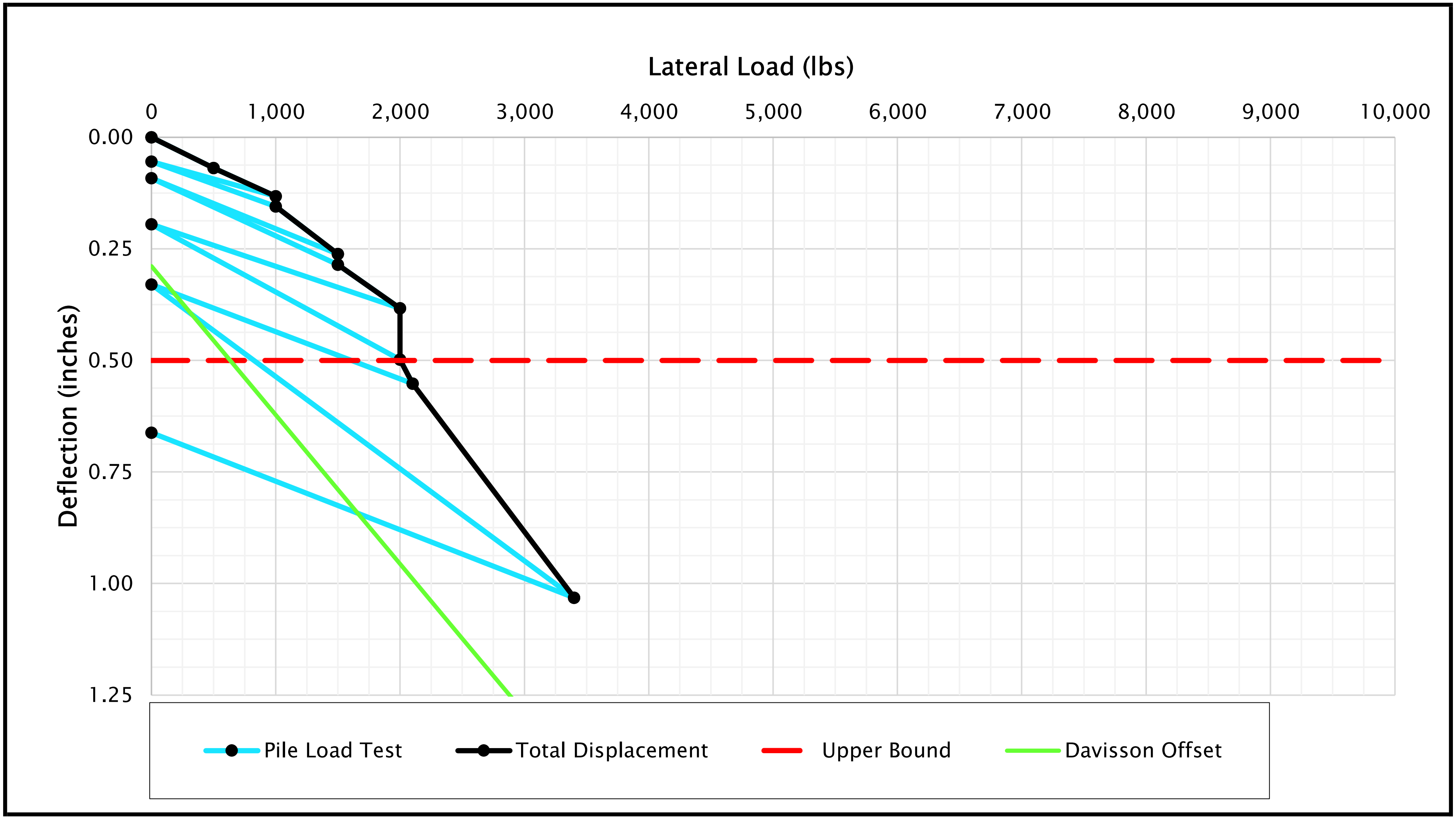


Figure No. 42C

Pile Load Test Results

Test Location: PLT-2B
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 5 feet
GPS Coordinates: 37.23715˚, -85.26460˚

Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021

Tension Load Test						Lateral Load Test					
Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)	Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)
1	0	--	0.000	0.000	0.000	1	0	--	0.000	0.000	0.000
2	500	1	0.012	0.007	0.010	2	500	1	0.113	0.044	0.079
3	1,000	1	0.218	0.183	0.201	3	1,000	1	0.266	0.118	0.192
4	1,200	1	0.288	0.244	0.266	4	0	1	0.105	0.054	0.080
5	0	1	0.276	0.235	0.256	5	1,000	1	0.281	0.150	0.216
6	1,700	1	1.089	1.015	1.052	6	1,500	1	0.392	0.230	0.311
7	0	1	1.066	0.994	1.030	7	0	1	0.214	0.107	0.161
Reset Gauges to Zero						8	1,500	1	0.439	0.262	0.351
8	0	1	0.000	0.000	0.000	9	2,000	1	0.530	0.334	0.432
9	2,300	1	0.566	0.464	0.515	10	0	1	0.304	0.165	0.235
10	0	1	0.523	0.419	0.471	11	2,000	1	0.547	0.468	0.508
						12	0	1	0.354	0.244	0.299
						13	3,700	1	1.013	1.187	1.100
						14	0	1	0.741	0.801	0.771

Figure No. 43A

Tension Pile Load Test Results

Test Location:	PLT-2B	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	5 feet	Test Date:	12/8/2021
GPS Coordinates:	37.23715˚, -85.26460˚		

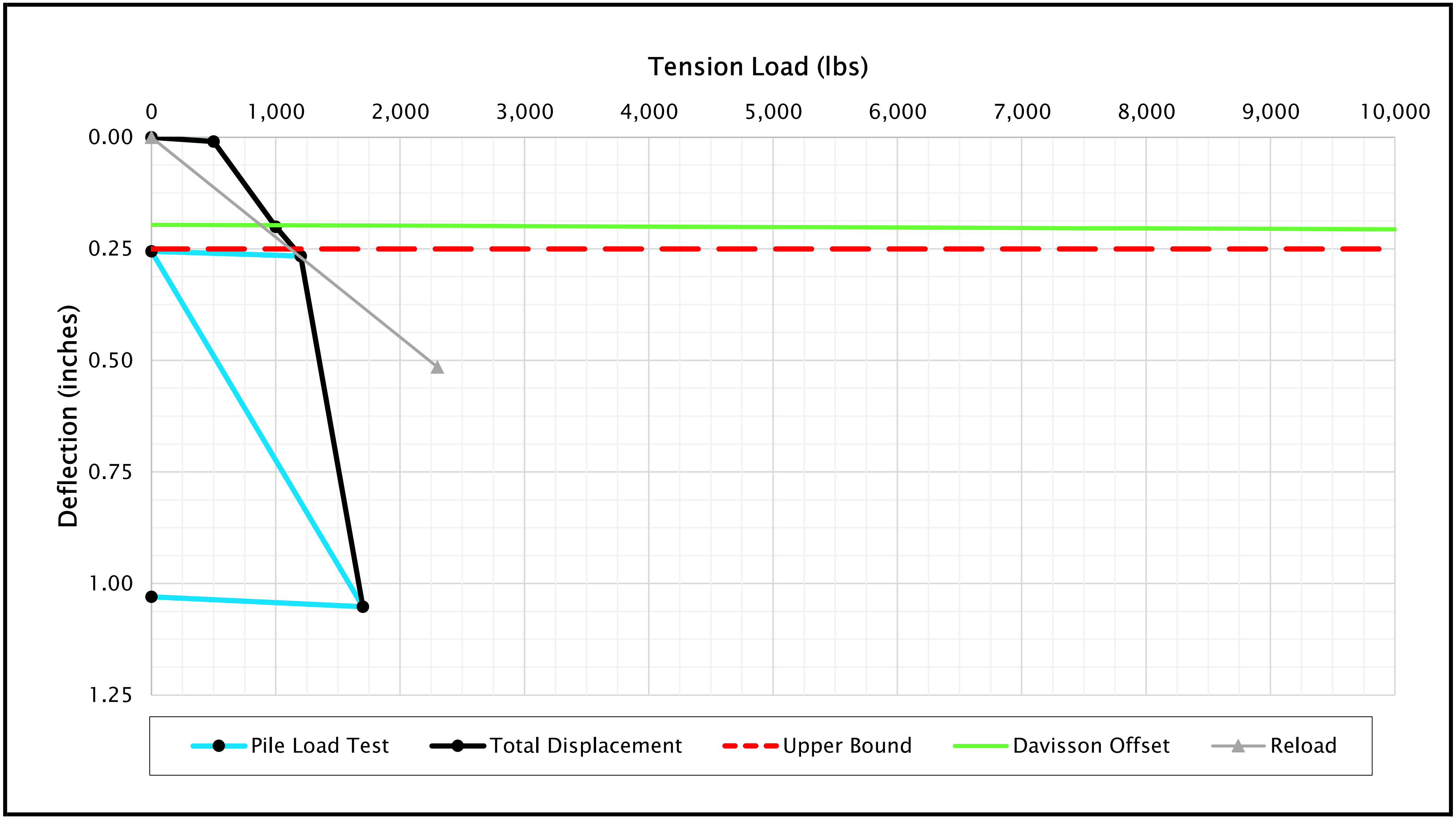


Figure No. 43B

Lateral Pile Load Test Results

Test Location:	PLT-2B	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	5 feet	Test Date:	12/8/2021
GPS Coordinates:	37.23715˚, -85.26460˚		

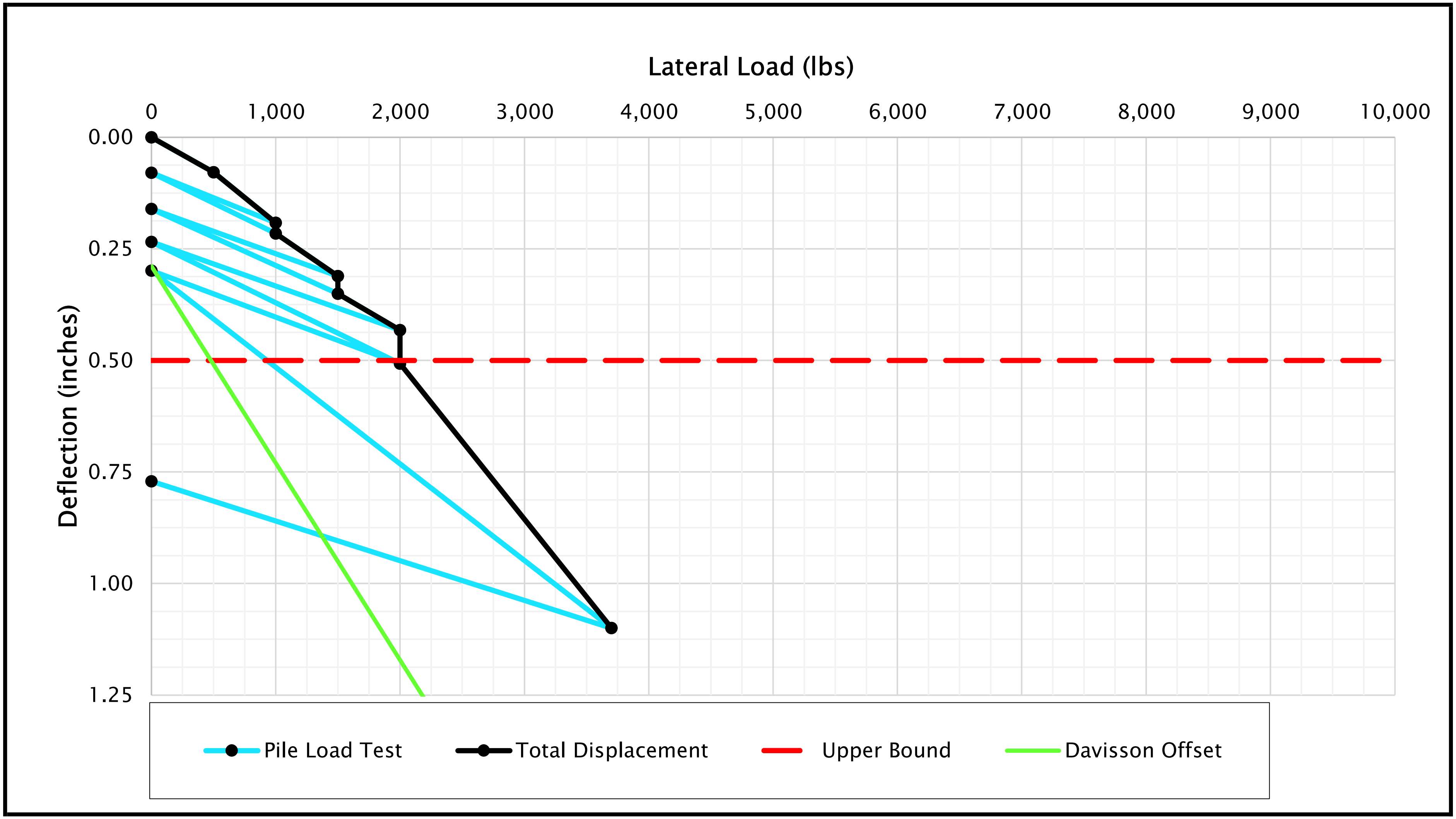


Figure No. 43C

CONSULTING GROUP

Pile Load Test Results

Test Location: PLT-3A
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 5 feet
GPS Coordinates: 37.23608˚, -85.90336˚

Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021

Tension Load Test						Lateral Load Test					
Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)	Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)
1	0	--	0.000	0.000	0.000	1	0	--	0.000	0.000	0.000
2	500	1	0.002	0.000	0.001	2	500	1	0.025	0.031	0.028
3	1,000	1	0.003	0.002	0.003	3	1,000	1	0.059	0.085	0.072
4	1,500	1	0.005	0.005	0.005	4	0	1	0.033	0.036	0.035
5	2,000	1	0.010	0.011	0.011	5	1,000	1	0.069	0.088	0.079
6	2,000	5	0.013	0.016	0.015	6	1,500	1	0.113	0.148	0.131
7	0	1	0.012	0.012	0.012	7	0	1	0.042	0.039	0.041
8	2,000	1	0.016	0.016	0.016	8	1,500	1	0.151	0.189	0.170
9	4,700	1	0.265	0.269	0.267	9	2,000	1	0.178	0.222	0.200
10	0	1	0.256	0.247	0.252	10	0	1	0.063	0.052	0.058
11	5,100	1	1.049	1.047	1.048	11	2,000	1	0.189	0.217	0.203
12	0	1	1.031	1.018	1.025	12	2,500	1	0.242	0.275	0.259
Reset Gauges to Zero						13	0	1	0.081	0.076	0.079
13	0	--	0	0	0	14	2,500	1	0.263	0.285	0.274
14	5,100	1	0.563	0.564	0.564	15	3,000	1	0.317	0.342	0.330
15	0	1	0.557	0.542	0.550	16	0	1	0.092	0.087	0.090
						17	3,000	1	0.356	0.377	0.367
						18	4,300	1	0.542	0.528	0.535
						19	0	1	0.126	0.091	0.109
						20	5,500	1	1.063	1.007	1.035
						21	0	1	0.632	0.438	0.535

Figure No. 44A

CONSULTING GROUP

Tension Pile Load Test Results

Test Location:	PLT-3A	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	5 feet	Test Date:	12/8/2021
GPS Coordinates:	37.23608˚, -85.90336˚		

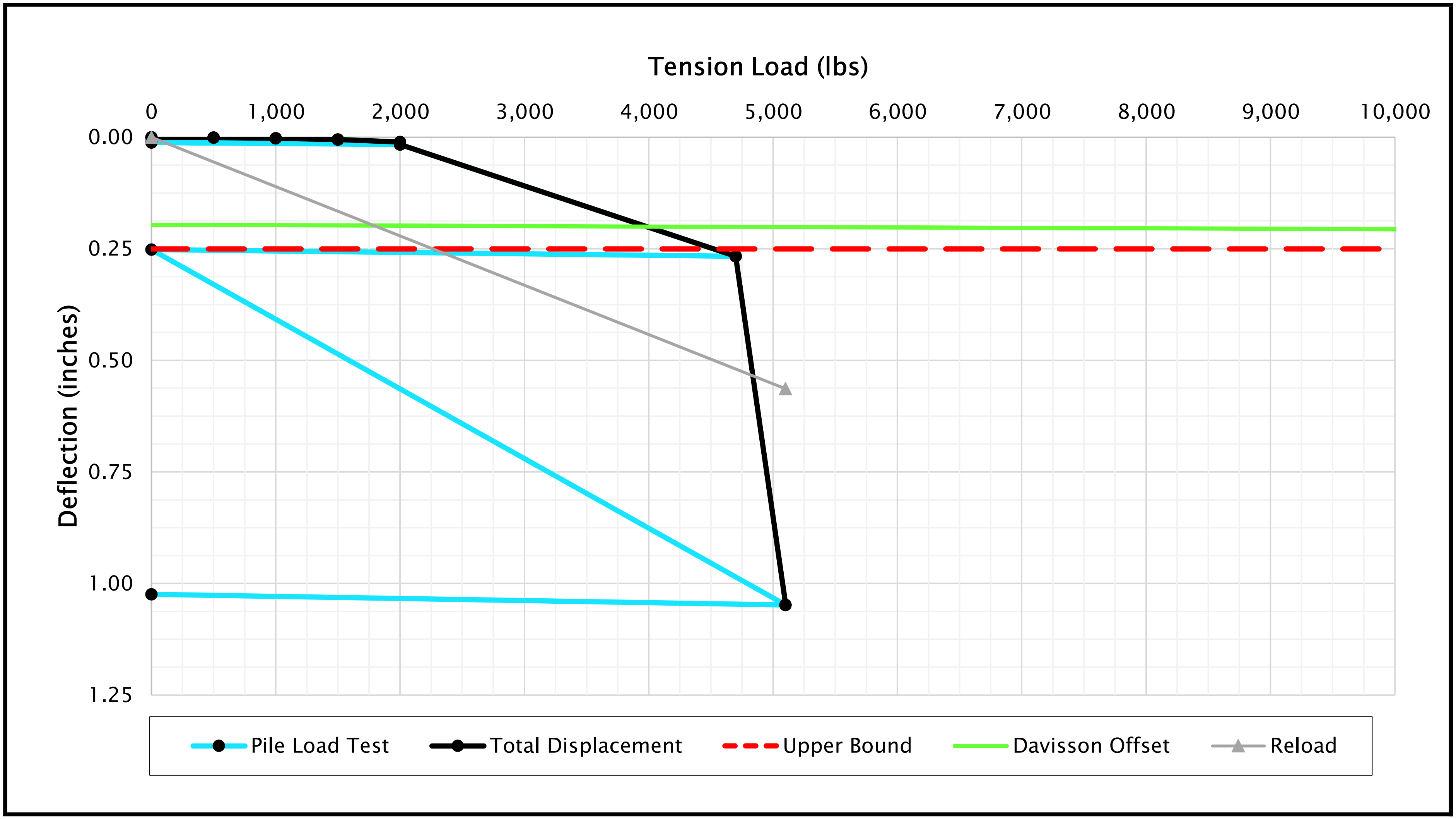


Figure No. 44B

Lateral Pile Load Test Results

Test Location: PLT-3A
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 5 feet
GPS Coordinates: 37.23608˚, -85.90336˚

Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021

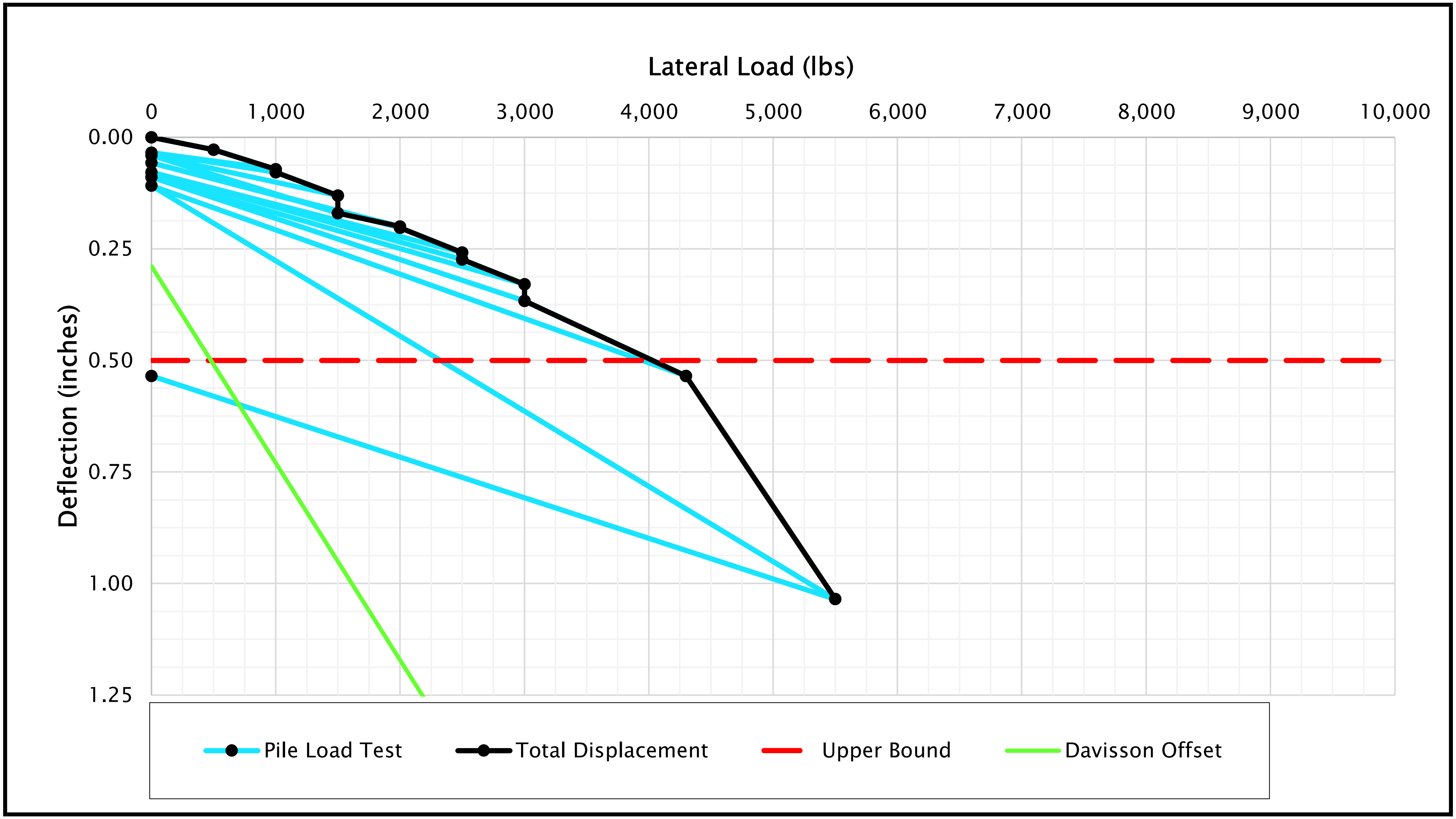


Figure No. 44C

CONSULTING GROUP

Pile Load Test Results

Test Location: PLT-3B
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 7 feet
GPS Coordinates: 37.23608˚, -85.90336˚

Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021

Tension Load Test						Lateral Load Test					
Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)	Step	Applied Load ± 50 (lbs)	Hold Time (min)	Deflection Gauge #1 (inches)	Deflection Gauge #2 (inches)	Average Deflection (inches)
1	0	--	0.000	0.000	0.000	1	0	--	0.000	0.000	0.000
2	500	1	0.002	0.002	0.002	2	500	1	0.067	0.033	0.050
3	1,000	1	0.004	0.003	0.004	3	1,000	1	0.107	0.084	0.096
4	1,500	1	0.005	0.005	0.005	4	0	1	0.005	0.018	0.012
5	2,000	1	0.005	0.005	0.005	5	1,000	1	0.110	0.094	0.102
6	2,000	5	0.005	0.005	0.005	6	1,500	1	0.152	0.160	0.156
7	0	1	0.004	0.003	0.004	7	0	1	0.012	0.035	0.024
8	2,000	1	0.007	0.007	0.007	8	1,500	1	0.152	0.169	0.161
9	7,000	1	0.240	0.263	0.252	9	2,000	1	0.185	0.225	0.205
10	0	1	0.233	0.231	0.232	10	0	1	0.010	0.051	0.031
11	7,100	1	1.025	1.087	1.056	11	2,000	1	0.186	0.232	0.209
12	0	1	1.016	1.052	1.034	12	2,500	1	0.226	0.298	0.262
Reset Gauges to Zero						13	0	1	0.011	0.068	0.040
13	0	--	0	0	0	14	2,500	1	0.277	0.252	0.265
14	7,200	1	0.505	0.579	0.542	15	3,000	1	0.324	0.307	0.316
15	0	1	0.503	0.539	0.521	16	0	1	0.016	0.070	0.043
						17	3,000	1	0.325	0.321	0.323
						18	4,900	1	0.484	0.532	0.508
						19	0	1	0.039	0.146	0.093
						20	7,300	1	0.989	1.083	1.036
						21	0	1	0.185	0.411	0.298

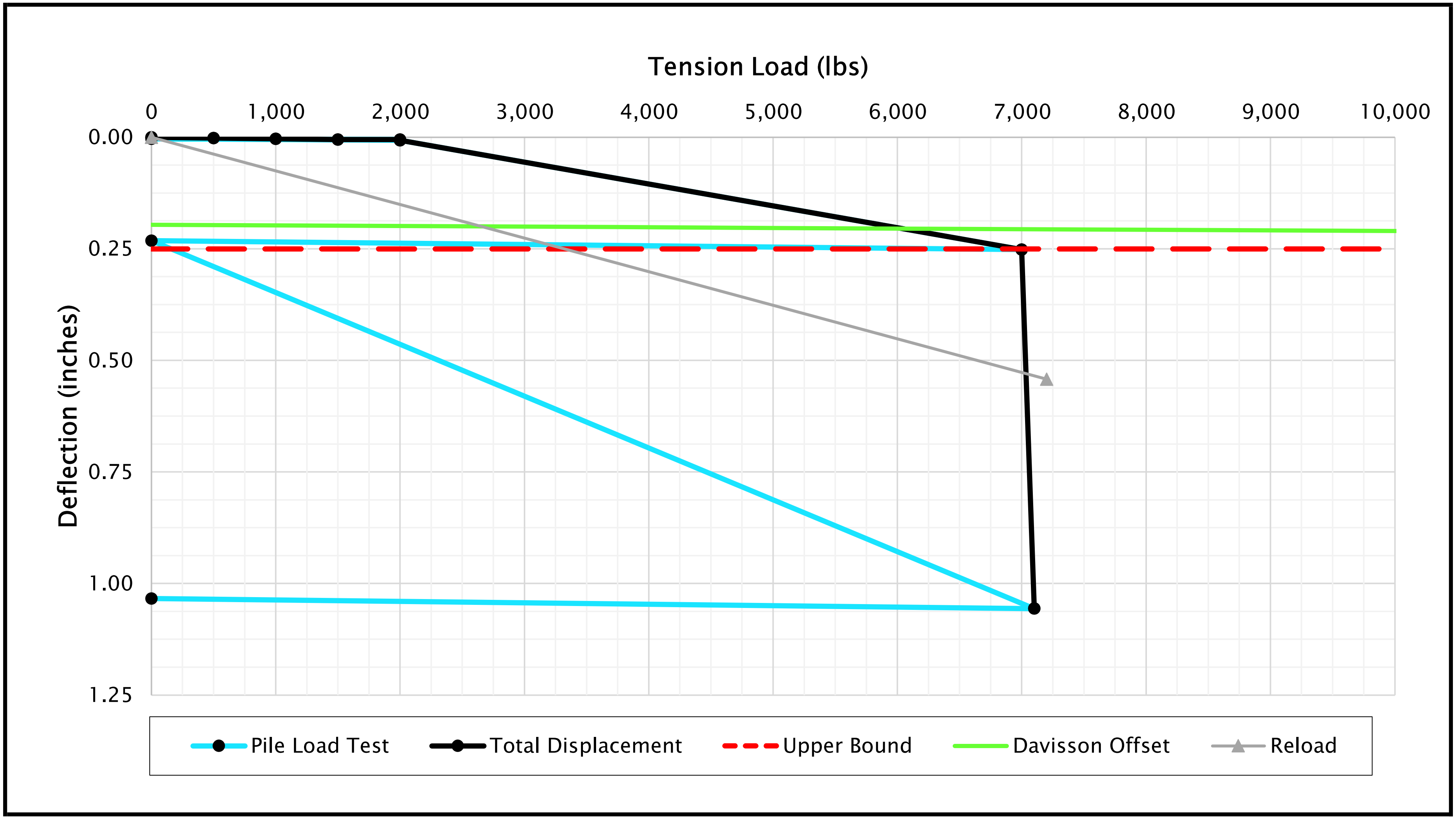
Figure No. 45A

CONSULTING GROUP

Tension Pile Load Test Results

Test Location: PLT-3B
Pile Size: W6x9 non-galvanized steel
Embedment Depth: 7 feet
GPS Coordinates: 37.23608˚, -85.90336˚

Project Name: Hart Solar Power Plant
Project Number: 213841
Test Date: 12/8/2021



Lateral Pile Load Test Results

Test Location:	PLT-3B	Project Name:	Hart Solar Power Plant
Pile Size:	W6x9 non-galvanized steel	Project Number:	213841
Embedment Depth:	7 feet	Test Date:	12/8/2021
GPS Coordinates:	37.23608˚, -85.90336˚		

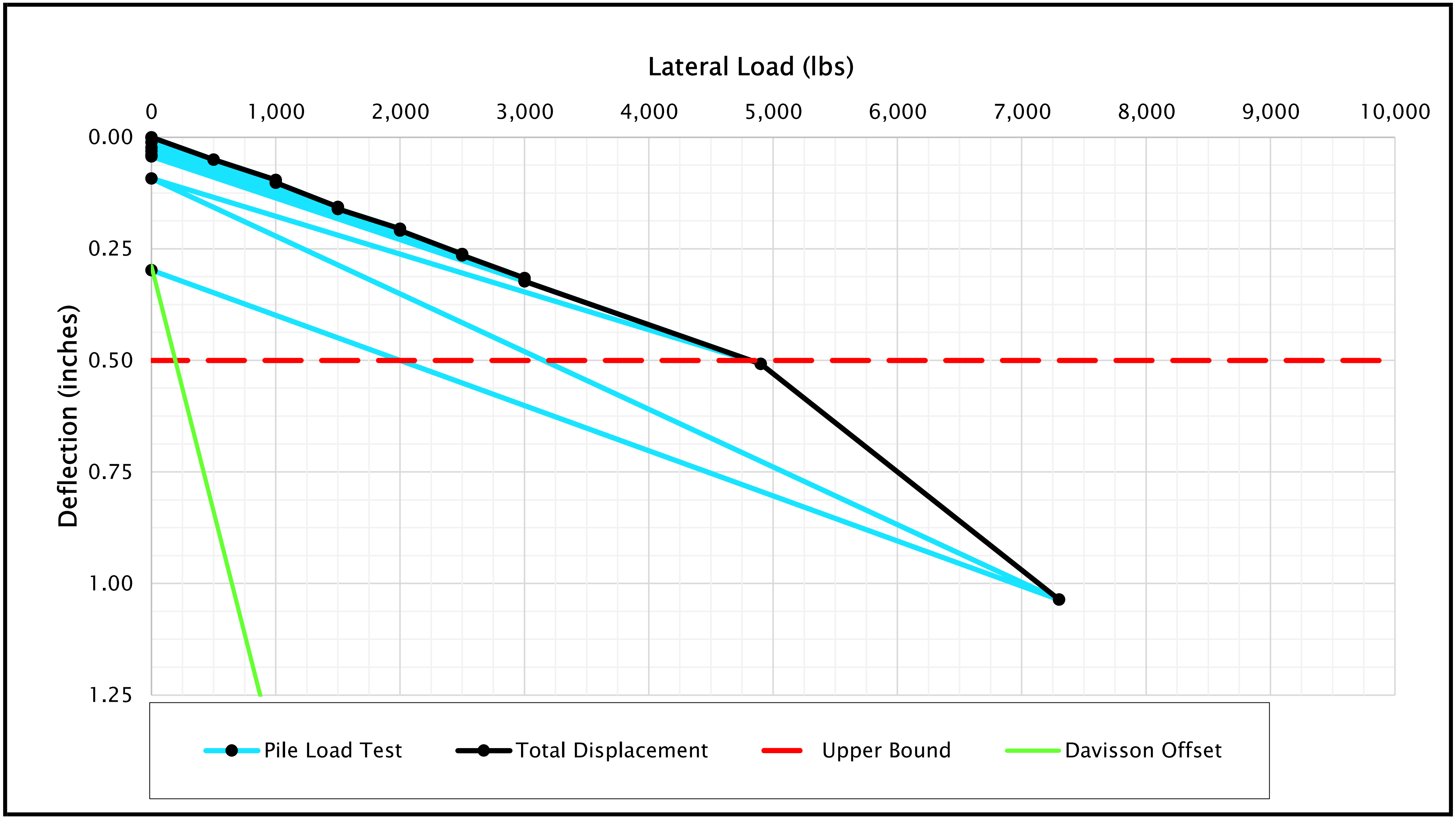


Figure No. 45C

CONSULTING GROUP

Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

APPENDIX E

LPILE (version 2019.11.03) Analyses

LPile v2019 Input Parameters						
Pile No.	Depth (feet)		LPILE Soil Type	Cohesion (psf)	ε_{50}	Unit Weight (pcf)
	From	To				
PLT-1A	0 1	1 5	Stiff Clay w/o Free Water Stiff Clay w/o Free Water	600 3,400	0.0127 0.0055	110 125
PLT-1B	0 1	1 8	Stiff Clay w/o Free Water Stiff Clay w/o Free Water	800 3,700	0.0110 0.0052	110 125
PLT-2A	0 1 3	1 3 4-1/2	Stiff Clay w/o Free Water Stiff Clay w/o Free Water Stiff Clay w/o Free Water	500 1,650 3,600	0.0200 0.0078 0.0050	110 125 125
PLT-2B	0 1 3	1 3 5	Stiff Clay w/o Free Water Stiff Clay w/o Free Water Stiff Clay w/o Free Water	500 1,100 3,600	0.0200 0.0100 0.0050	110 125 125
PLT-3A	0 1	1 5	Stiff Clay w/o Free Water Stiff Clay w/o Free Water	1,000 3,000	0.0080 0.0060	110 125
PLT-3B	0 1	1 7	Stiff Clay w/o Free Water Stiff Clay w/o Free Water	550 2,800	0.0150 0.0064	110 125

CONSULTING GROUP

Concurrence of LPILE Models
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

LPILE (version 2019.11.03) 1/2-inch Deflection Model of W6x9 Steel Piles						
Pile No.	Pile Embedment Depth (feet)	Actual Load @ 0.25-inch Deflection (lbs)	Modeled Deflection @ Actual Load (in)	Actual Load @ 1-inch or Maximum Deflection (lbs)	Modeled Deflection @ Actual Load (in)	Relative Model Concurrence
PLT-1A	5	2,440	0.14	5,920	1.84	Poor
PLT-1B	8	3,330	0.19	8,060	0.86	Fair
PLT-2A	4-1/2	1,440	0.16	3,310	3.70	Poor
PLT-2B	5	1,180	0.10	3,660	3.76	Poor
PLT-3A	5	2,420	0.14	5,420	1.84	Poor
PLT-3B	7	2,390	0.15	7,140	1.18	Fair

LEGEND

As-tested pile load test data

Modeled LPile Results

Good Relative Model Concurrence

Fair Relative Model Concurrence

Poor Relative Model Concurrence

Preliminary Lateral Capacities for Modeled Deflections
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

LPile v2019 Design Evaluation of the Lateral Capacity of Driven W6x9 Steel Piles											
Capacity Area No.	Load Height Above Grade	Pile Embedment Depth (feet)	Lateral Deflection at Load Height Above Grade (inches)								Figure Nos.
			1,000 lbs	2,000 lbs	3,000 lbs	4,000 lbs	5,000 lbs	6,000 lbs	7,000 lbs	8,000 lbs	
1	6 inches	5	0.04	0.19	0.70	2.02	4.79	---	---	---	49
		6	0.04	0.14	0.31	0.68	1.40	2.68	4.79	12.1	50
		7	0.04	0.14	0.27	0.46	0.75	1.22	1.95	3.03	51
		8	0.04	0.13	0.27	0.44	0.65	0.91	1.26	1.75	52
		9	0.04	0.14	0.27	0.45	0.66	0.91	1.19	1.51	53
		10	0.04	0.14	0.27	0.44	0.65	0.90	1.18	1.50	54
	48 inches	5	0.50	3.10	14.5	---	---	---	---	---	55
		6	0.44	1.28	3.49	8.73	---	---	---	---	56
		7	0.44	1.12	2.07	3.77	6.81	12.8	---	---	57
		8	0.44	1.12	1.96	2.97	4.39	6.46	9.54	15.7	58
		9	0.44	1.15	1.95	2.91	3.99	5.26	6.85	8.92	59
		10	0.44	1.12	1.95	2.92	4.00	5.20	6.49	7.93	60
	72 inches	5	1.29	9.78	---	---	---	---	---	---	61
		6	0.97	3.06	9.15	---	---	---	---	---	62
		7	0.98	2.38	4.68	9.11	19.2	---	---	---	63
		8	0.98	2.36	4.01	6.19	9.48	14.7	31.6	---	64
		9	0.98	2.36	4.00	5.85	7.99	10.7	14.3	20.2	65
		10	0.98	2.37	4.01	5.87	7.90	10.1	12.6	15.6	66
2	6 inches	5	0.10	1.07	5.13	---	---	---	---	---	67
		6	0.08	0.29	0.93	2.56	5.92	---	---	---	68
		7	0.08	0.24	0.48	0.93	1.76	3.19	5.48	10.5	69
		8	0.08	0.24	0.46	0.73	1.08	1.61	2.37	3.47	70
		9	0.08	0.24	0.46	0.73	1.05	1.41	1.83	2.36	71
		10	0.08	0.24	0.46	0.73	1.04	1.40	1.80	2.24	72
	48 inches	5	1.44	18.8	---	---	---	---	---	---	73
		6	0.61	2.83	10.8	---	---	---	---	---	74
		7	0.59	1.54	3.47	7.63	17.4	---	---	---	75
		8	0.59	1.48	2.58	4.15	6.65	10.6	19.5	---	76
		9	0.59	1.48	2.56	3.79	5.22	7.06	9.53	12.9	77
		10	0.59	1.48	2.56	3.79	5.14	6.60	8.23	10.1	78
	72 inches	5	4.22	---	---	---	---	---	---	---	79
		6	1.32	7.06	---	---	---	---	---	---	80
		7	1.22	3.22	7.77	18.5	---	---	---	---	81
		8	1.22	2.93	5.14	8.66	14.6	41.3	---	---	82
		9	1.22	2.93	4.93	7.20	10.1	14.0	19.7	---	83
		10	1.22	2.93	4.93	7.15	9.56	12.2	15.3	19.1	84

Note: Default layering correction applied.

Options for Response of Layered Soils

- [x] Use Layering Correction (Method of Georgiadis)
- [ ] Do not Compute Layering Correction if Layer Above is of Same Type

