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Geogrid Reinforcement of Piedmont Residual Soil

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16. Abstract

Soil-geosynthetic composites such as those used in Mechanically Stabilized Earth (MSE) retaining walls are experiencing widespread use, particularly in transportation applications. These structures offer substantial economic and, in some cases, performance advantages over traditional options like reinforced concrete walls. Continuing growth in the use of MSE walls, particularly in critical applications such as bridge abutments, is anticipated. The economic advantage of MSE walls is markedly increased if on-site soils are used as the backfill material in the reinforced zone. Ideally, this backfill material is relatively clean (e.g., limited fines content) and cohesionless. Practically, this is not often available on-site. The potential economic benefit of using "lower-quality", on-site material in MSE retaining wall applications is substantial. Using on-site material would eliminate the time and expense associated with identifying and transporting select fill.

An experimental research program investigating soil-geosynthetic interaction was performed at the University of North Carolina at Charlotte. To study this composite behavior, the research program employed a large (7' L by 4' W by 2' D) Pullout Box equipped with state-of-the-art electronic instrumentation and data acquisition. The interaction of two "lower quality" Piedmont residual soils (A-2-4 and A-4) with four, representative, geosynthetic reinforcement materials (rigid geogrid, flexible geogrid, high strength geotextile, and medium strength geotextile) was examined through a series of anchorage strength tests. Through these tests, insight into the load versus deformation behavior of the reinforcing materials embedded in Piedmont residuum was obtained. This report describes the test methodology and presents test results.

17. Key Words Retaining walls, geosynthetics, mechanically stabilized earth walls, MSE	18. Distribution Statement

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SUMMARY

Soil-geosynthetic composites such as those used in Mechanically Stabilized Earth (MSE) retaining walls and embankments are experiencing widespread use, particularly in transportation applications. These structures offer substantial economic and, in some cases, performance advantages over traditional options such as reinforced concrete gravity or cantilever walls. Continued growth in the use of MSE walls, particularly in critical applications such as bridge abutments, is anticipated.

Several methods for designing these structures are currently in use. Two commonly used design guidelines are published by the National Concrete Masonry Association (NCMA, 1996) and the American Association of State Highway and Transportation Officials (AASHTO, 1992 and subsequent interims). The NCMA guidelines are followed primarily within the private sector; the AASHTO specifications are employed in the public sector. The successful application of these or any of the other design guidelines may be distilled to two concepts, (1) proper assessment of the anticipated loading conditions and (2) proper characterization of the load transfer mechanisms between the components of the MSE systems (backfill soil, reinforcing materials, and fascia units). This research project addresses issues related to the second concept. More specifically, the research examines the interactions and load transfer mechanisms between the backfill soil and reinforcing materials.

The economic advantage of MSE walls is markedly increased if on-site soils are used as the backfill material in the reinforced zone. Ideally, this backfill material is relatively clean (e.g., limited fines content) and cohesionless. Practically, this is not often available on-site. The potential economic benefit of using “lower quality”, on-site material in MSE retaining wall applications is substantial. Using on-site material would eliminate the time and expense associated with identifying and transporting select fill.

This research examined the suitability of “lower quality” backfill soil by studying the load transfer mechanisms between representative soils and geosynthetic reinforcing materials. The primary method of studying this interaction was via a series of “pullout” tests as described in subsequent sections of this report.

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CHAPTER 1: INTRODUCTION

1.1. Introduction

Soil-geosynthetic composites such as those used in Mechanically Stabilized Earth (MSE) retaining walls and embankments are experiencing widespread use, particularly in transportation applications. These structures offer substantial economic and, in some cases, performance advantages over traditional options such as reinforced concrete gravity or cantilever walls. Continued growth in the use of MSE walls, particularly in critical applications such as bridge abutments, is anticipated. Other application areas of the MSE concept include foundation reinforcement and in-situ slope reinforcement.

Conventional retaining wall systems, typically constructed of either reinforced concrete or masonry, resist destabilizing forces by either their large mass (gravity-type) or by their geometry and structural stiffness (cantilever-type). The Mechanically Stabilized Earth structures, with layers of reinforcement extending from the wall face into the backfill soil resist destabilizing forces through complex interaction between the backfill soil and the reinforcing elements. Many variations of the MSE concept are currently in use. These include the following (Koerner, 1998):

- facing panels with metal strip reinforcement
- facing panels with metal wire mesh reinforcement
- solid panels with tieback anchors
- anchored gabion walls
- anchored crib walls
- geotextile-reinforced walls
- geogrid-reinforced walls

Several methods for designing these structures are currently in use. Many have been developed by the manufacturers of the various reinforcing materials. Two commonly used design guidelines are published by the National Concrete Masonry Association

(NCMA, 1996) and the American Association of State Highway and Transportation Officials (AASHTO, 1992 and subsequent interims). The NCMA guidelines are followed primarily within the private sector; the AASHTO specifications are employed in the public sector. The successful application of these or any of the other design guidelines may be distilled to two concepts:

1) Proper assessment of the anticipated loading conditions.

2) Proper characterization of the load transfer mechanisms between the components of the MSE systems (backfill soil, reinforcing materials, and fascia units).

This research project addresses issues related to the second concept. More specifically, the research examines the interactions and load transfer mechanisms between the backfill soil and reinforcing materials.

The economic advantage of MSE walls is markedly increased if on-site soils are used as the backfill material in the reinforced zone. Ideally, this backfill material is relatively clean (e.g., limited fines content) and cohesionless. Practically, this is not often available on-site. The potential economic benefit of using "lower quality", on-site material in MSE retaining wall applications is substantial. Using on-site material would eliminate the time and expense associated with identifying and transporting select fill.

This research examined the suitability of "lower quality" backfill soil by studying the load transfer mechanisms between representative soils and geosynthetic reinforcing materials. The primary method of studying this interaction was via a series of "pullout" tests as described in subsequent sections of this report.

1.2. Background

According to the American Society for Testing and Materials (ASTM), a geosynthetic is "a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system" (ASTM D 4439-92a). Geosynthetics are made of a variety of different polymers such as polyester (PET), polypropylene (PP), polyvinyl chloride (PVC), polyethylene (PE), polyamide (PA), and polystyrene (PS). Reasons for using geosynthetics may include economics, construction expediency, and in some cases, functional superiority. In general, geosynthetic products perform five major functions: separation, filtration, drainage, containment, and reinforcement. Brief descriptions of these functions are given below:

- *Separation* – provide barrier to intermingling of dissimilar materials
- *Filtration* - allow cross-plane fluid flow across the plane of the geosynthetic
- *Drainage* – allow in-plane liquid flow within the plane of the geosynthetic
- *Containment* – act as an impervious liquid or vapor barrier
- *Reinforcement* – add tensile strength to a soil mass

Although typically designed and manufactured to perform one of these functions, a particular geosynthetic may actually perform multiple functions simultaneously. Geosynthetics are grouped by material type, manufacturing method, and intended application. These groups include geotextiles, geonets, geomembranes, geosynthetic clay liners, geocomposites, and geogrids. General characteristics of these families are described in the following paragraphs.

A geotextile is a permeable geosynthetic comprised solely of textiles (ASTM D 4439 –92a). Geotextiles are either woven or non-woven. These products resemble heavy fabrics and are typically very flexible and porous. A geotextile may perform one or more of the five primary functions.

A geonet consists of integrally connected sets of parallel ribs overlying similar sets oriented at obtuse angles. This geometric orientation creates void space within the plane of the product that allows easy movement of liquids or gases (ASTM D 4439-92a). A geonet is a specialized geosynthetic product that generally performs the drainage function.

ASTM defines a geomembrane in two ways. First, "a geomembrane is a very low permeability synthetic membrane liner or barrier used with any geotechnical engineering related material so as to control fluid migration in a man-made project, structure, or system" (ASTM D 4833). The second ASTM definition for a geomembrane is "an essentially impermeable geosynthetic composed of one or more synthetic sheets" (ASTM D 4439-92a). The most common geomembranes are extruded polymeric sheets. These products perform the primary function of liquid or vapor barrier.

Geosynthetic Clay Liners are made of a layer of bentonite clay sandwiched between two non-woven geotextiles or a layer of bentonite clay glued to a geomembrane. As with geomembranes, the primary function of a geosynthetic clay liner is as a liquid or vapor barrier.

Geocomposites are formed by the combination of one or more geotextiles, geonets, geogrids, or geomembranes. The functions of products within this family are product specific. Any one of the five primary functions can be targeted.

Koerner (1998) defines a geogrid as a "geosynthetic material consisting of connected parallel sets of tensile ribs with apertures of sufficient size to allow strike-through of surrounding soil, stone, or other geotechnical material." Geogrids are geosynthetics formed with open apertures and grid-like configurations of orthogonal ribs. Extruding and drawing sheets of PE or PP plastic in one or two directions or weaving and knitting PET ribs are methods used to produce geogrids. Geogrids are designed to satisfy the reinforcement function.

The ribs of a geogrid are defined as either longitudinal or transverse. The longitudinal ribs are parallel to the manufactured direction (a.k.a. the machine direction); the transverse ribs are perpendicular to the machine direction. In a geogrid, the intersection of a longitudinal rib and a transverse rib is known as a junction. Junctions can be created in several ways including weaving or knitting. Figure 1.1 shows a section of geogrid in plan view and labels the different grid components.