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

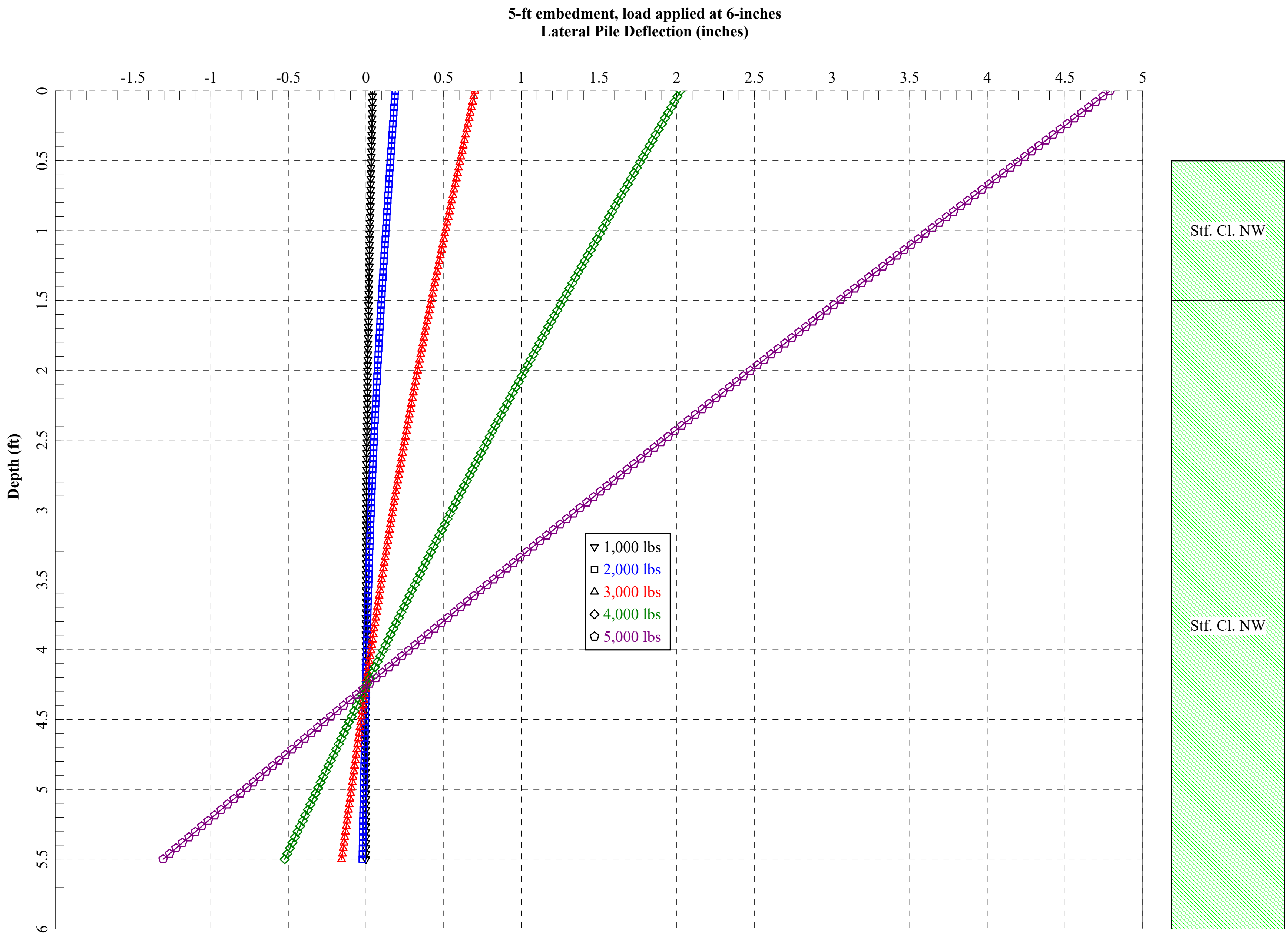


Figure No. 49

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

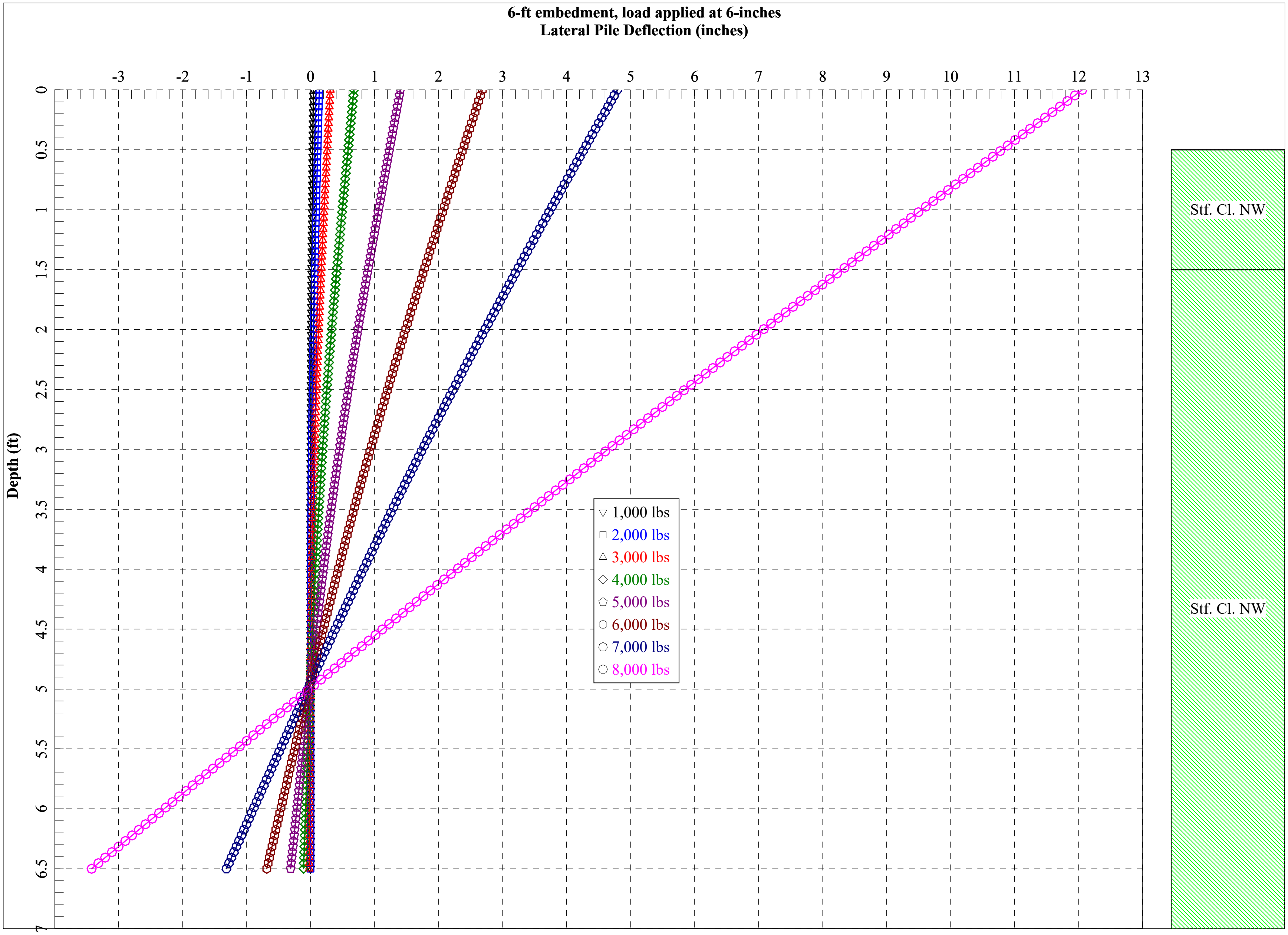


Figure No. 50

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

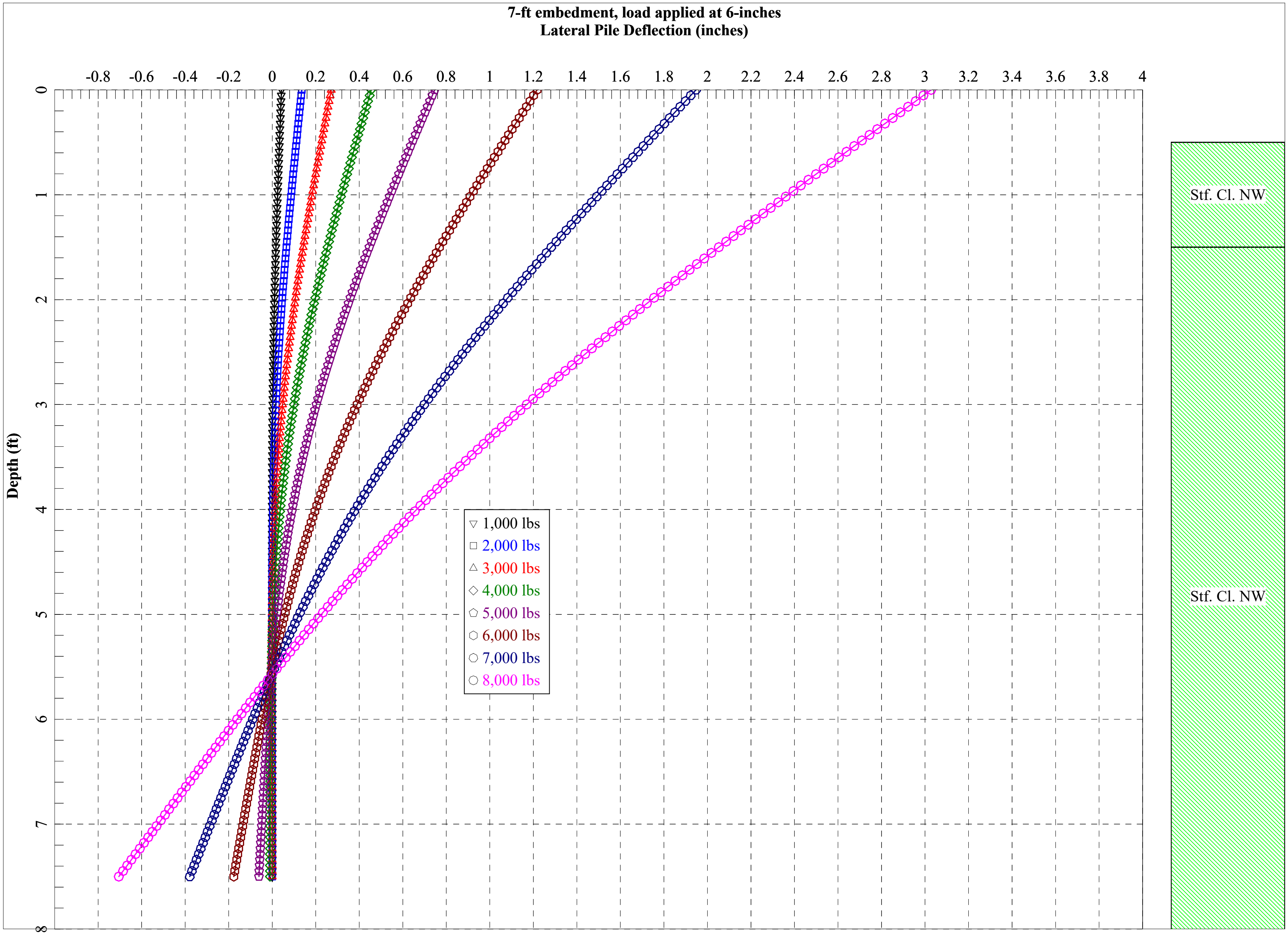


Figure No. 51

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

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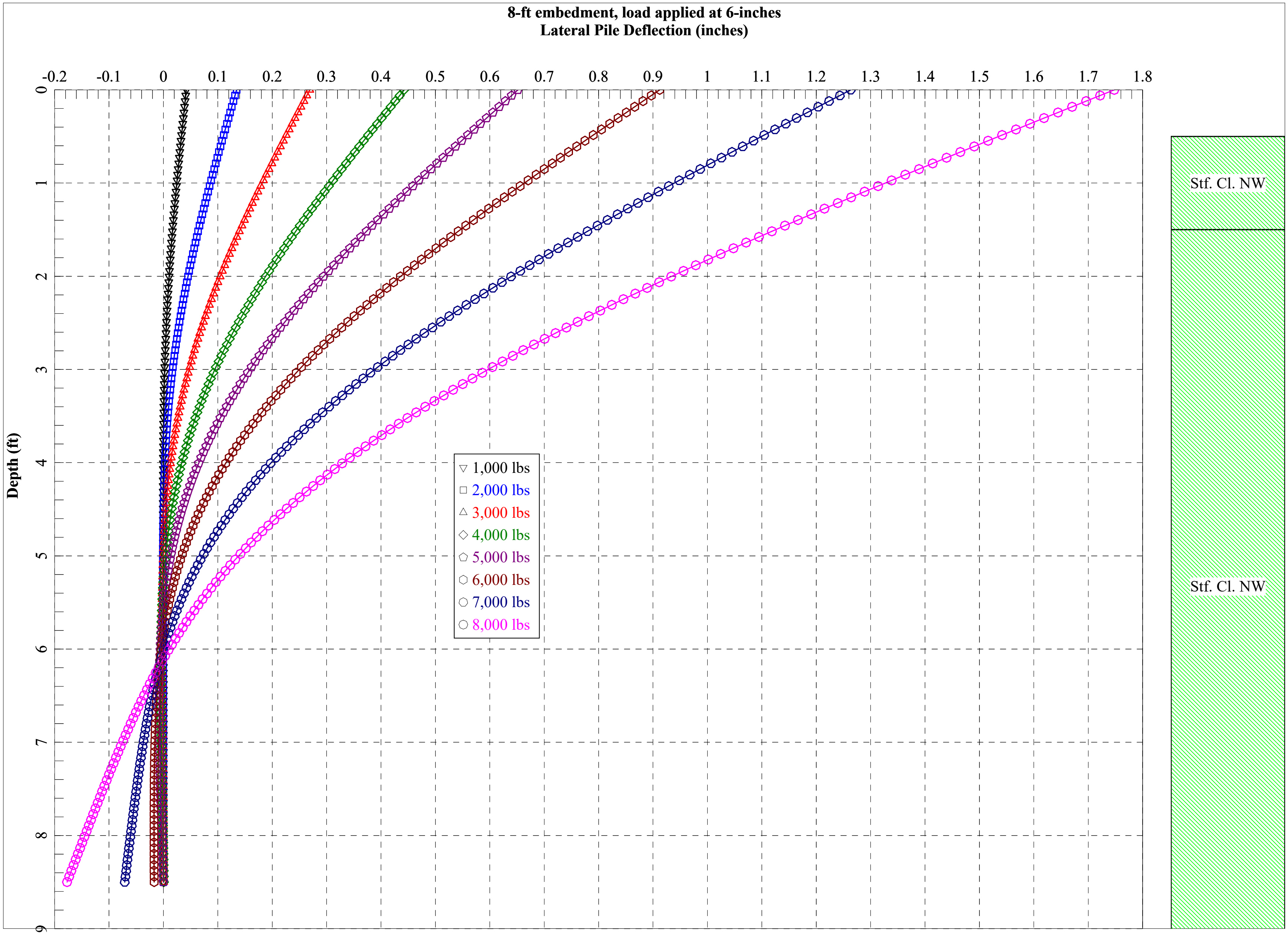

Figure No. 52

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

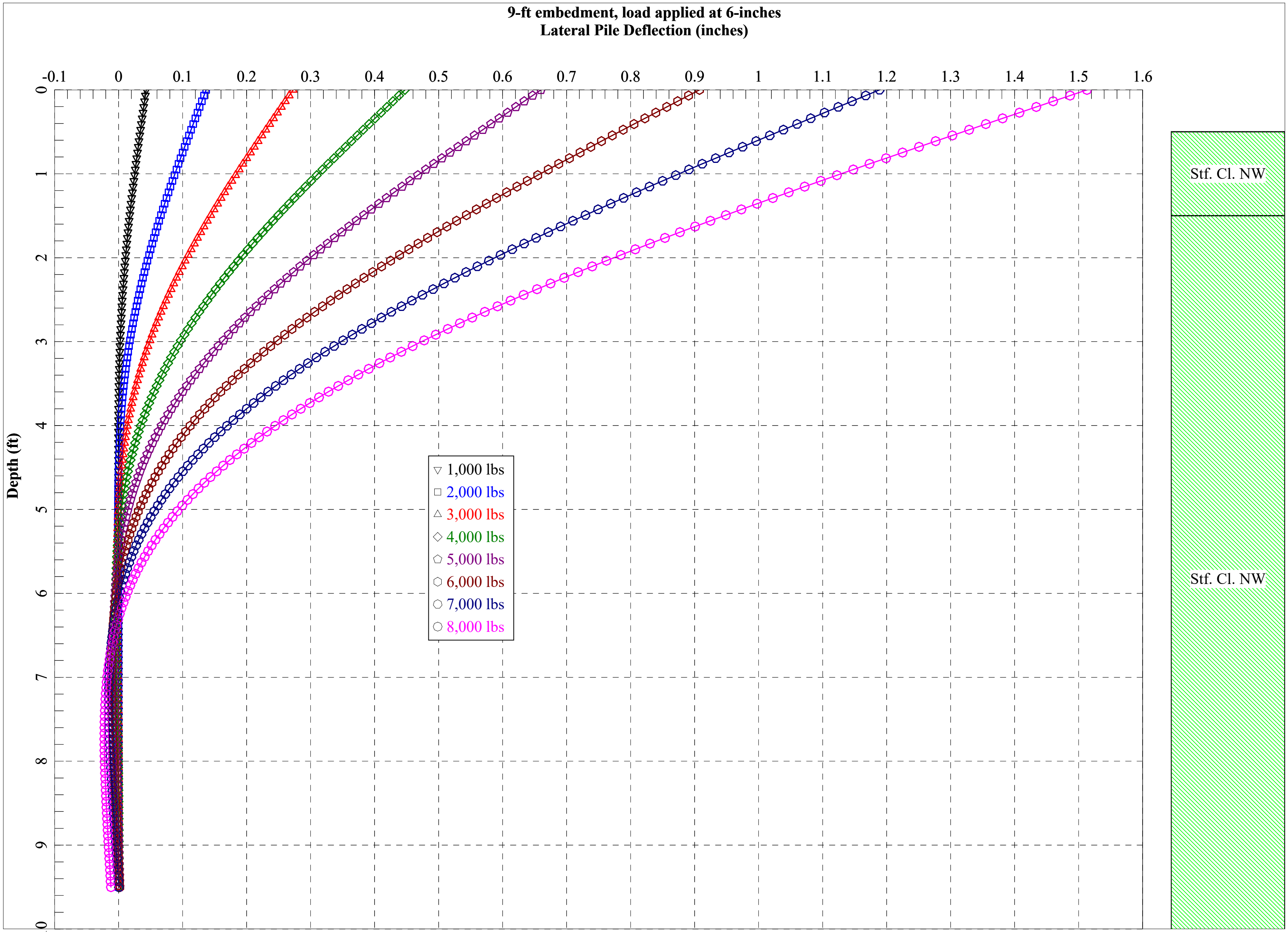

Figure No. 53

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

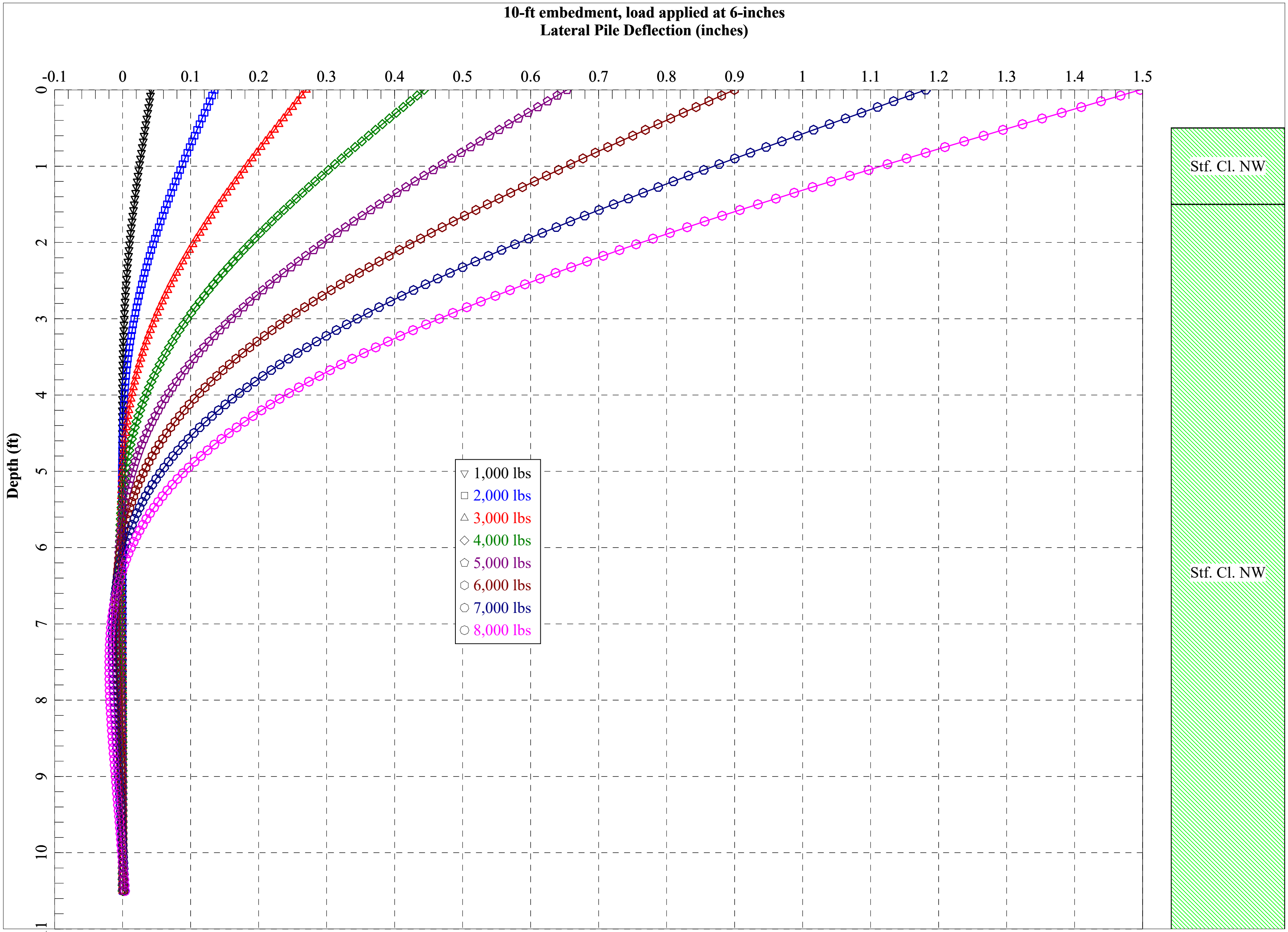

Figure No. 54

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

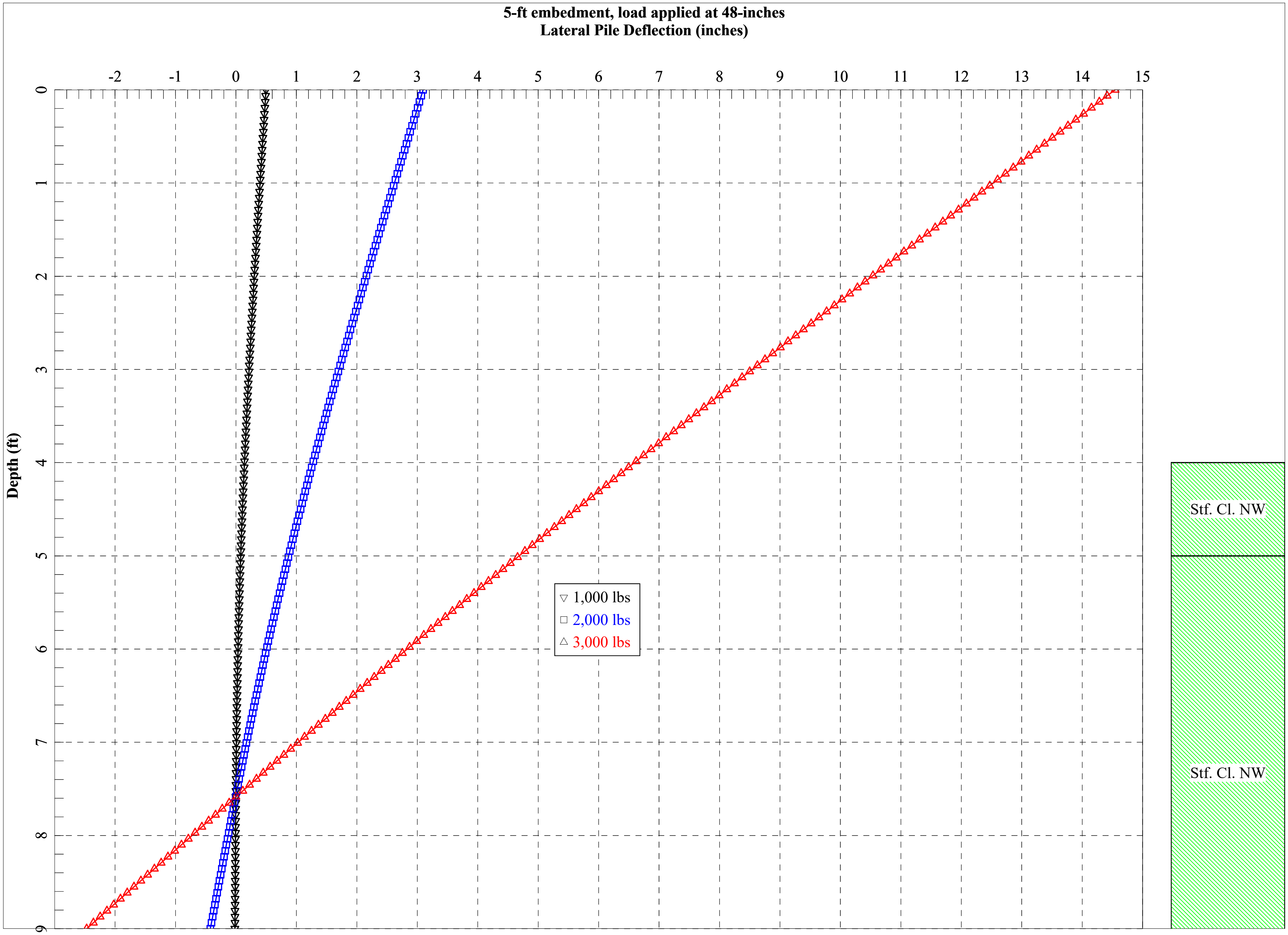

Figure No. 55

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

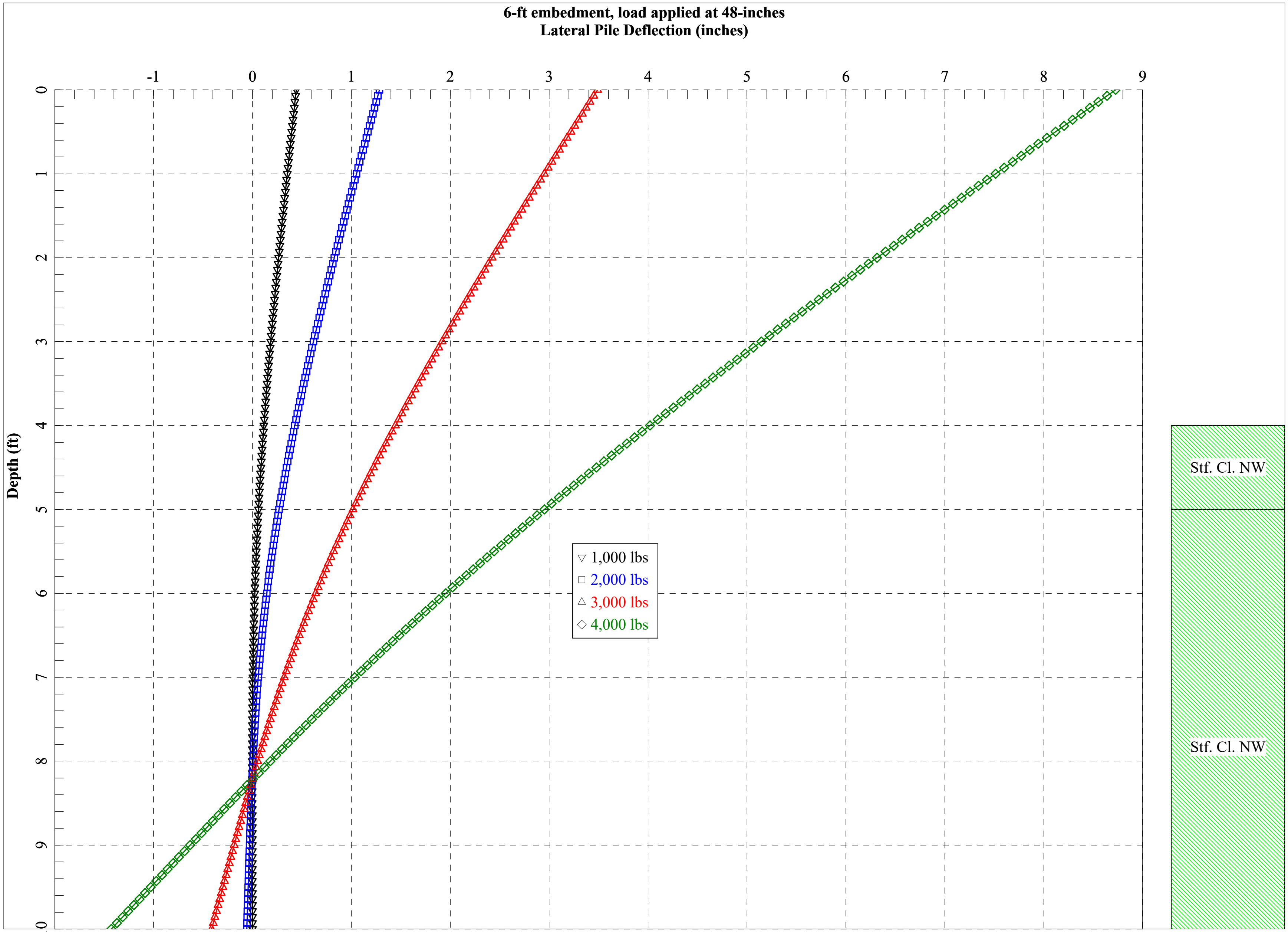

Figure No. 56

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

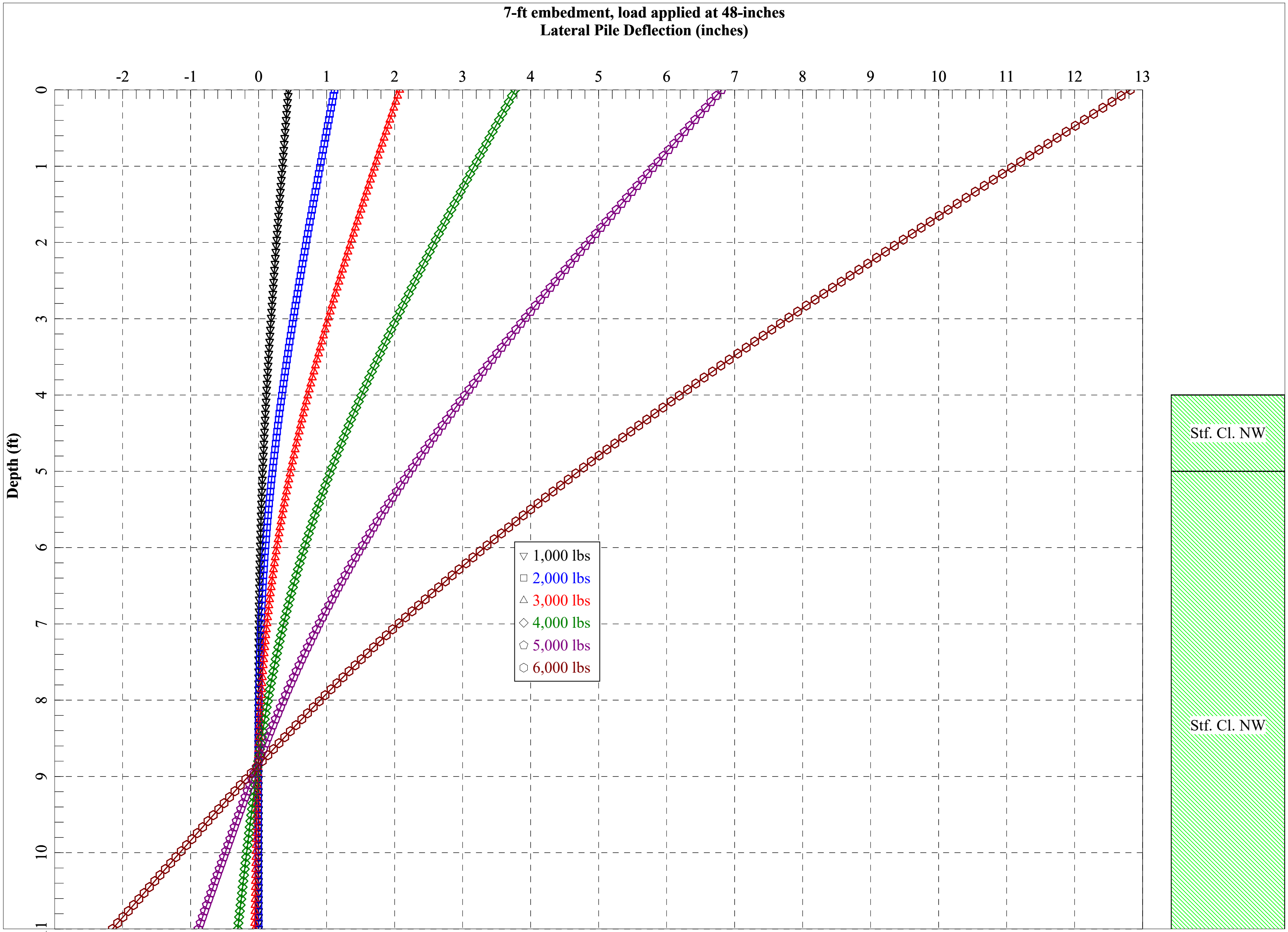

Figure No. 57

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

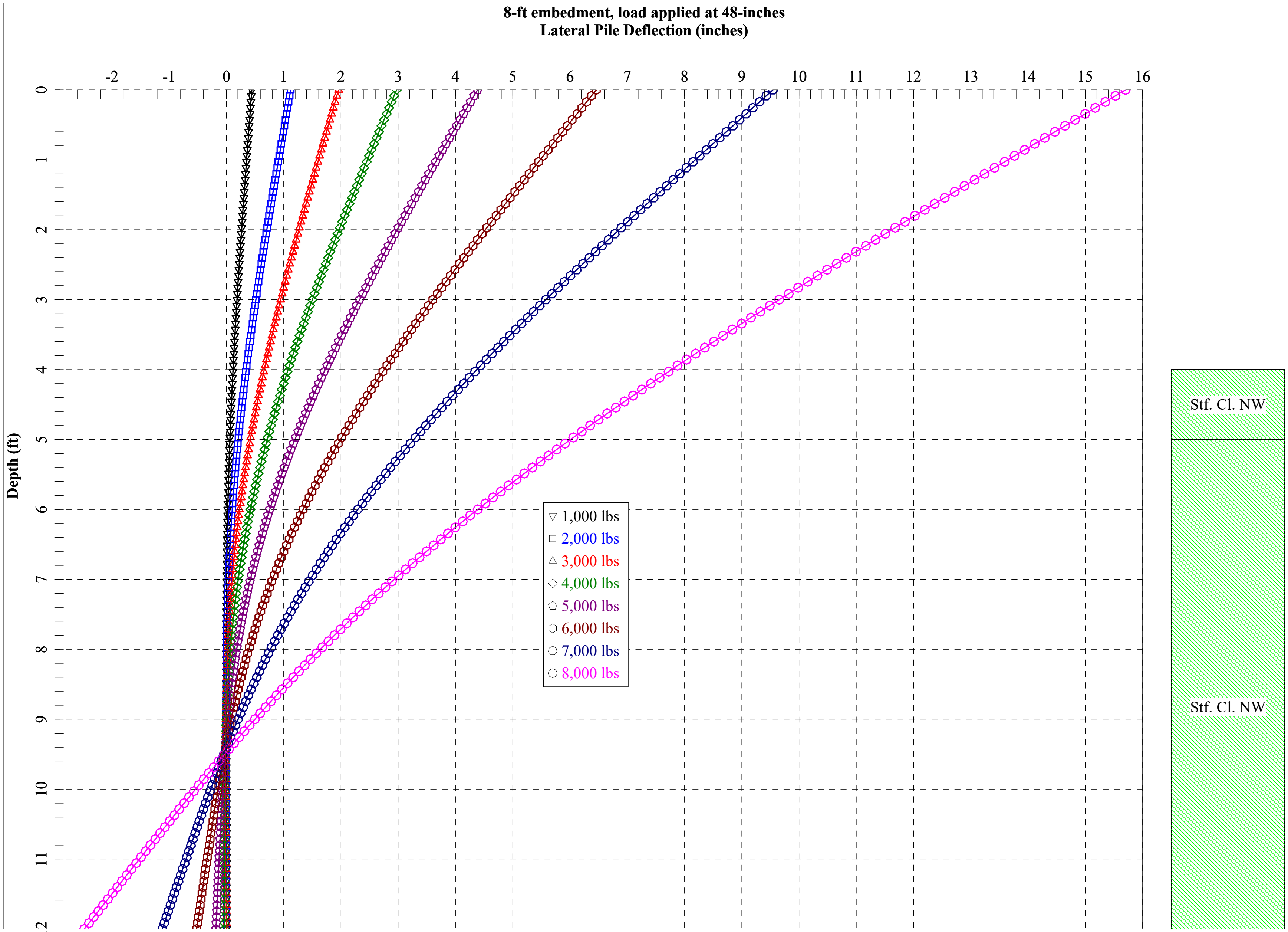

Figure No. 58

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

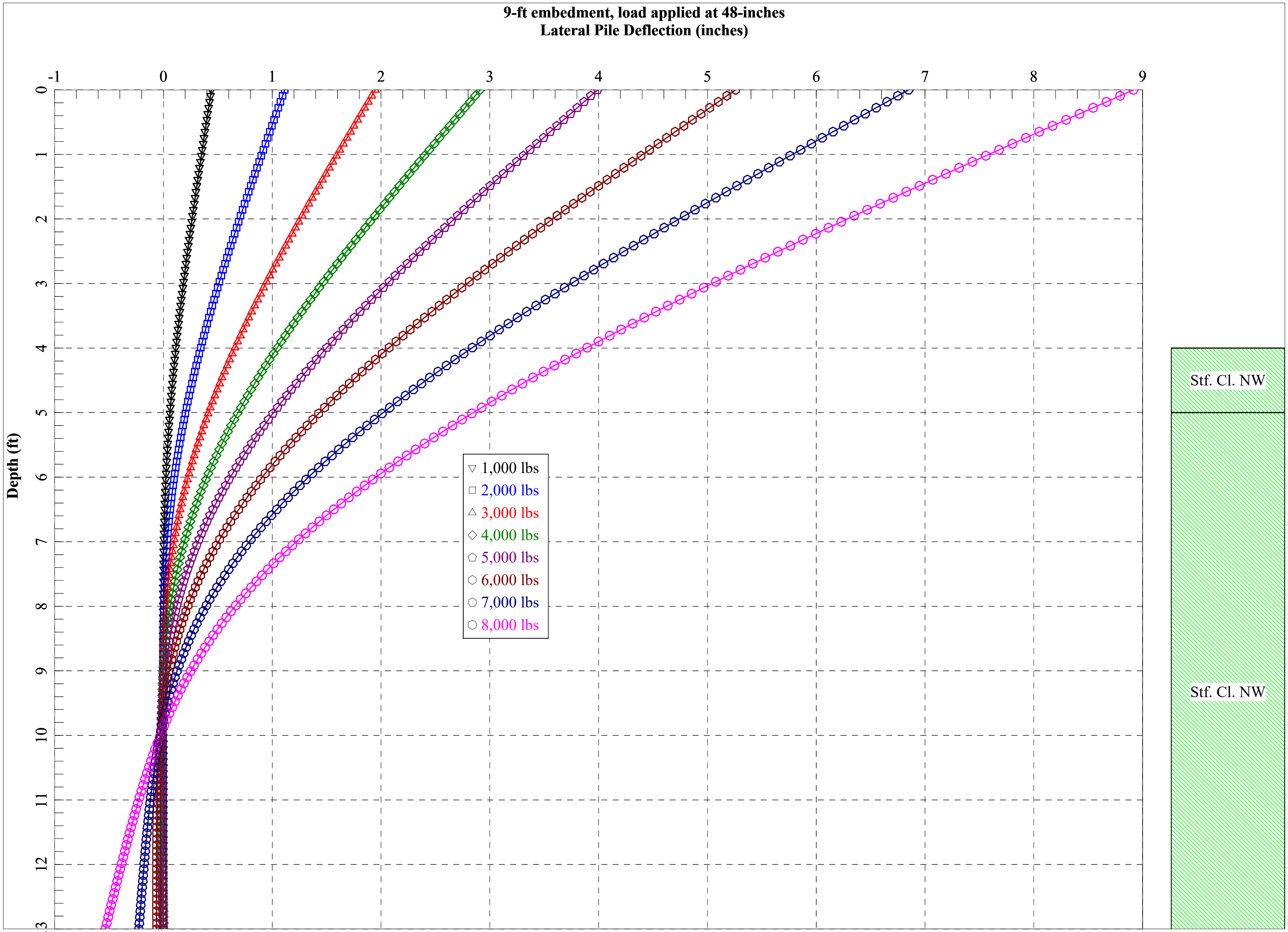

Figure No. 59

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