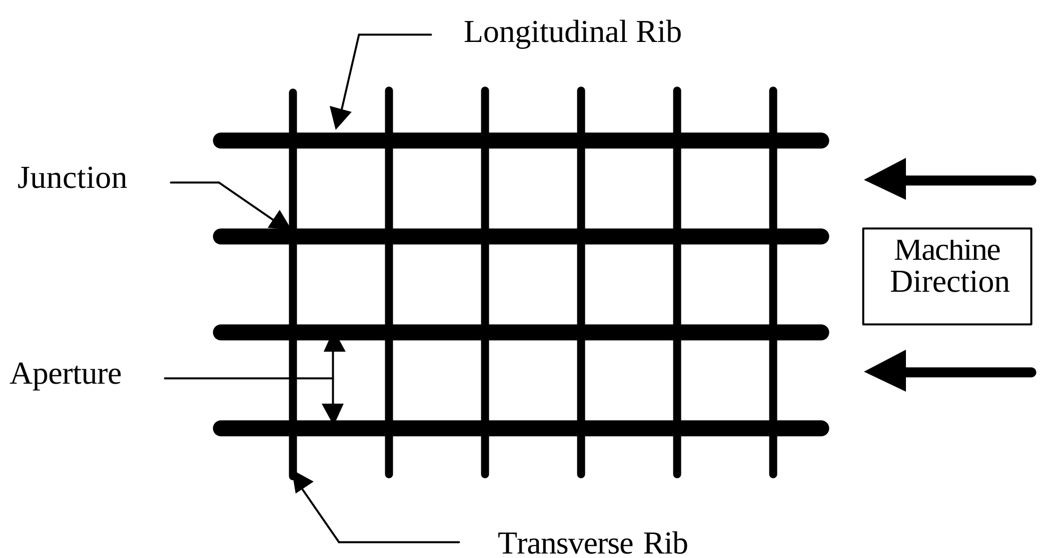


Figure 1.1. Geogrid Component Nomenclature

To provide tensile reinforcement to a soil mass, a geosynthetic must possess adequate tensile strength and have sufficient embedment length to resist pullout. Pullout resistance is derived via interaction with the adjacent, confining soil. This interaction is called the geosynthetic's anchorage strength or pullout resistance. The coefficient of interaction, C_i, is used to relate the pullout resistance of a geosynthetic to the available soil shear strength (NCMA, 1996). C_i is expressed mathematically as follows:

$$C_i = \frac{R_{po}}{2 * L_e * s_n * \tan\phi_i}$$

R_{po}	=	maximum pullout resistance (kN/m)
L_e	=	length of geosynthetic embedded in the pullout box (m)
s_n	=	normal stress acting over the embedded geosynthetic (kN/m^2)
$\tan\phi_i$	=	peak angle of internal friction for the reinforced soil (deg)

For a geotextile, the pullout resistance is mobilized via the shear strength along the top and bottom surfaces. The pullout resistance of a geogrid is mobilized by two mechanisms: shear strength along the top and bottom surfaces of the longitudinal and transverse ribs and the passive resistance along the front of the transverse ribs (Figure 1.2). In the second mechanism, transverse ribs resist pullout in a manner analogous to the bearing capacity of a shallow foundation. The transverse rib's bearing resistance is developed by the passive resistance of the soil in front of the rib.

Junction strength is the ultimate strength at which a junction fails. A failure occurs when the transverse rib shifts relative to the longitudinal rib at the failed junction. This shift decreases the distance between adjacent transverse ribs and closes the apertures.

When a junction fails, the transverse rib is no longer able to effectively mobilize pullout resistance.