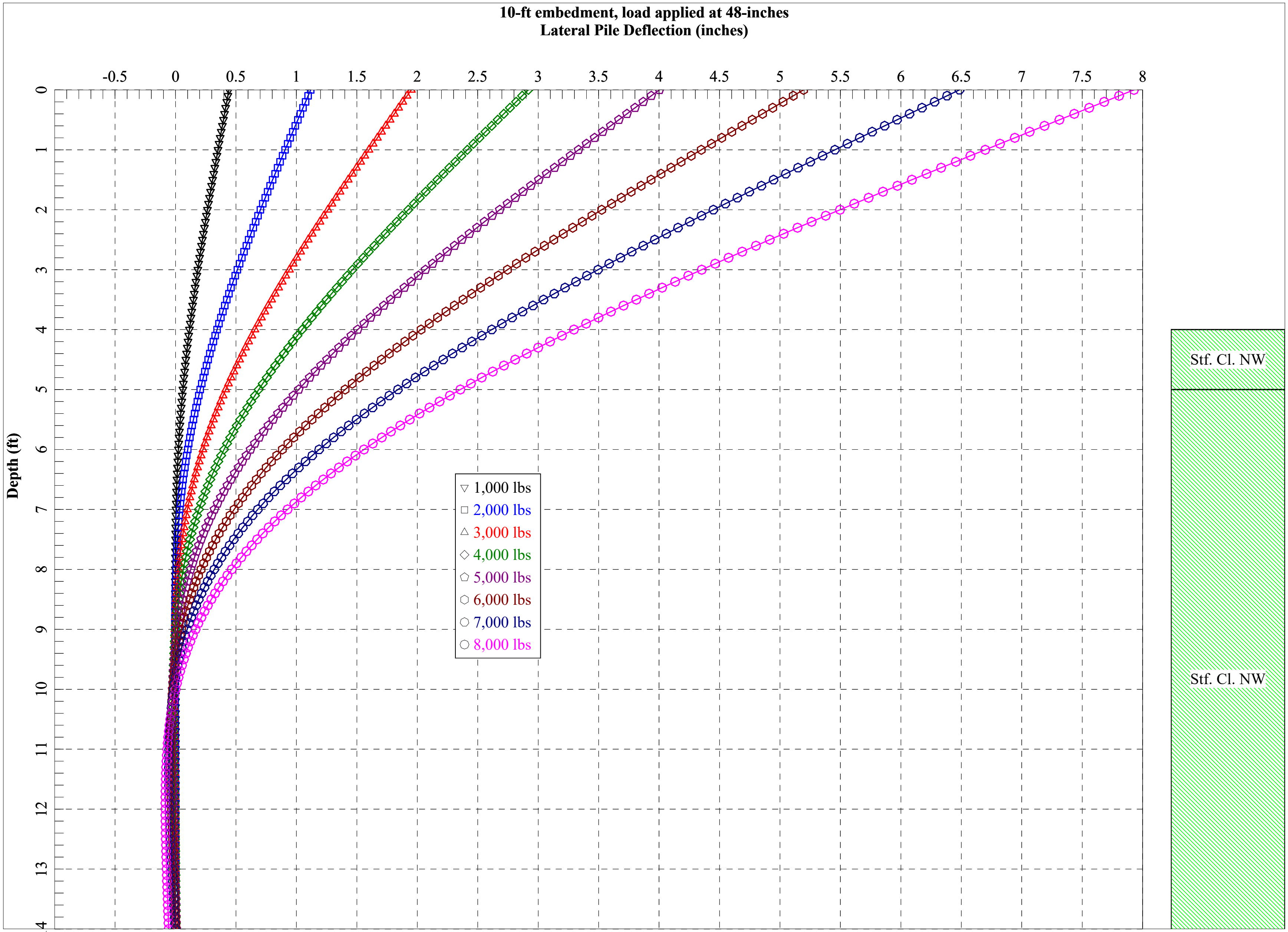

Figure No. 60

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

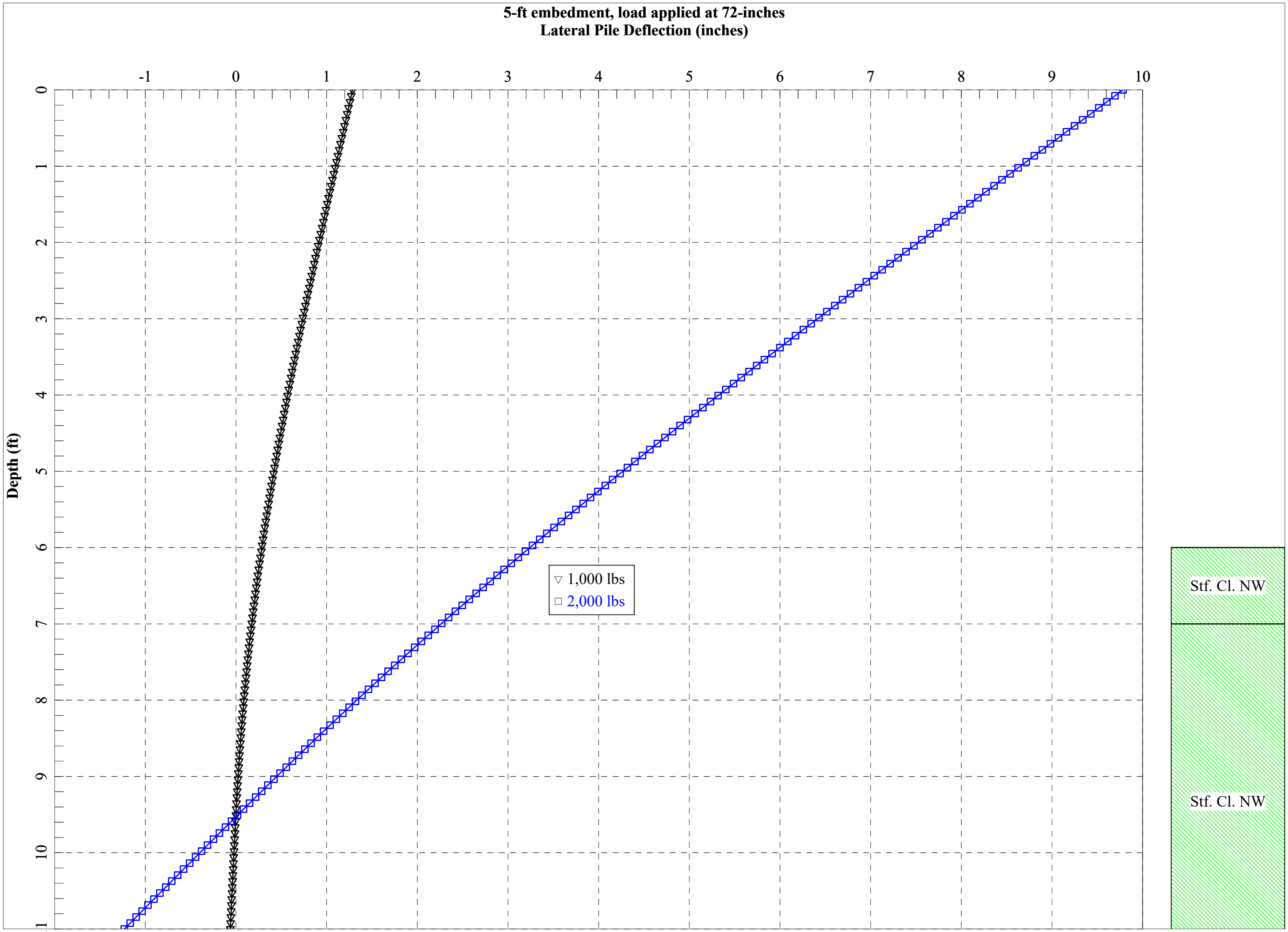

Figure No. 61

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

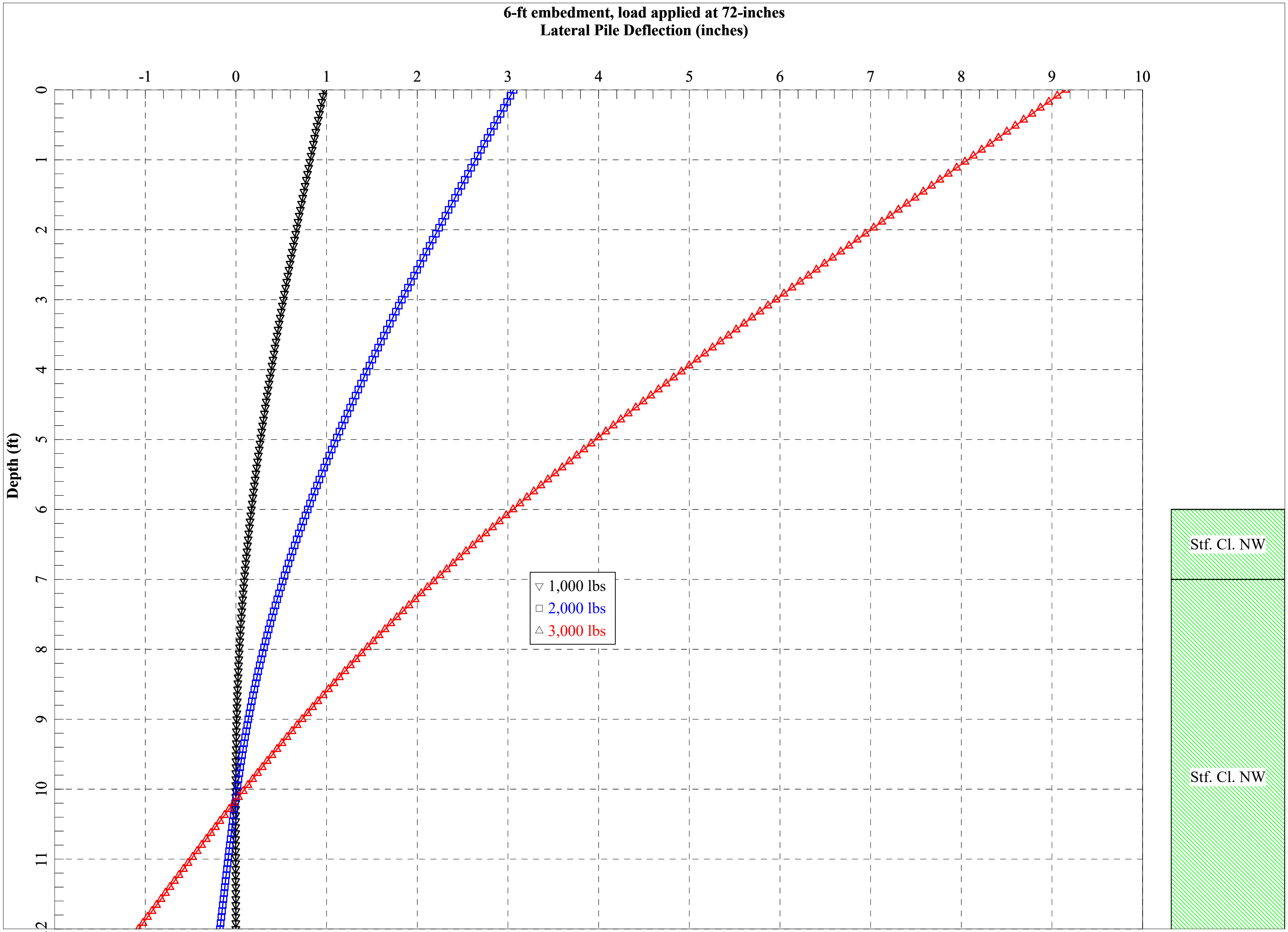

Figure No. 62

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

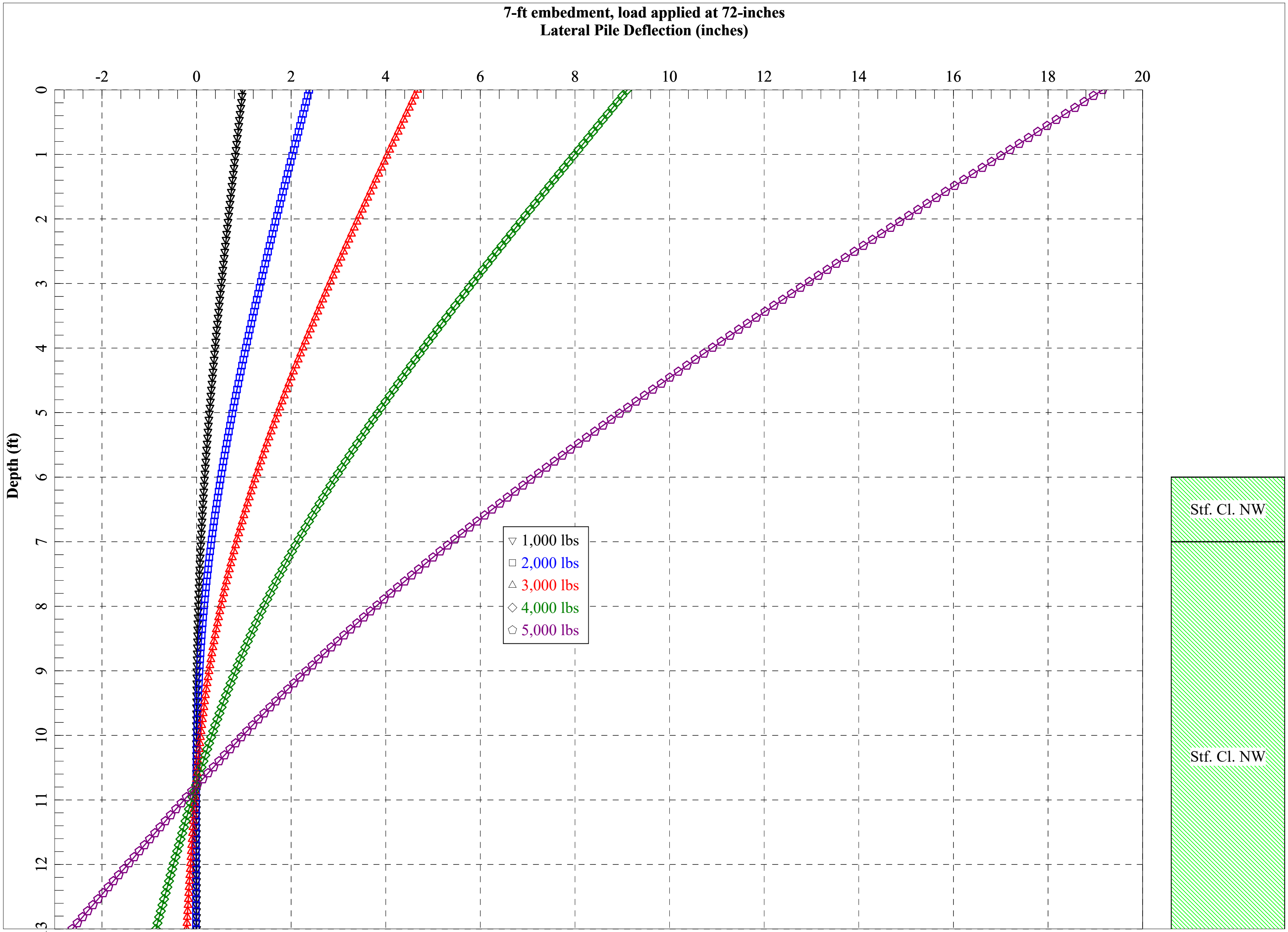

Figure No. 63

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

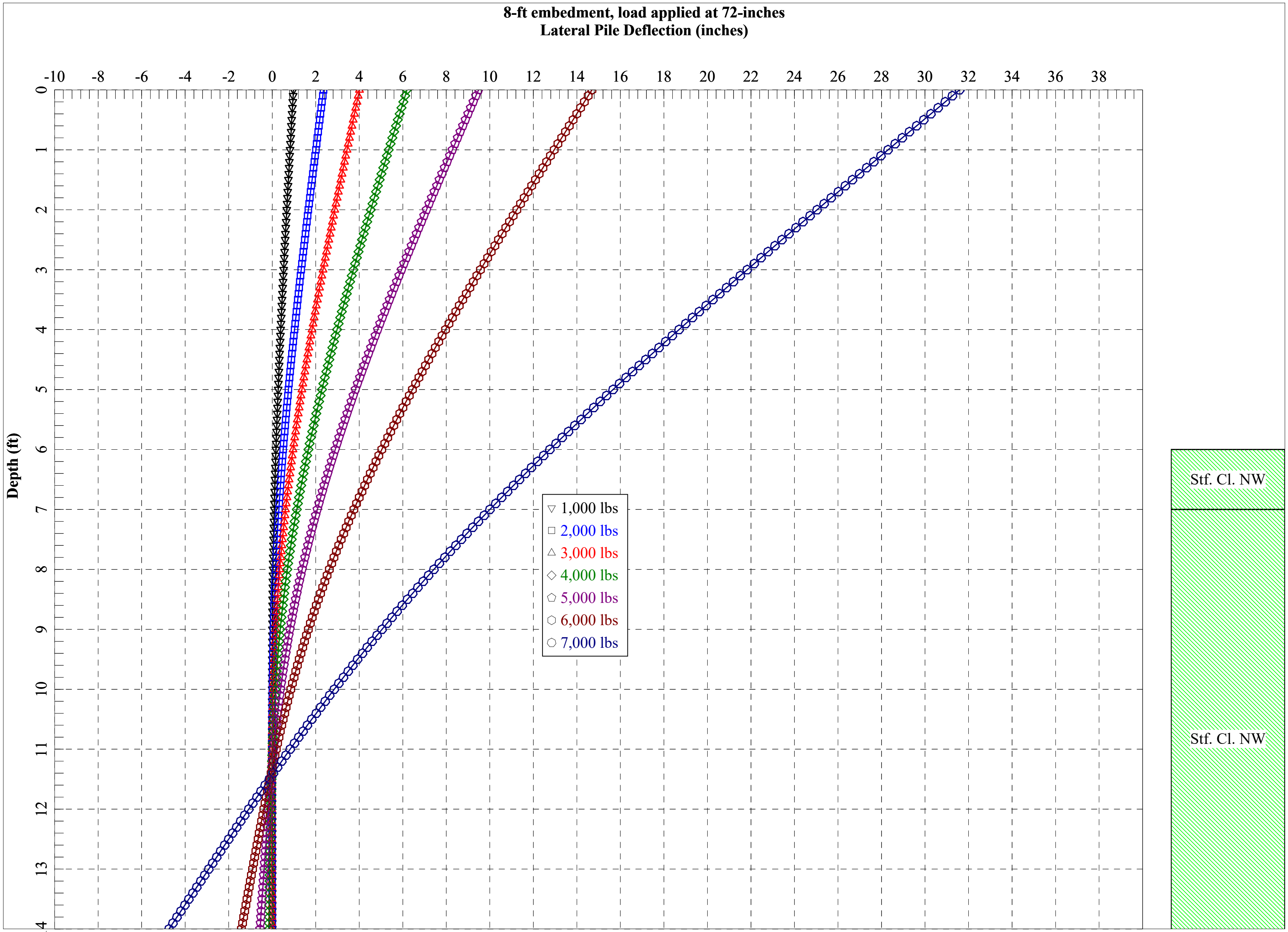

Figure No. 64

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

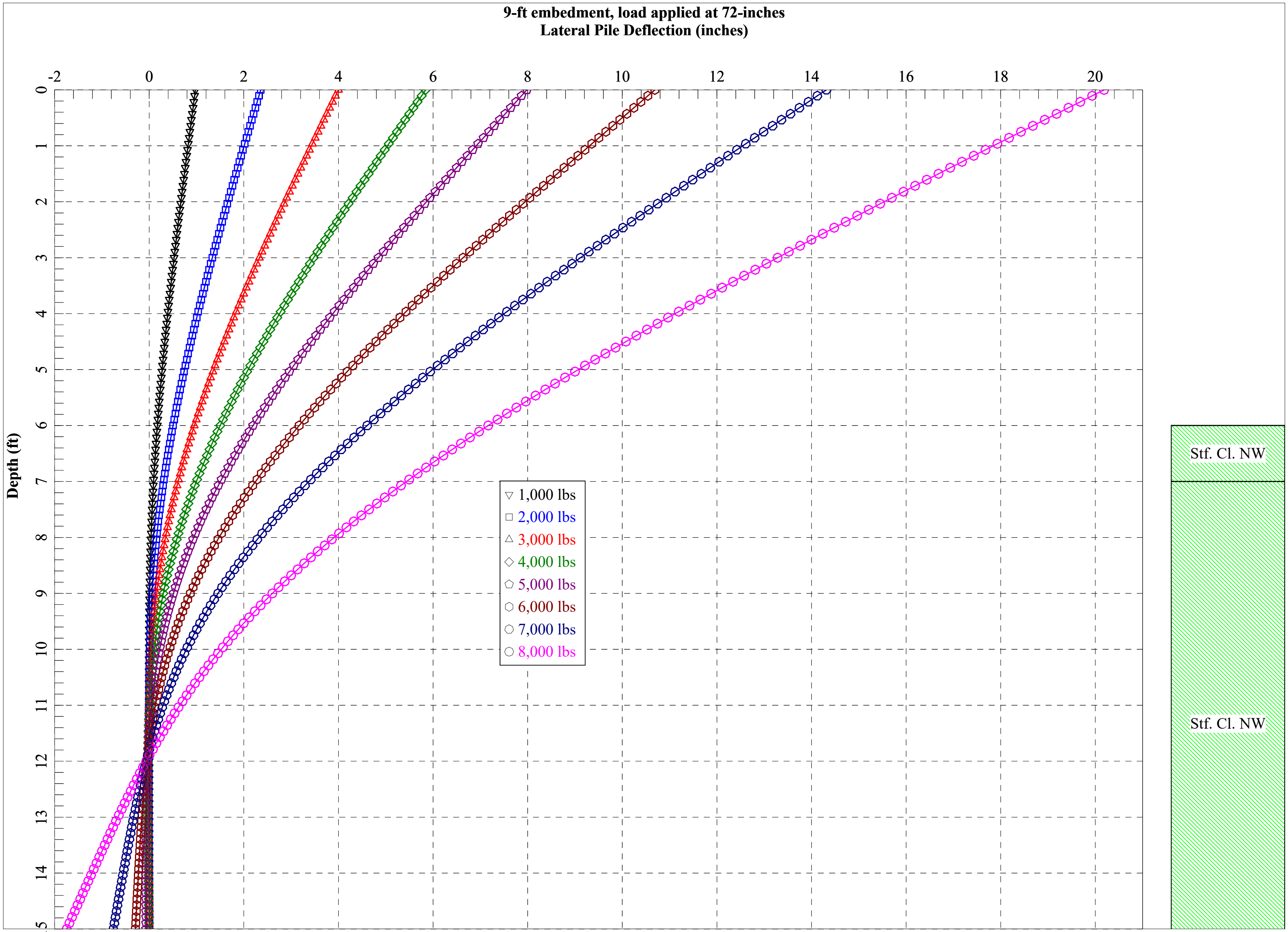

Figure No. 65

W6x9 LATERAL CAPACITY AREA NO. 1
(See Capacity Area Plan, Plate No. 3)

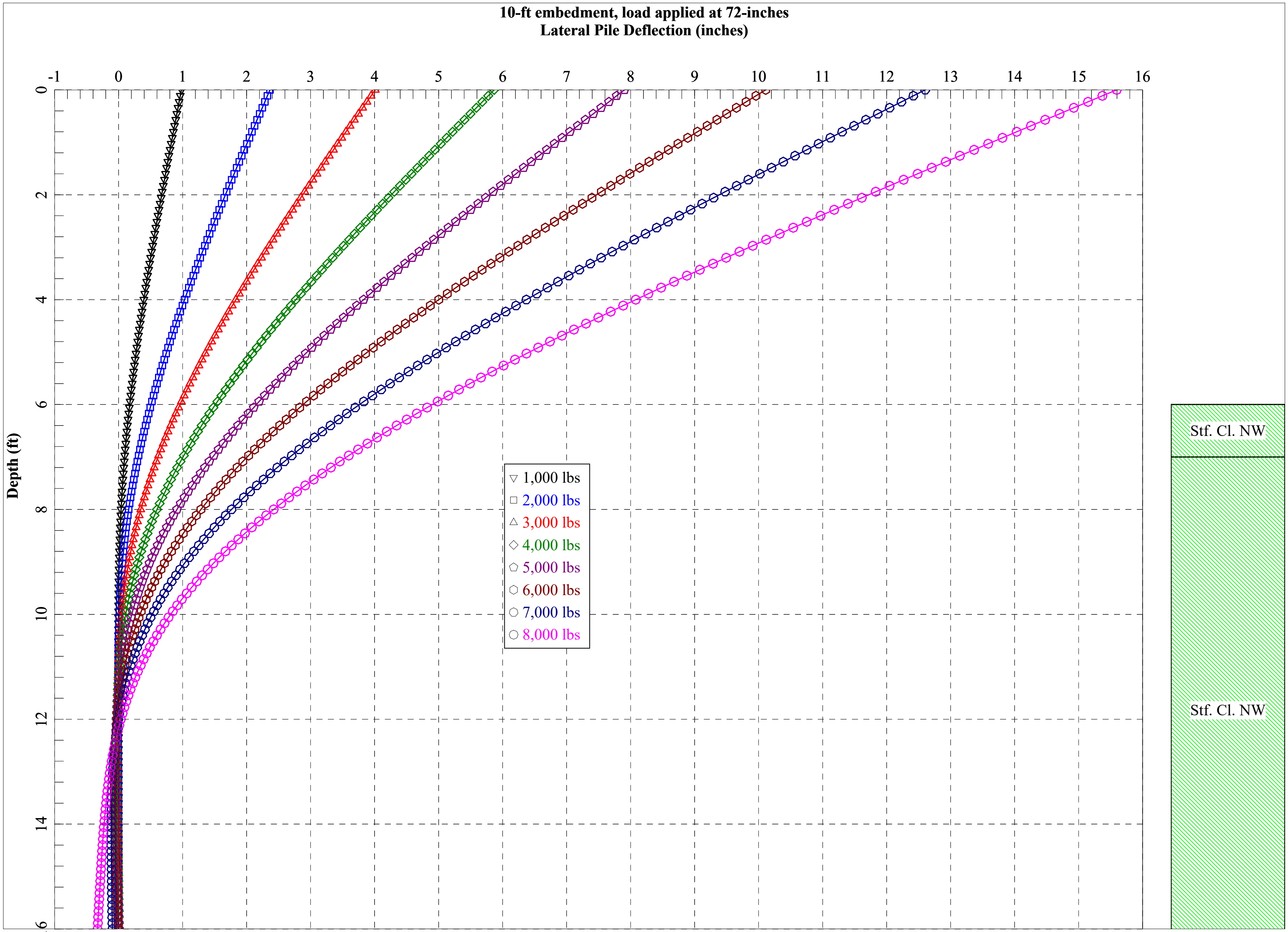

Figure No. 66

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

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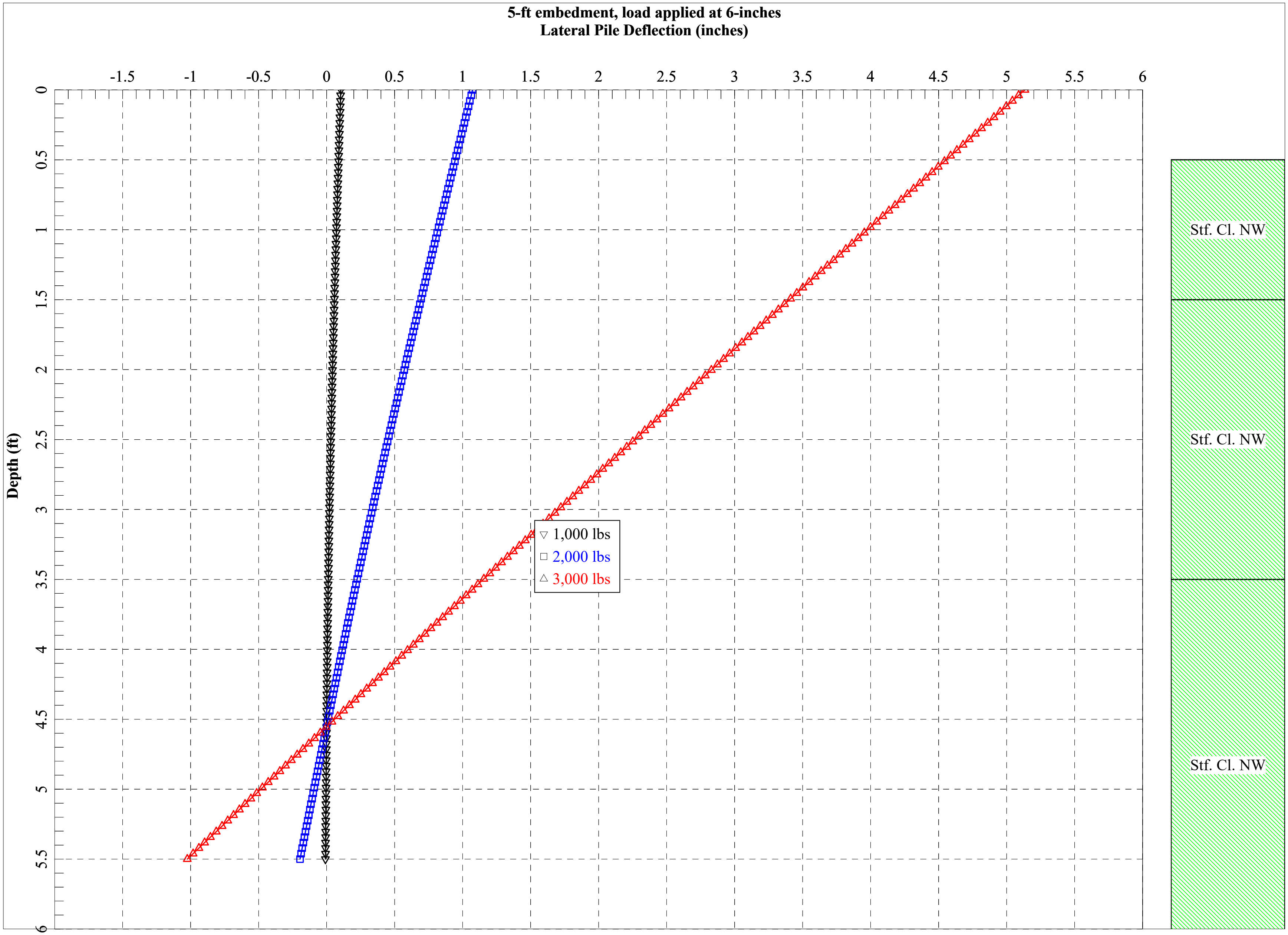

Figure No. 67

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

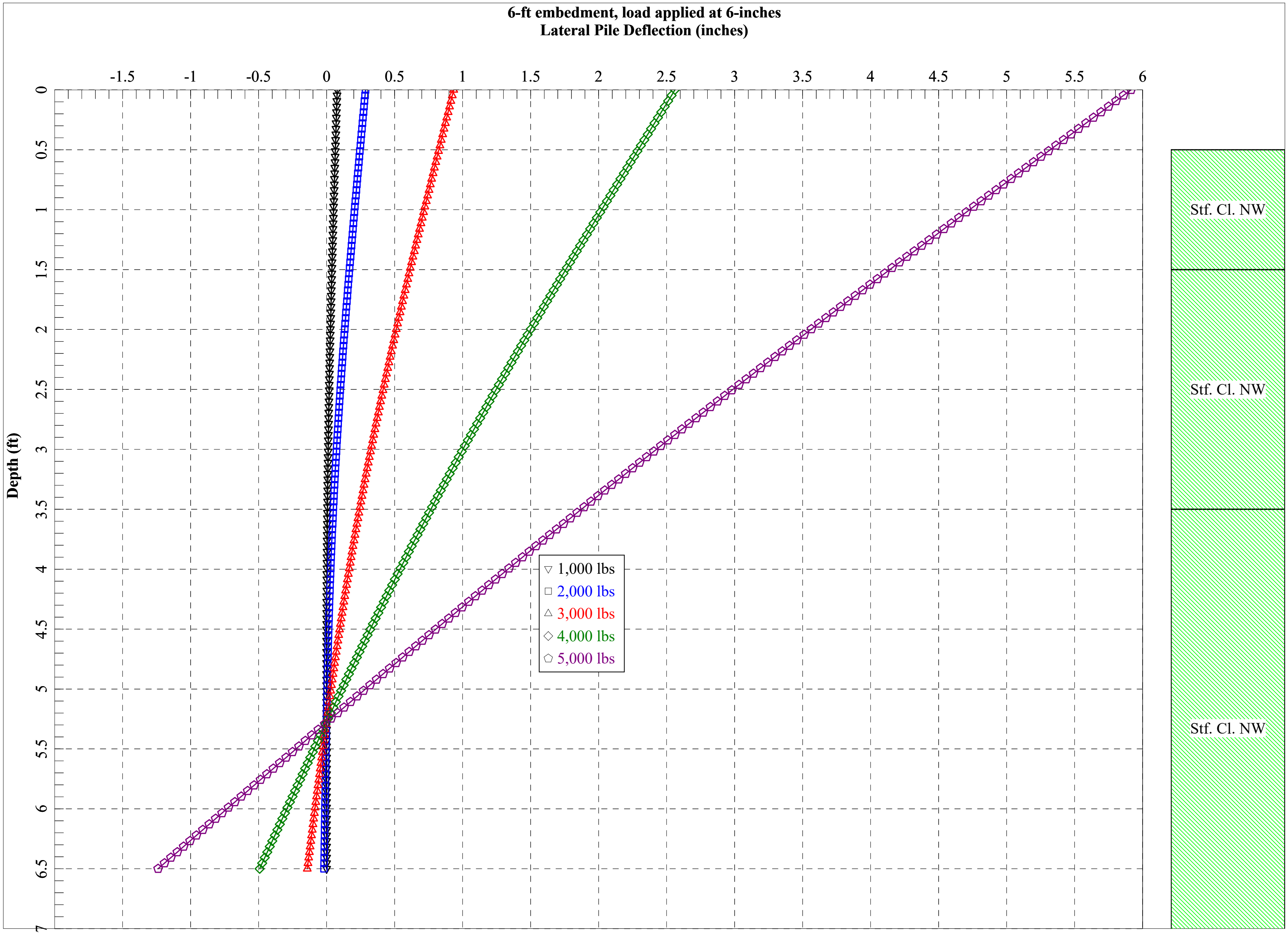

Figure No. 68

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

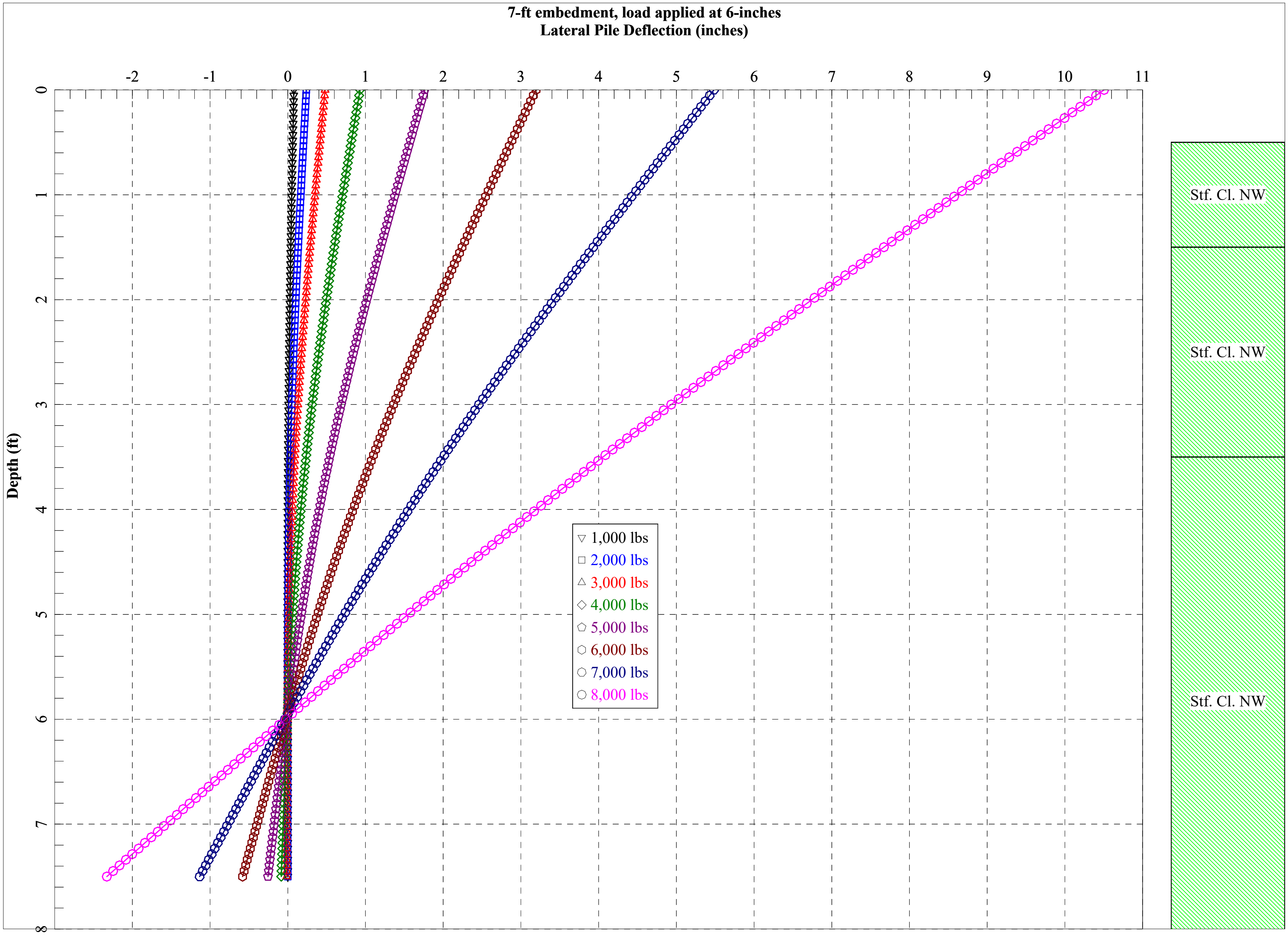

Figure No. 69

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

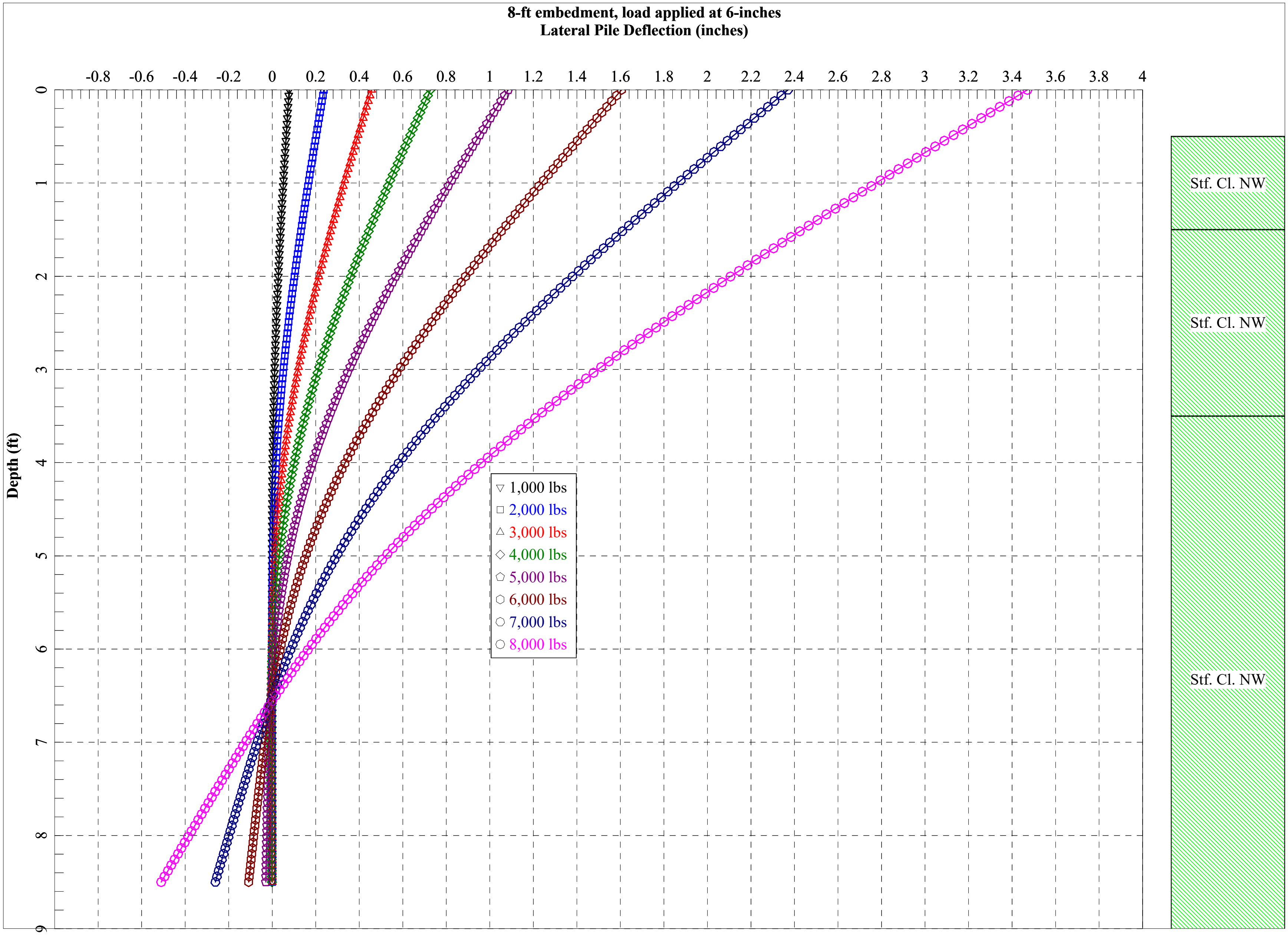

Figure No. 70

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

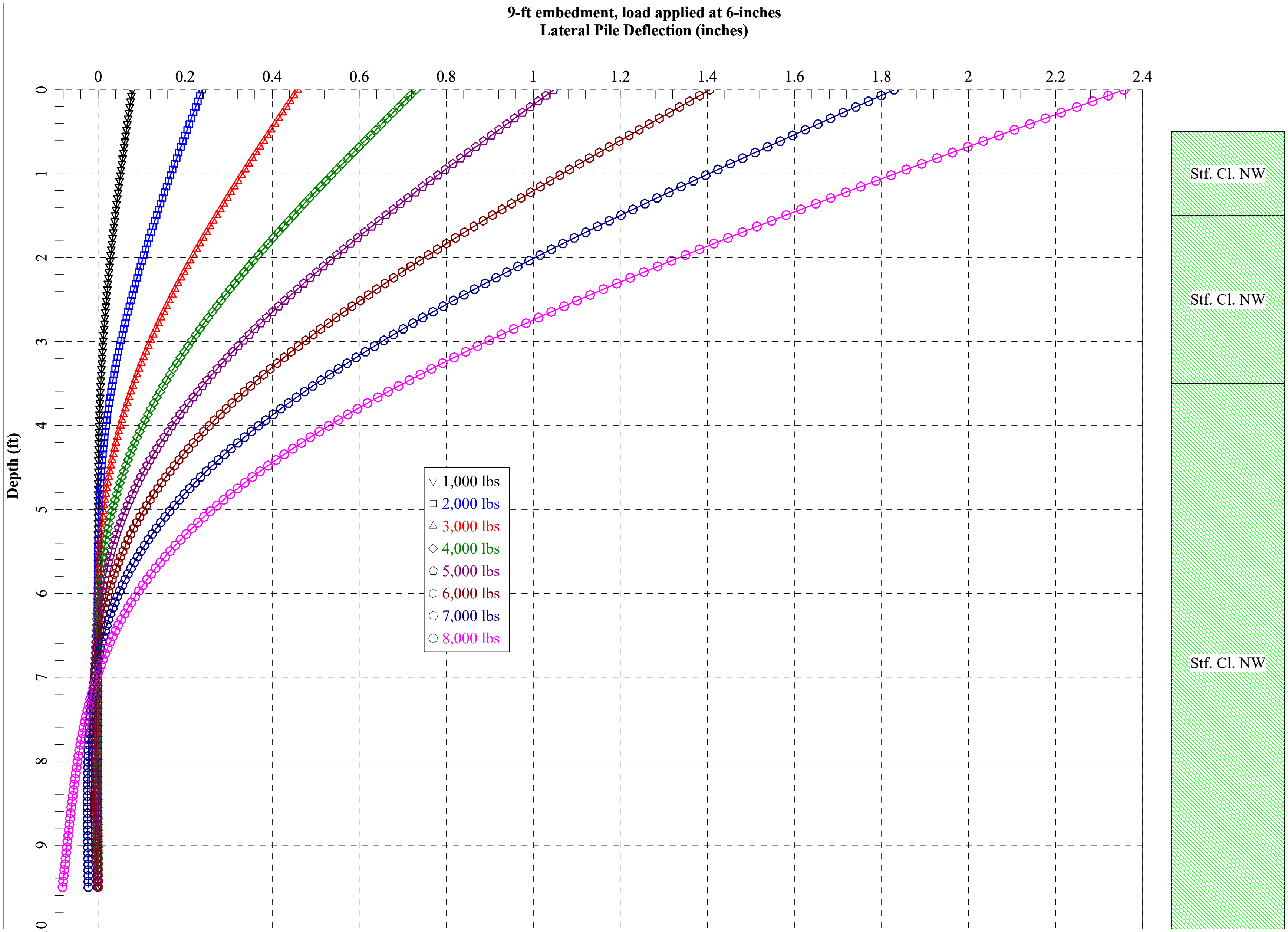

Figure No. 71

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

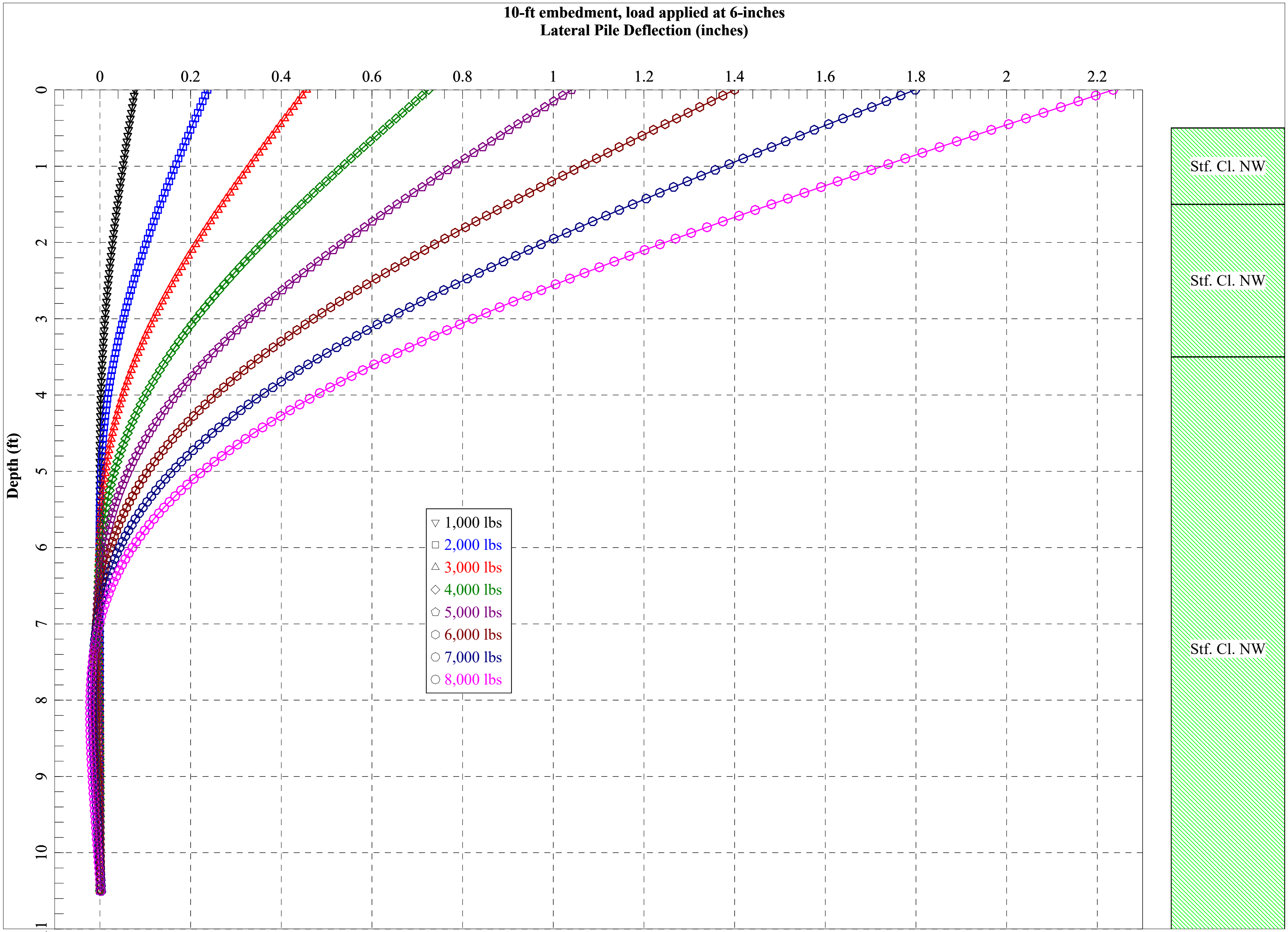

Figure No. 72

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

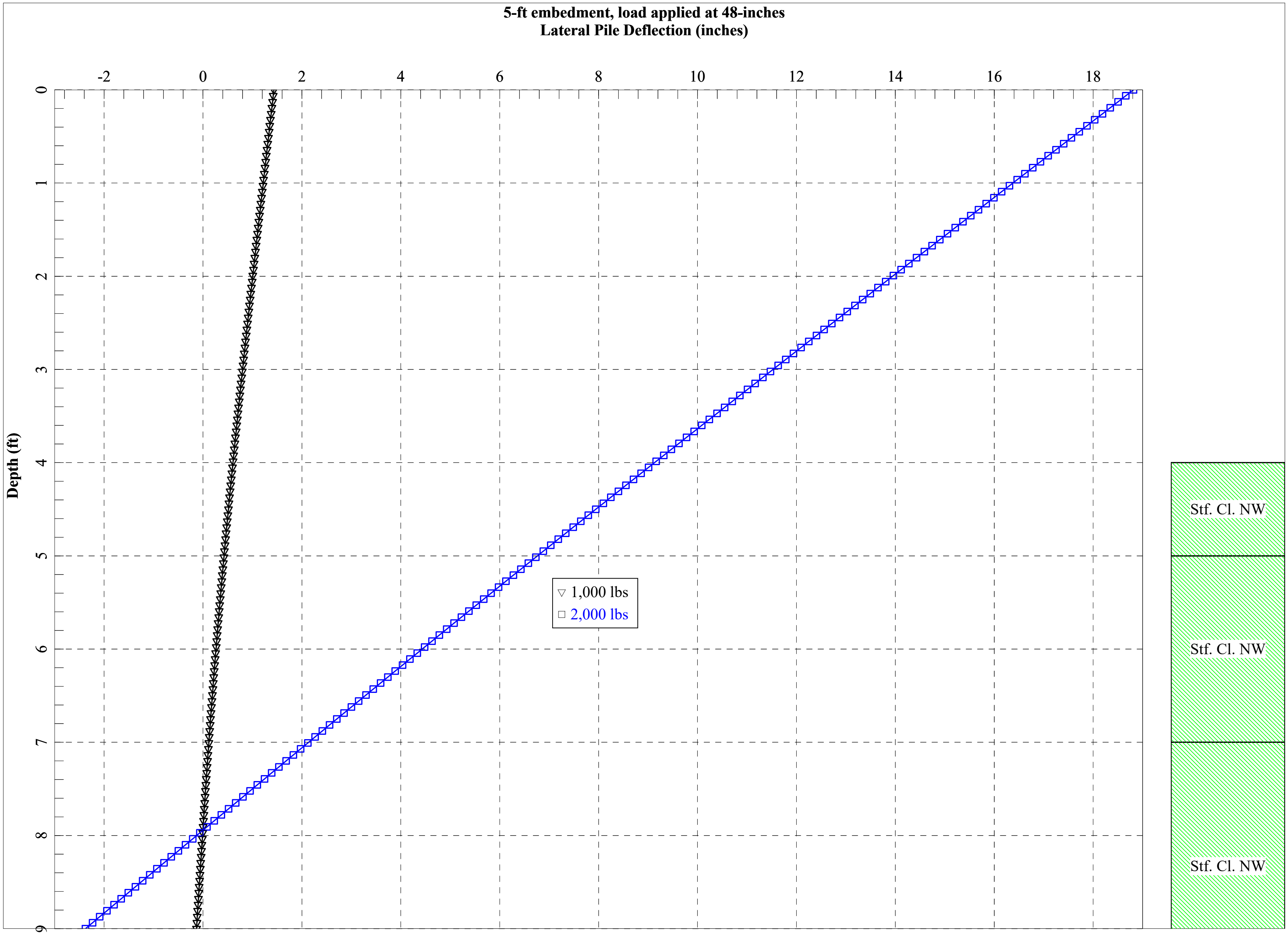

Figure No. 73

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

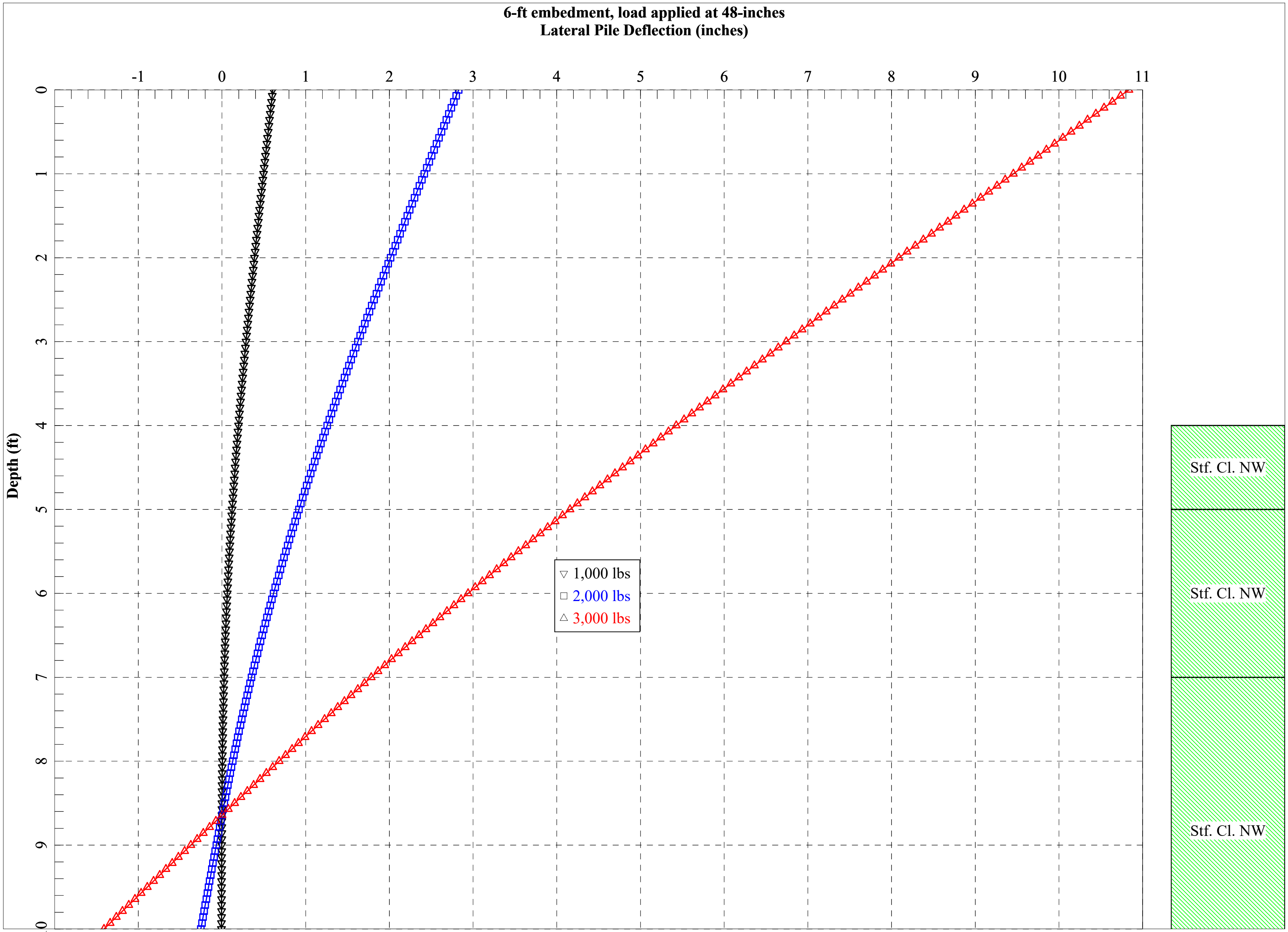

Figure No. 74

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

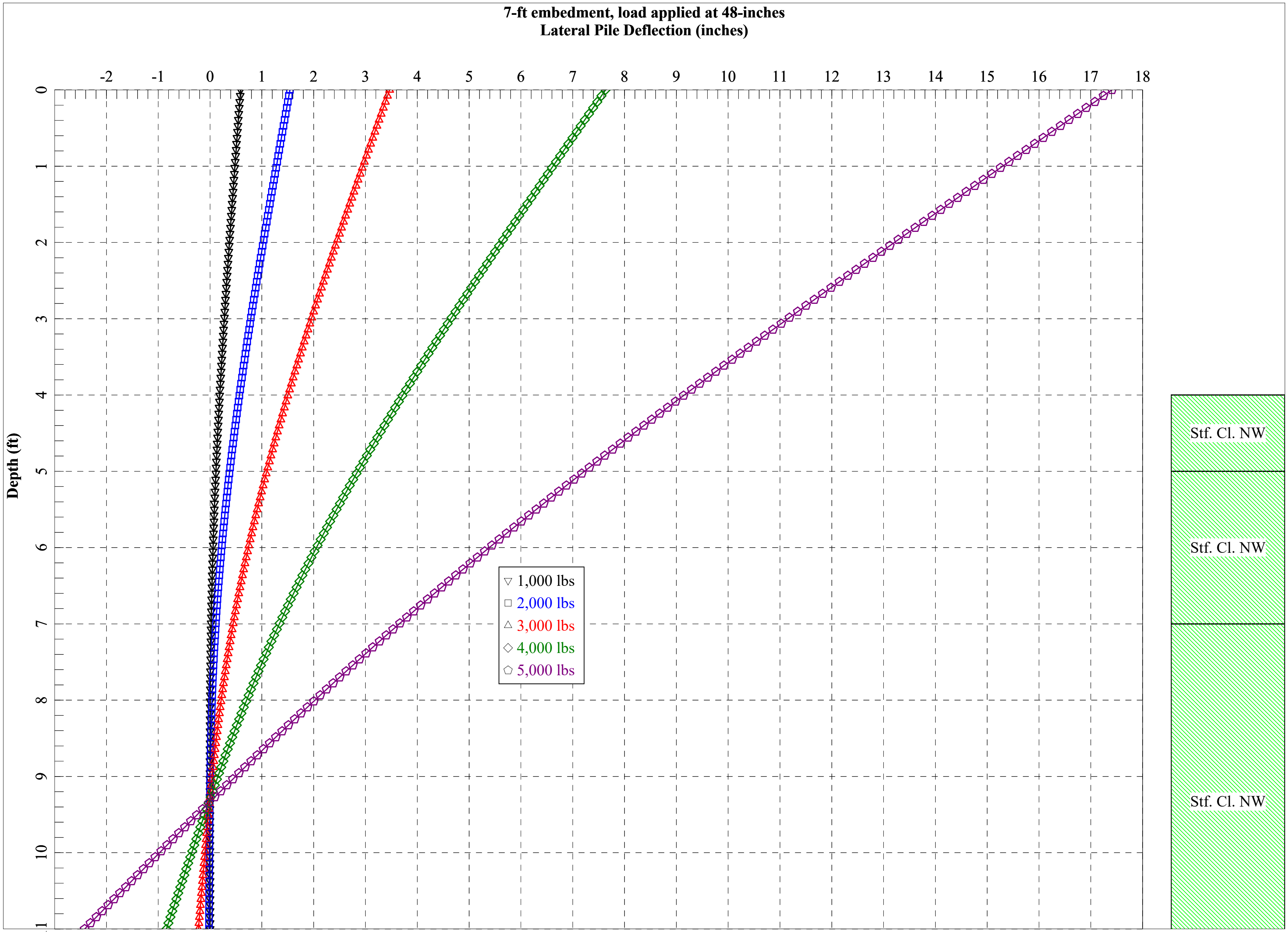

Figure No. 75

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

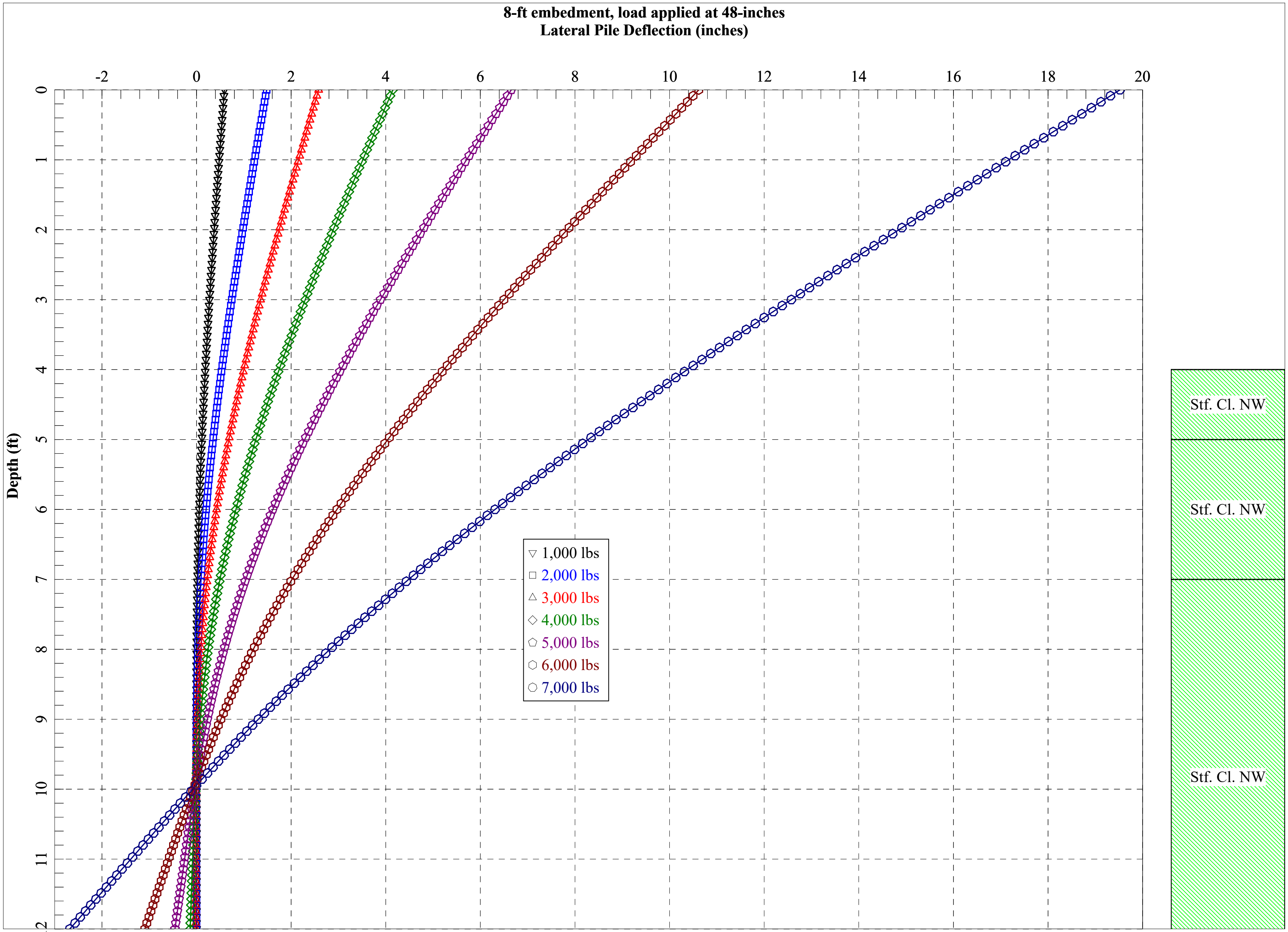

Figure No. 76

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

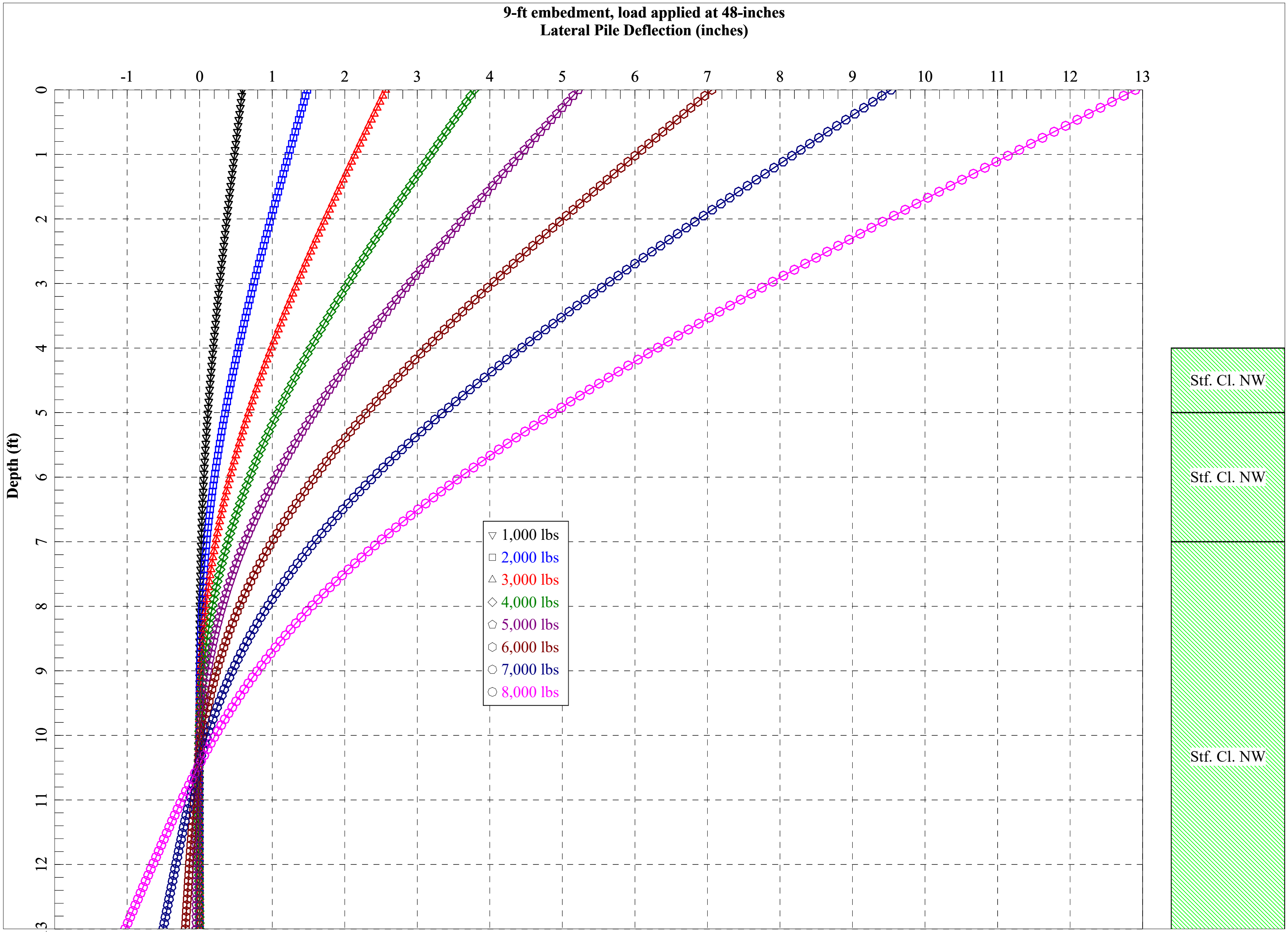

Figure No. 77

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

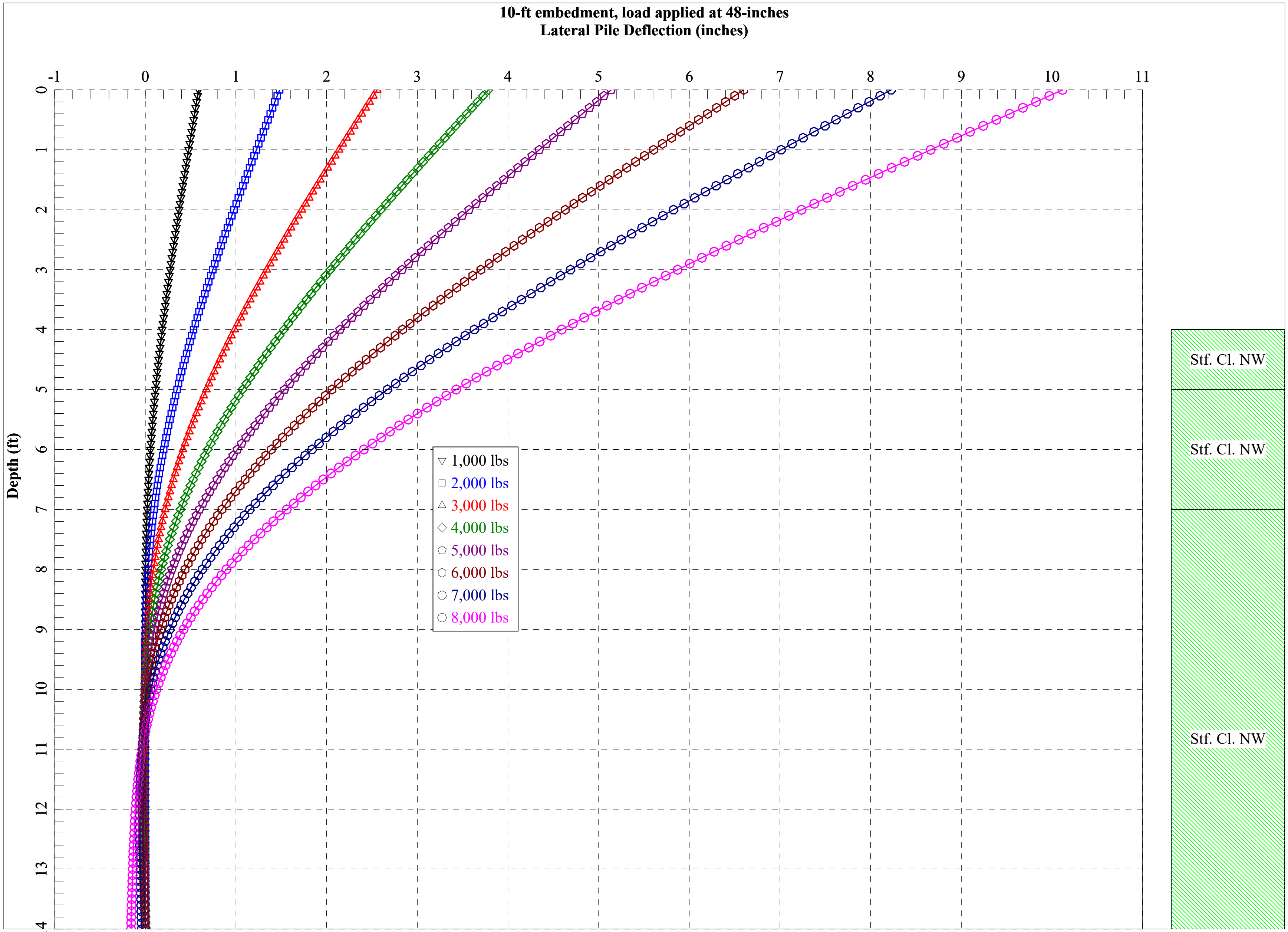