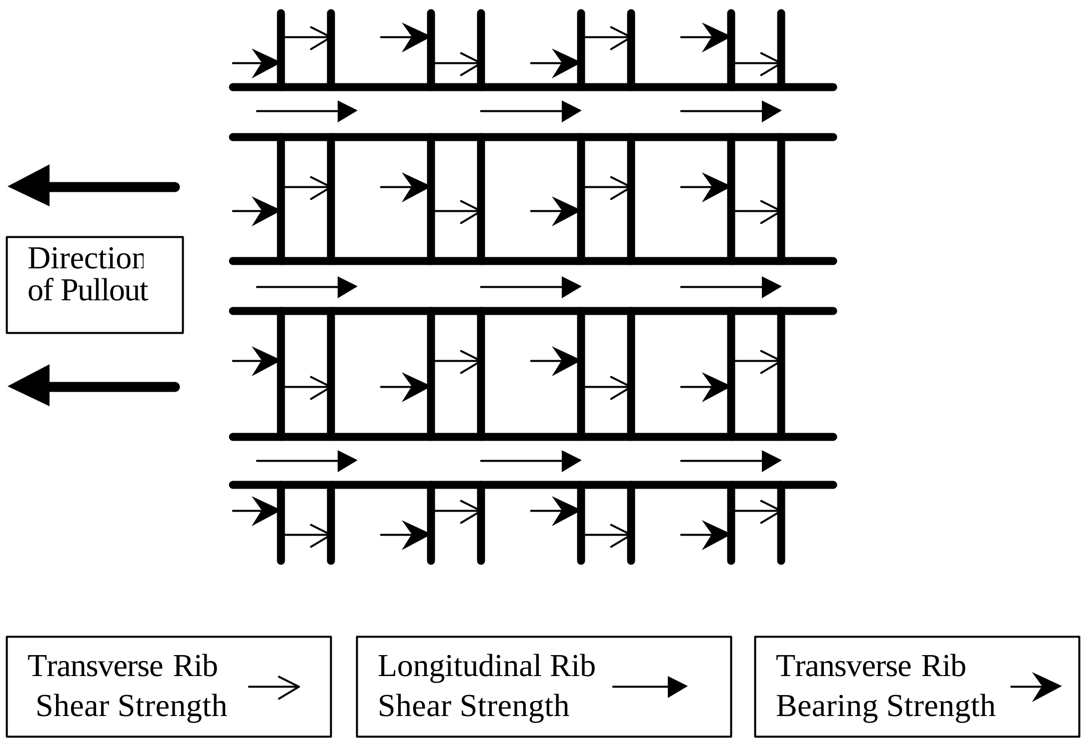


Figure 1.2. Geogrid Pullout Resistance Mechanisms

CHAPTER 2: PROJECT OVERVIEW

2.1. Objectives

The main objective of this research program was to assess the suitability of various soils as backfill material in reinforced soil applications by testing “lower quality” Piedmont residuum in combination with different types of geosynthetic reinforcement materials.

In order to address this objective, this study investigated the load transfer characteristics between two Piedmont residual soils and four geosynthetic reinforcing materials. The residual soils were classified as A-2-4 and A-4 using the AASHTO system. According to the NCDOT Standard Specifications for Roads and Structures, Section 1016-3 (NCDOT, 1995), these soils are classified as Class II, Type 2 materials. The geosynthetics selected included a flexible, woven polyester geogrid (Husker, Inc., Fortrac 55/30-20), medium and high strength, polyester geotextiles (TC Mirafi, Geolon HS800 and HS1150), and a rigid, biaxial, polypropylene geogrid (Enkagrid MAX 20). These specific soil types and geosynthetic materials were selected in conjunction with NCDOT personnel. Pullout resistance was assessed experimentally using a large-scale pullout box (7’ Long by 4’ Wide by 2’ Deep) equipped with state-of-the-art electronic instrumentation and data acquisition (Figures 1 and 2). A total of twenty-four tests were performed and evaluated.

Results of this experimental program can be used to guide decisions regarding the specification of both constituent materials, the geosynthetic reinforcement and the backfill soil. By selecting a variety of geosynthetic products, direct comparisons of performance are possible. This information can be used to develop appropriate material specifications. With respect to the backfill soil, by focusing the experimental program on “lower quality” but more prevalent soil types, the specifications for backfill soil in reinforcement applications can be more clearly defined.

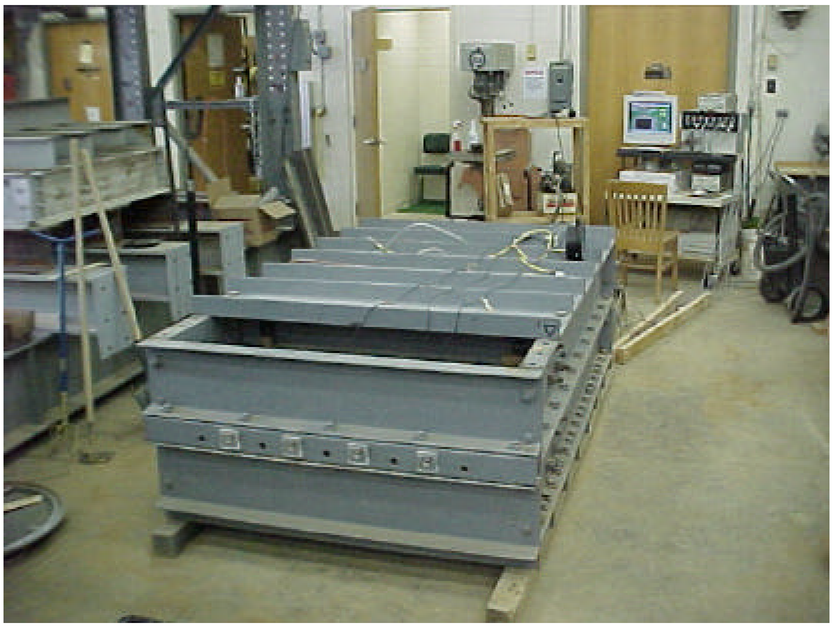


Figure 2.1. Pullout Box (rear view) and Data Acquisition System

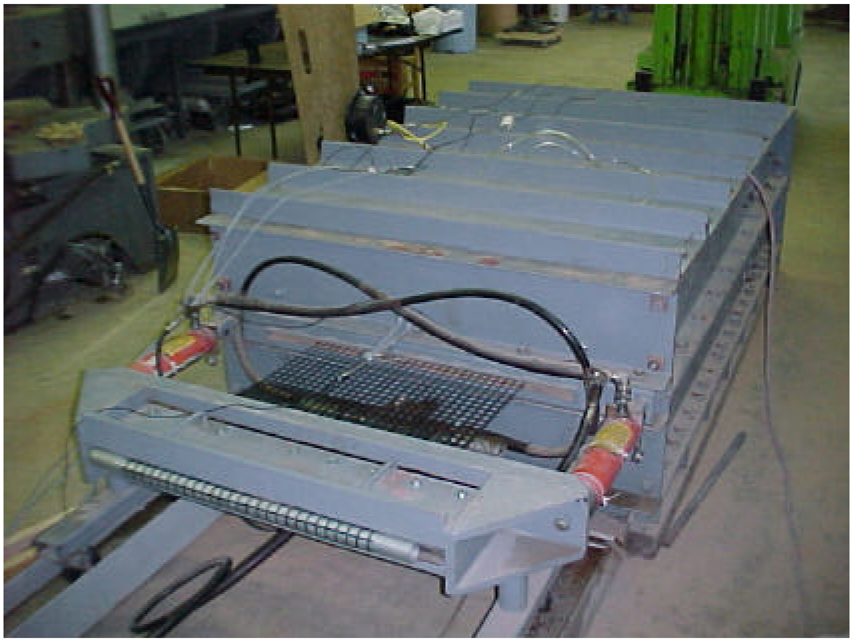


Figure 2.2. Pullout Box (front view)

2.2. Research Methodology & Tasks

The scope of work for this research project is summarized in Table 1. The activities were divided into three phases, (1) Literature Review and Experimental Preliminaries, (2) Laboratory Testing, (3) Data Analysis and Interpretation. The tasks within each phase are listed in Table 1 and are briefly described in the following paragraphs.

Table 2.1. Scope of Work

Phase I	Task 1	Literature review.
	Task 2	Selection and acquisition of geosynthetic reinforcing materials.
	Task 3	Acquisition and processing of soils.

Phase II	Task 4	Material characterization for geosynthetic products.
	Task 5	Material characterization tests for soils.
	Task 6	Pullout tests.
Phase III	Task 7	Written documentation of Phase 1 and Phase 2 activities.
	Task 8	Interpretation of test results.
	Task 9	Written report addressing specifications for selection and placement of backfill soil used in mechanically stabilized earth structures

Phase I: Literature Review and Experimental Preliminaries

Task 1: Literature Review

A thorough search of relevant databases was made using the resources available through the Internet and UNC Charlotte's Atkins Library facilities.

Task 2: Selection and acquisition of geosynthetic reinforcing materials

As mentioned earlier, the selection of the geosynthetic reinforcing materials was made in collaboration with appropriate NCDOT personnel. Four geosynthetic products were used in the test program. Together, these materials cover the following categories:

- flexible, polyester (PET) geogrid
- stiff, high density polyethylene (HDPE) geogrid
- high-strength, woven geotextile

Task 3: Acquisition and processing of soils

Again, the selection of the appropriate soils was made in collaboration with appropriate NCDOT personnel. Two soil types were selected. Both soils were Piedmont residuum.

The soils were classified in the AASHTO system as A-2-4 and A-4 with appropriate PI restrictions to satisfy the Class II – Type 2 specification as described in the NCDOT Standard Specifications for Roads and Structures.

Phase II: Laboratory Testing

Task 4: Material characterization tests for geosynthetic products

Material characterization tests were performed on the actual geosynthetic products used to verify data provided by the manufacturers, particularly the Wide Width Tensile (WWT) strength (ASTM D 4595).

Task 5: Material characterization tests for soils

Material characterization tests for the selected soils included the following tests:

- Grain size distribution
- Atterberg Limits
- Specific gravity of soil solids
- Standard Proctor test
- Modified Proctor test
- Sand cone density test

Task 6: Pullout tests

Twenty-five pullout tests were performed as summarized in the text matrix shown in Table 2. Thirteen tests were performed using the A-2-4 soil (Soil #1), ten tests were performed using the A-4 soil (Soil #2) and two tests were "empty box" tests performed to calibrated the pullout system. Tests for a given soil- geosynthetic combination were performed at different confining pressures to simulate reinforcement at different depths below the top of a retaining wall. The work was performed by nine advanced undergraduate students and one graduate student under the direction of the PI and laboratory support personnel. This is a substantially larger volume of work than the twelve tests originally proposed by the PI.

Table 2.2. Pullout Test Matrix for Each Soil Type

Reinforcement Type	Soil Type A-2-4 (Soil #1)	 A-4 (Soil #2)
Flexible geogrid	1, 2, 3, 4, 22, 23	11, 12, 20, 21
Low strength geotextile	7, 24	15, 18
High strength geotextile	5, 6	16, 17
Rigid geogrid	8, 9, 10	13, 14

Note: Tests 19 and 25 were "empty box" tests**.**

Phase III: Data Analysis and Interpretation

Task 7: Written documentation of Phase 1 and Phase 2 activities

The results of Tasks 1-6 have been compiled and are part of this written report.

Task 8: Interpretation of test results

Interpretation of test results is included in this written report.

Task 9: Written report addressing specifications for selection and placement of backfill soil used in mechanically stabilized earth structures

This task is partially addressed in this draft report and will be finalized with input from the NCDOT Technical Advisory Committee.

2.3. Significance of Work

The significance of this project can be viewed in two ways. First, the potential economic benefit of using "lower quality", on-site material in MSE retaining wall and embankment applications is substantial. Using on-site material would eliminate the time and expense associated with identifying and transporting select fill. This research program addressed this issue directly by performing tests on representative samples of “lower quality” soil. Second, this work provides direct performance comparisons of a variety of geosynthetic products. In an area where most information comes from the manufacturers of the products being sold, this independent source of information is vitally important to the engineers making important design decisions.