Figure No. 78

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

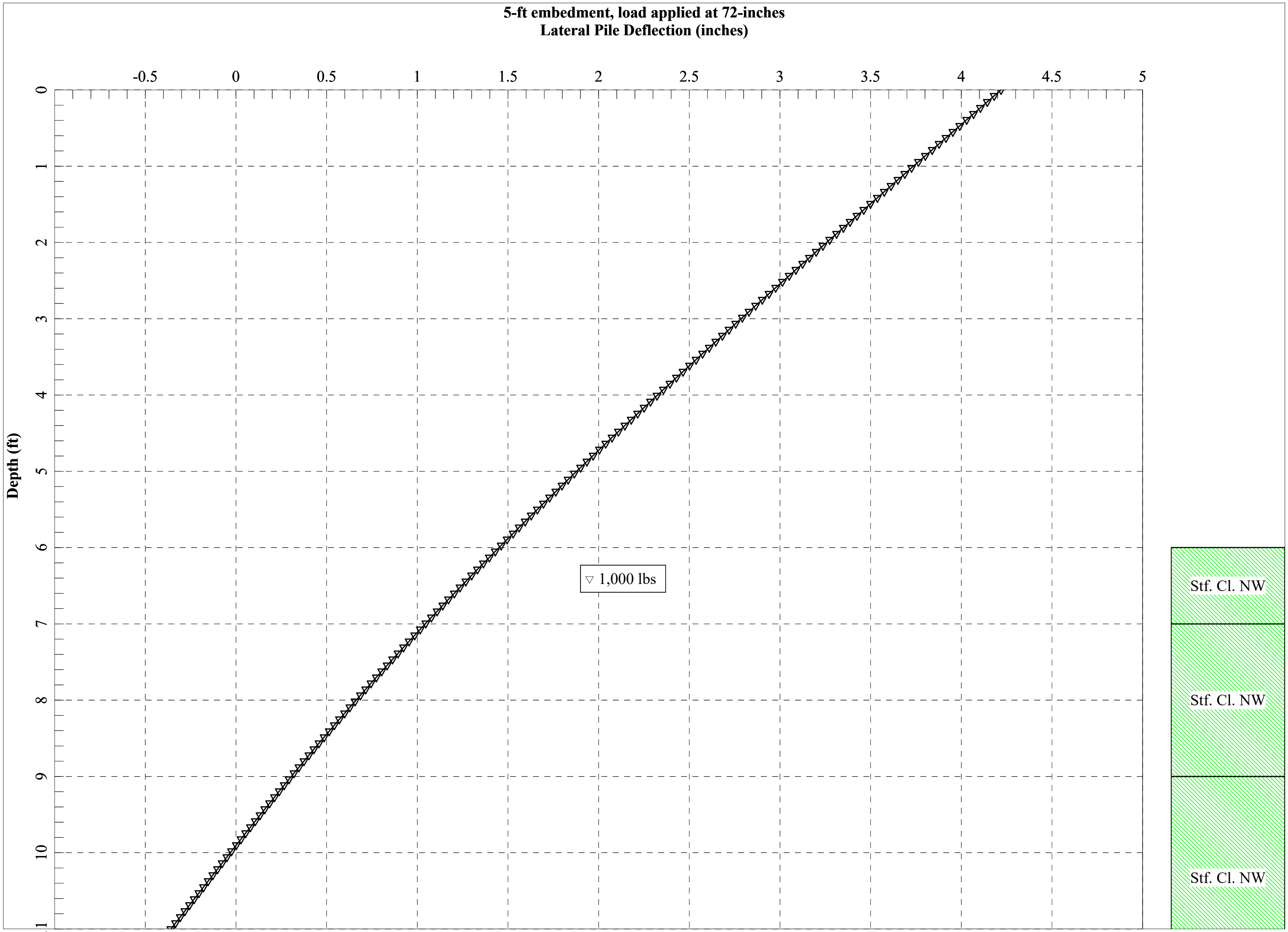

Figure No. 79

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

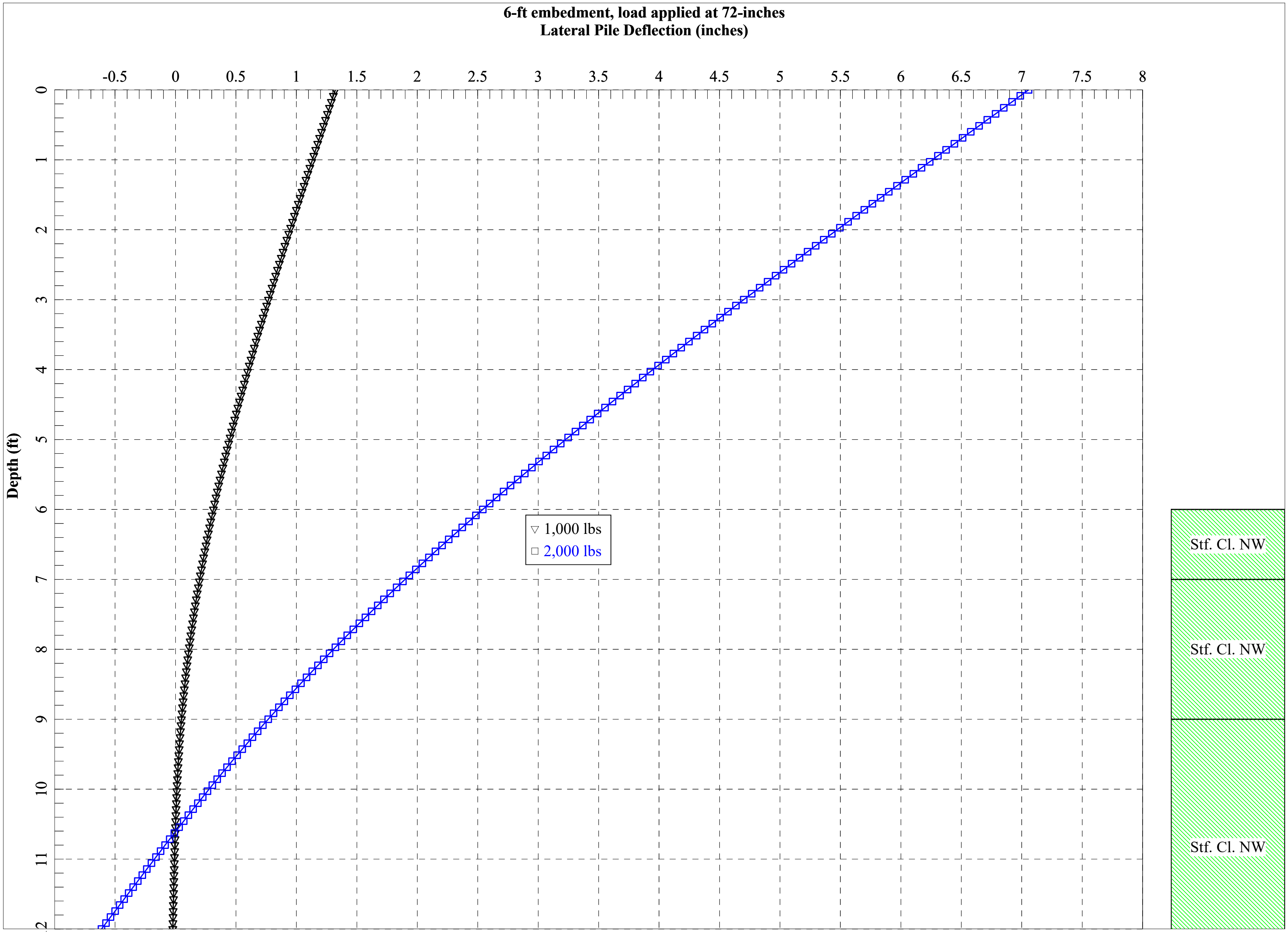

Figure No. 80

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

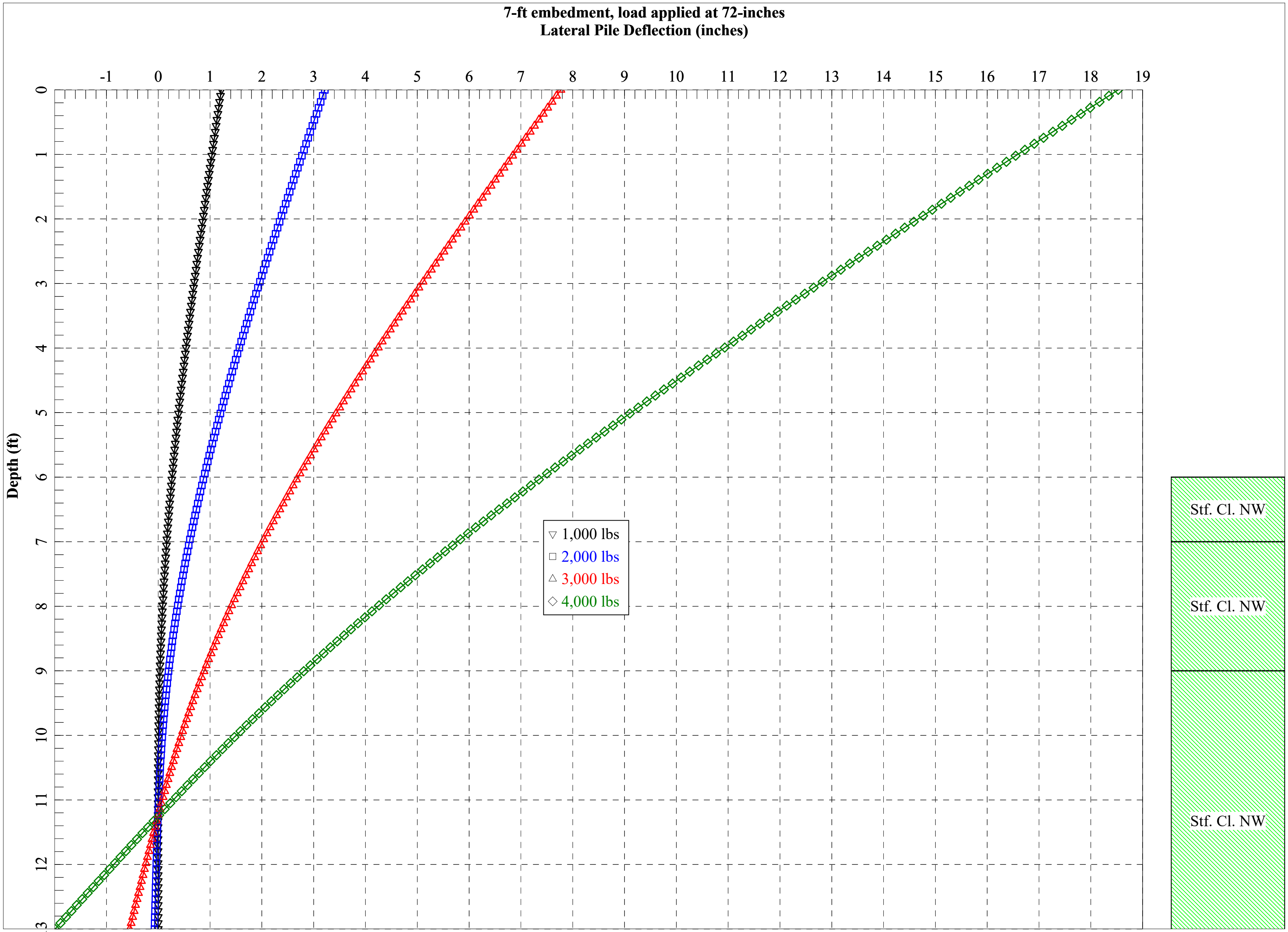

Figure No. 81

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

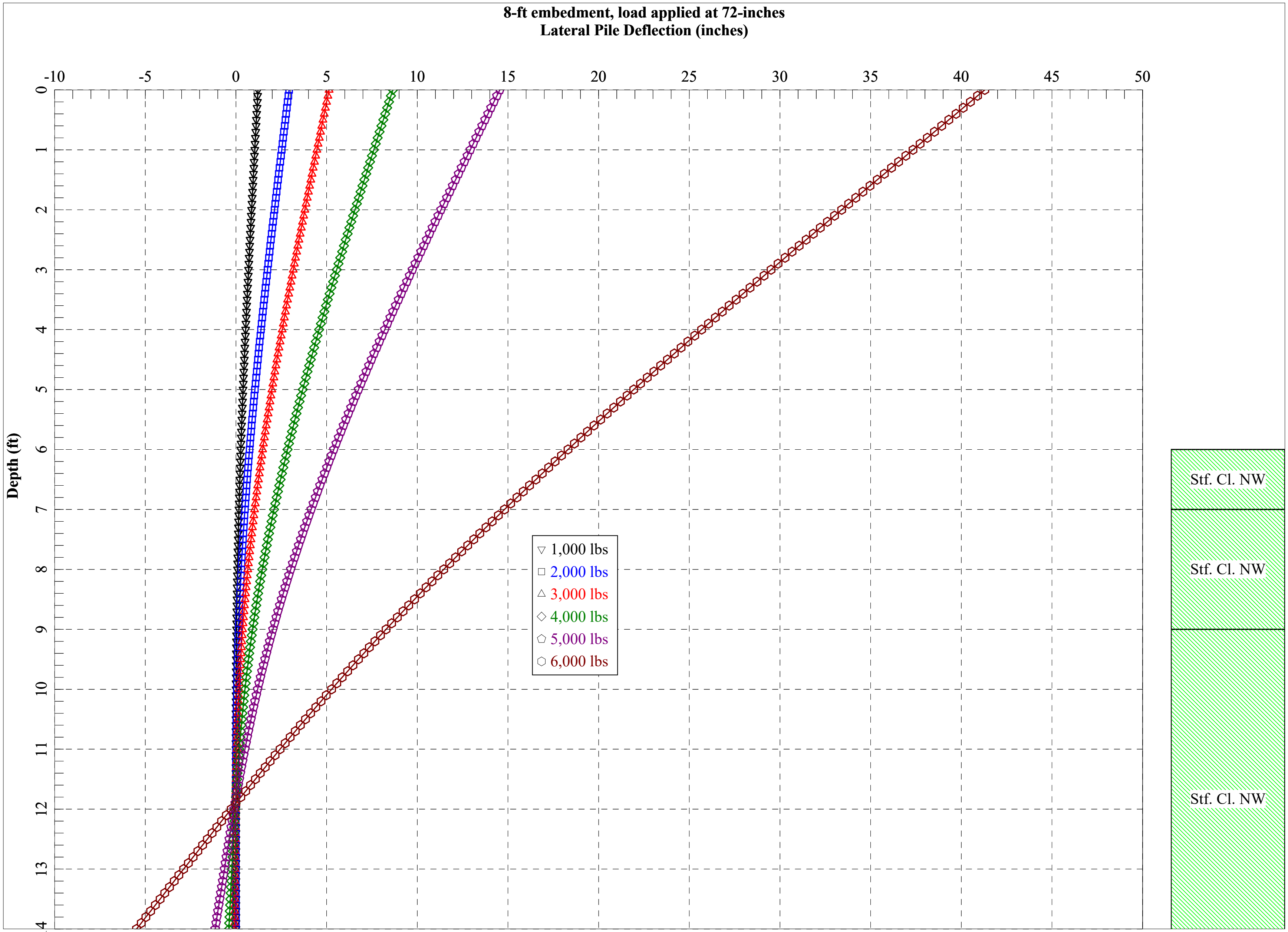

Figure No. 82

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

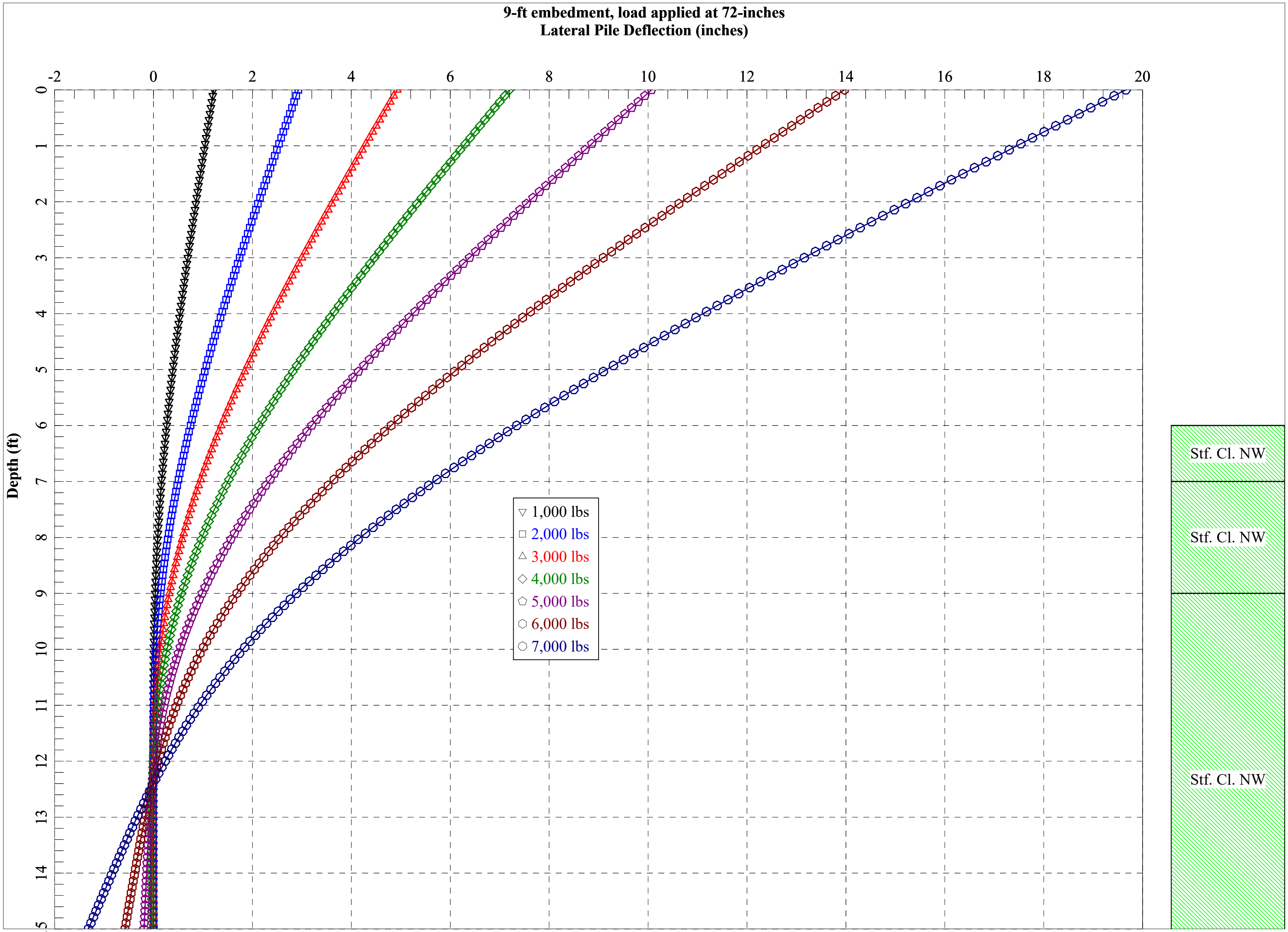

Figure No. 83

W6x9 LATERAL CAPACITY AREA NO. 2
(See Capacity Area Plan, Plate No. 3)

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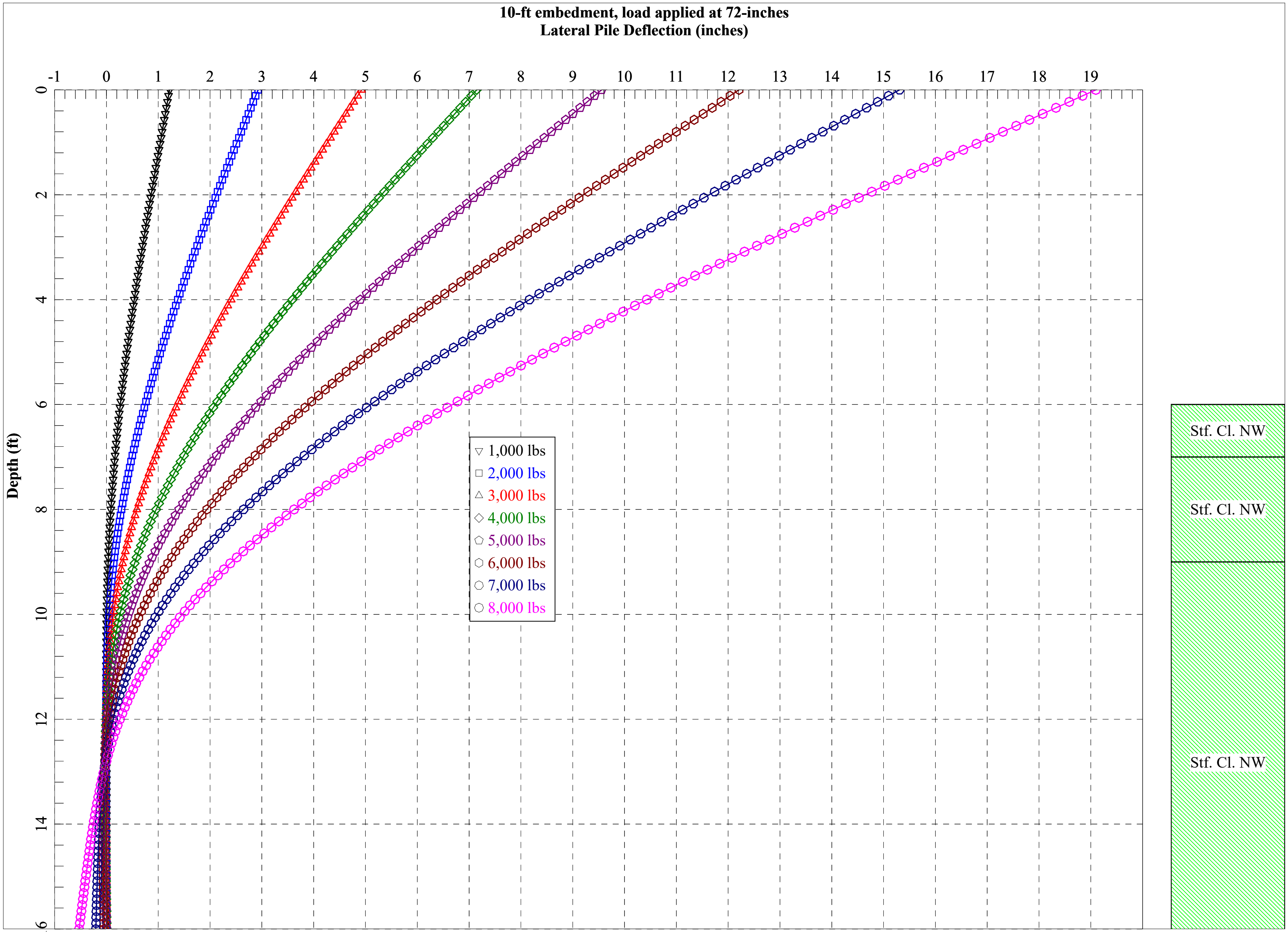


Figure No. 84

APPENDIX F

Design Calculations

Design Chart for Low Volume Aggregate Surfaced Roads
(AASHTO 1993, II-74)
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841

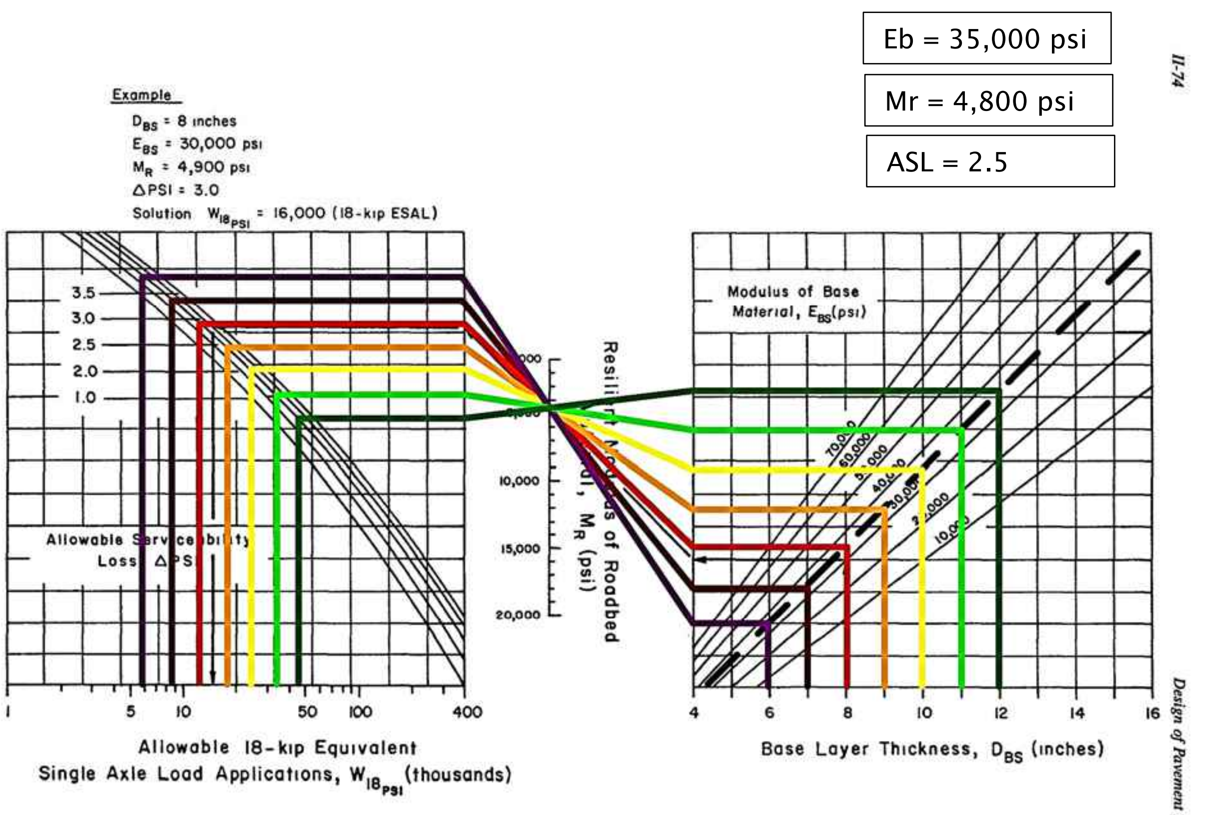
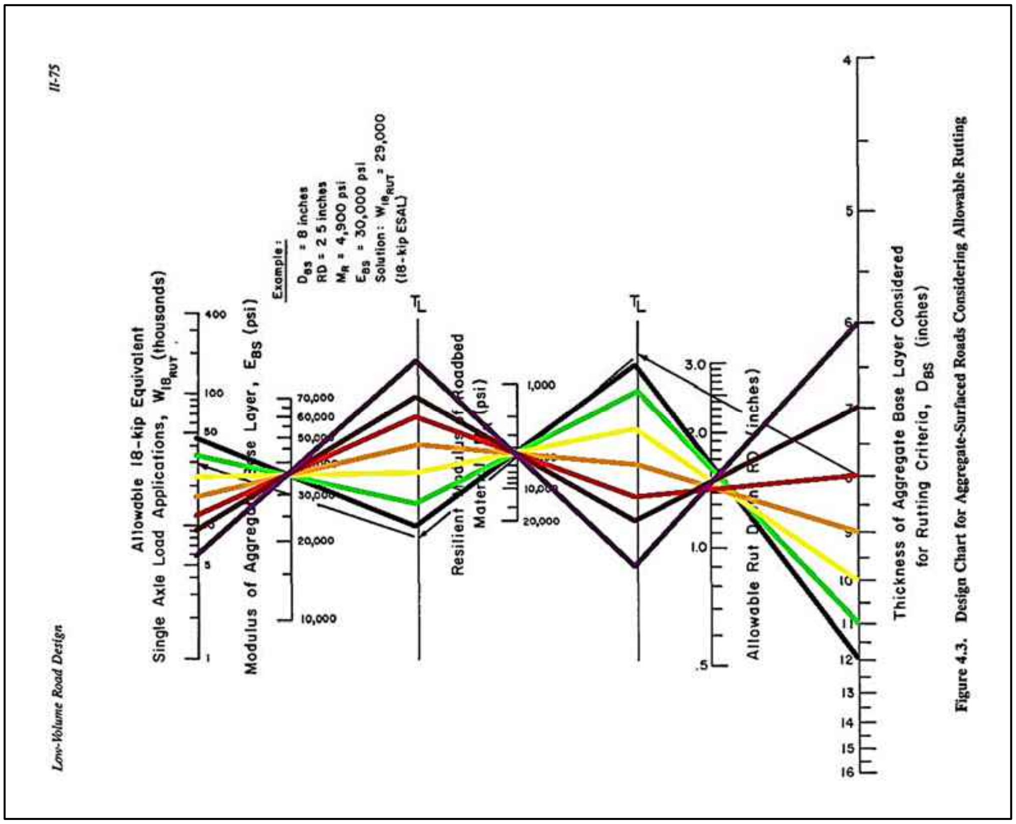


Figure 4.2. Design Chart for Aggregate-Surfaced Roads Considering Allowable Serviceability Loss

Calculation: Allowable ESALs for aggregate base thickness of 6 through 12 inches