CHAPTER 3: LITERATURE REVIEW

3.1. Background

The use of tensile inclusions in soil structures is not a new idea. The first application dates back several thousand years to the construction of religious structures in ancient Babylonia (Sprague, 1998). However, over the last three decades the use of geosynthetics in soil reinforcement has increased dramatically. The manufacture, sales, and installation of geosynthetic materials has grown into a multibillion dollar per year industry (Koerner and Soong, 1997).

Henri Vidal, a French architect, pioneered modern earth reinforcement techniques nearly four decades ago by incorporating resisting elements (steel strips) into a soil mass. The steel strips complemented the soil's compressive strength and acted in composite fashion to improve the mechanical properties of the soil mass. The first Reinforced Earth wall using Henri Vidal's patented system was constructed along California State Highway 39 in 1972 (Sprague, 1998).

3.2. Geosynthetic Pullout Resistance

Planar, geosynthetic reinforcing products are similar to Henri Vidal's steel strips in that they provide tensile strength to a soil mass. However, the mechanisms by which geosynthetics function in a composite manner with the soil differs from those of steel strips. The pullout resistance of a sheet type geosynthetic (such as a geotextile) is developed via frictional resitance along the upper and lower surfaces. The pullout resistance of a geogrid is mobilized by two mechanisms: frictional resistance along the top and bottom of the longitudinal and transverse ribs and the passive resistance along the front of the transverse ribs.

A good, basic description of pullout testing with respect to geogrid materials is provided by Koerner (1998). As mentioned in this description, the reason for performing these tests is to determine the anchorage strength of a reinforcing material for a particular soil type and confining pressure. The test is described by Koerner as, “probably the most sophisticated and expensive of all geosynthetic tests at this time”. Papers by Fannin et al (1993), Lo (1998), and Madhav et al (1998) are examples of both experimental and theoretical studies that employed pull out testing of geosynthetic materials. Related research has been and continues to be published in the conference proceedings of the bi-annual *Geosynthetics* conferences and the peer-reviewed journal, *Geosythetics International.*

Geosynthetic reinforcement behavior under pullout conditions is complex and not entirely understood (Zettler et al, 1998). Pullout force is recorded as a function of the horizontal displacement of a geosynthetic (Koutsourais et al, 1998). Each geosynthetic exhibits a value of displacement that corresponds to a maximum value of mobilized resistance (Ochiai et al, 1992). “Pullout resistance may be a combination of sliding, rolling interlocking of soil particles, geosynthetic surfaces, and shear strain within the geosynthetic specimen” (Koutsourais et al, 1998). Other factors affecting pullout resistance are the reinforced soil properties: angle of internal friction, cohesion, and density (Fannin and Raju, 1993).

According to Alfaro et al (1995), pullout resistance of geogrids is developed via two mechanisms: frictional resistance of the longitudinal members, and bearing resistance on the transverse members. The bearing resistance is developed by passive resistance against the transverse elements of open structure geogrids (NCMA, 1996). "Passive resistance occurs through the development of bearing type stresses on transverse reinforcement surfaces normal to the direction of soil reinforcement relative movement" (FHWA, 1990). There are a number of factors that contribute to the development of the passive resistance. These factors include the following: roughness of the surface (skin friction), normal effective stress, grid opening dimensions, thickness of the transverse members, and elongation characteristics of the reinforcement (FHWA, 1990).

3.3. Coefficient of Interaction

The development of adequate pullout resistance is essential to the performance of reinforced soil structures and is governed by soil-geosynthetic interaction (Fannin and Raju, 1993). The coefficient of interaction, C_i, relates pullout resistance of the geosynthetic to the available shear strength of the reinforced soil (NCMA, 1990). C_i measures a reinforcing product's efficiency in transferring stresses from adjacent soil particles to the geosynthetic reinforcement (Koutsourais et al, 1998). The coefficient of interaction is used in the design of MSE walls to determine required reinforcement embedment lengths. The embedment length is the portion of the geosynthetic beyond the anticipated failure plane required to prevent pullout of the reinforcing component. "It is important to note that C_i will vary between geosynthetic products and may change with magnitude of normal pressure applied to samples of the geosynthetic" (NCMA, 1996).

3.4. Geogrid Junctions

"It is considered that pullout resistance concentrates and acts on each of the geogrid junctions" (Ochiai et al, 1992). The pullout resistance of each rib at right angles to the direction of pulling is transferred to the grid junctions. "Passive stresses due to the interlocking of soil particles along the cross rib would definitely influence the measured resistance" (Zettler et al, 1998). However, resistance developed on each grid junction has a greater magnitude than that mobilized on each rib (Ochiai et al, 1992). Because geogrid deformation occurs, the pullout resistance is mobilized on both the grid junctions and ribs. "An Elliptic slip field is formed in front of each grid junction and expanded with increasing the displacement level of grid junction. When the displacement reaches some large value, adjacent slip fields interact each other so the pullout resistance acting on each junction decreases and reaches a residual state" (Ochiai et al, 1992). The slip field effect is evidence of the pullout resistance developed by geogrid junctions.

However, Cowell and Sprague provided a different conclusion based on extensive pullout testing. "They concluded junction strength of geogrids and the contribution to pullout resistance contributed from transverse ribs does not have a significant affect on pullout performance" (Sprague, 1998).

The widely varying results of different research projects illustrate the importance of documenting the type of soil and geosynthetic used for the research. Results of geosynthetic pullout tests cannot be generalized. The type of soil used clearly affects the pullout resistance of a geosynthetic. Also, the different geosynthetic reinforcing materials exhibit different pullout behavior because of the unique material properties of each geosynthetic.

CHAPTER 4: GEOSYNTHETIC REINFORCING MATERIALS

4.1 Introduction

Four geosynthetic reinforcing materials were selected, with at least one product from each of the following categories:

- flexible, polyester (PET) geogrid
- stiff, high density polyethylene (HDPE) geogrid
- high-strength, woven geotextile

The geosynthetics selected included a flexible, woven polyester geogrid (Husker, Inc., Fortrac 55/30-20), medium and high strength, polyester geotextiles (TC Mirafi, Geolon HS800 and HS1150), and a rigid, biaxial, polypropylene geogrid (Enkagrid MAX 20). These specific soil types and geosynthetic materials were selected in conjunction with NCDOT personnel. Material characterization tests were performed on the actual geosynthetic products used to verify data provided by the manufacturers. Descriptions of the four geosynthetic reinforcing materials and their material properties are contained in the following sections.

4.2 Flexible Geogrid

The flexible geogrid used in this research was Fortrac 55/30-20 donated by Huesker Inc. of Charlotte, North Carolina, a subsidiary of Huesker Synthetic GmbH & Co. of Germany. Fortrac 55/30-20 is manufactured from high modulus, low creep polyester yarns protected by a polymeric coating. The longitudinal and transverse ribs are woven on a Huesker developed loom and coated before being prepared for shipment.

Huesker performs extensive quality control testing on all their products. They publish a data sheet of the physical properties of Fortrac geogrids based on the minimum average roll values (MARV) (Table 4.1). The MARV is the minimum average value of a representative number of tests made on selected rolls of the lot in question (Koerner, 1998).

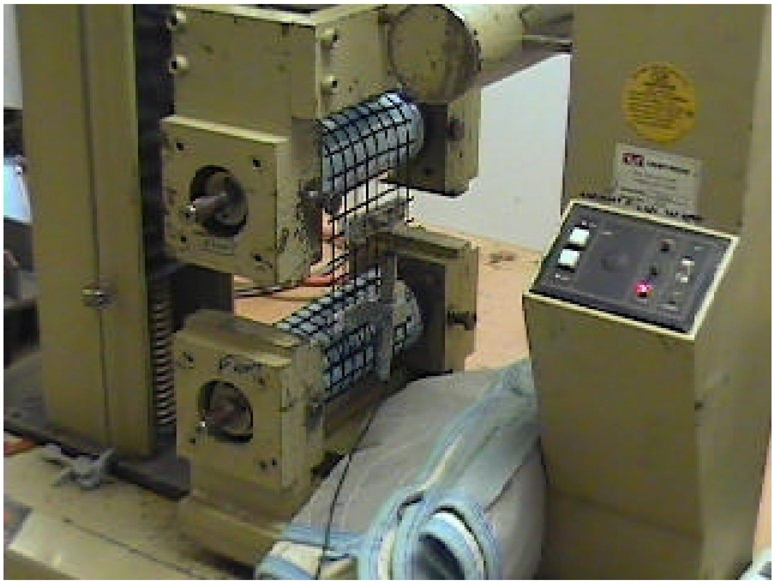
Table 4.1. Physical and Mechanical Properties of Fortrac 55/30-20 (Huesker)

Physical / Mechanical Property	Value
Mass per Unit Area	333g/m^2
Aperture Size (mm)	20
% Open Area	70+
Wide Width Tensile Strength (lb/ft) (ASTM D 4595) @ Ultimate Machine Direction @ Ultimate Cross-Machine Direction @ 5% Strain Machine Direction	 3700 (54 kN/m) 2020 (29.5 kN/m) 1500 (21.9 kN/m)
Elongation at Break, % (ASTM D 4595)	11.0