Reference: AASHTO pavement design criteria for aggregate-surfaced roads
Date: December 28, 2021
Performed by: Kathryn Crow
Reviewed by: Jeffrey Crow

Permanent Aggregate Surfaced Roads bearing on Native Subgrade (Allowable Serviceability Loss = 2.5)		
Aggregate	Thickness	Allowable 18-kip ESALs
KTC Dense-Graded Aggregate (DGB) or Crushed Stone Base (CSB)	6 inches	6,000
	7 inches	9,000
	8 inches	12,000
	9 inches	18,000
	10 inches	22,000
	11 inches	32,000
	12 inches	47,000

Design Chart for Low Volume Aggregate Surfaced Roads
(AASHTO 1993, II-75)
Proposed Hart 50 MWac Solar Power Plant
Rowletts, Hart County, Kentucky
G2 Project No. 213841



Calculation: Allowable ESALs for aggregate base thickness of 7 through 13 inches
Reference: AASHTO pavement design criteria for aggregate-surfaced roads considering allowable rutting
Date: December 28, 2021
Performed by: Kathryn Crow
Reviewed by: Jeffrey Crow

Permanent Aggregate Surfaced Roads bearing on Native Subgrade			
Aggregate	Thickness	Allowable 18-kip ESALs	Allowable Rut Depth (inches)
KTC Dense-Graded Aggregate (DGB) or Crushed Stone Base (CSB)	6 inches	6,000	1.4
	7 inches	9,000	1.4
	8 inches	12,000	1.4
	9 inches	18,000	1.4
	10 inches	22,000	1.5
	11 inches	32,000	1.6
	12 inches	47,000	1.7