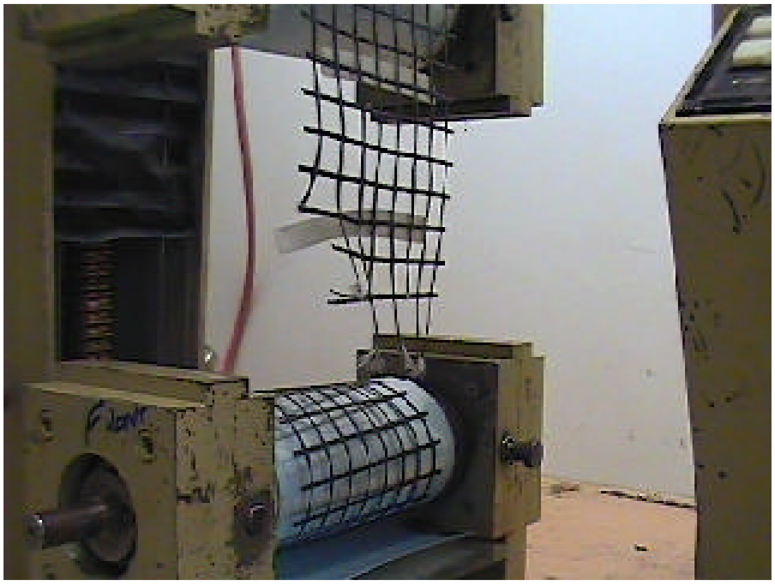
A 3.7 m wide by 200 m long roll of the geogrid sample was delivered to the Geotechnical Engineering Research Laboratory at UNC-Charlotte. The sample was partially unrolled and the first 2 m trimmed and discarded. The sample was unrolled further and the test specimens were cut from the roll. These specimens were checked for manufacturing defects, shipping damage, or other irregularities.

To test the geogrid's ultimate and junction strengths, a 1.5 m long by 3.7 m wide sample was taken to Geosyntec Consultants in Atlanta, Georgia. The geogrid's ultimate strength was determined in general accordance with ASTM D 4595. The ultimate strength of the material was tested using roller grips designed and constructed by

Geosyntec Consultants. The sample was loaded into the roller grips and drawn in tension until rupture (Figures 4.1 and 4.2).



Figures 4.1. Pretest WWT Sample



Figures 4.2. Post-Test WWT Sample

A total of 6 WWT tests were performed with the following ultimate strength results:

Test 1. 3679.0 lb/ft (53.7 kN/m)

Test 2. 4212.0 lb/ft (61.5 kN/m)

Test 3. 4070.3 lb/ft (59.4 kN/m)

Test 4. 3785.6 lb/ft (55.2 kN/m)

Test 5. 3846.7 lb/ft (56.1 kN/m)

Test 6. 4037.8 lb/ft (58.9 kN/m)

The average ultimate strength from the six WWT tests performed was 57.5 kN/m (3938.6 lb/ft) with a standard deviation of 2.9 (200.5). The tested average ultimate strength is 106% of the Huesker reported ultimate strength of (54 kN/m).

The single rib clamps used for junction strength testing were also designed and constructed at Geosyntec Consultants. A sample was cut to form a “cross” of one longitudinal rib and one transverse rib (Figure 4.3). The sample was loaded into the clamps by compressing the lower portion of the longitudinal rib in the lower clamps and compressing the transverse rib in the upper clamps (Figure 4.4). Each test specimen was drawn in tension until the transverse rib was pulled from the longitudinal rib. The load required to break the transverse rib free from the longitudinal rib is the junction strength.

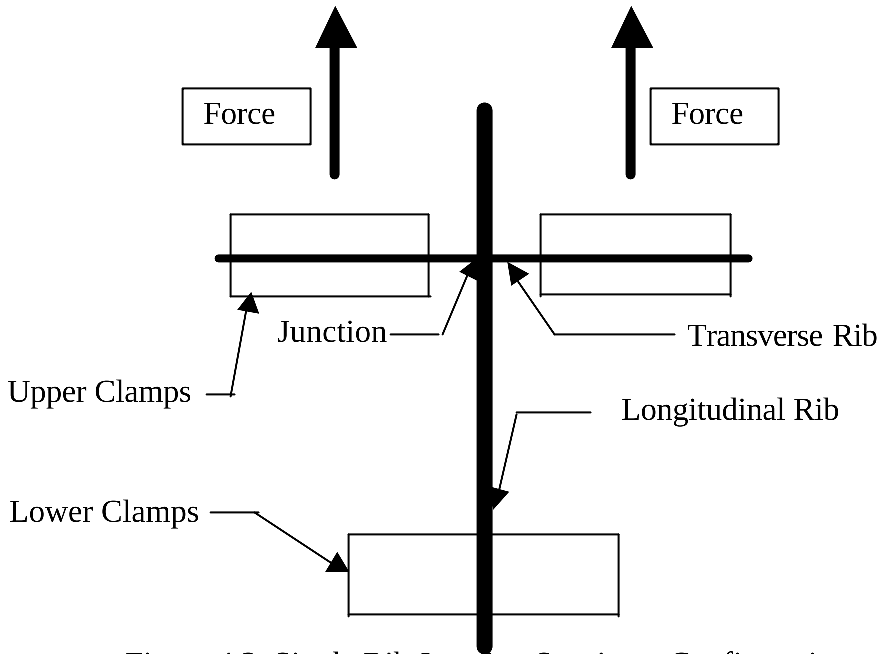


Figure 4.3. Single Rib Junction Specimen Configuration

Five specimens were tested with the following junction strength results:

Test 1. 22.2 lb (98.4 N)

Test 2. 23.9 lb (106.4 N)

Test 3. 20.3 lb (90.3 N)

Test 4. 22.9 lb (102.1 N)

Test 5. 23.4 lb (104.1 N)

The average junction strength from the five tests performed was 100.3 N (22.5 lb).

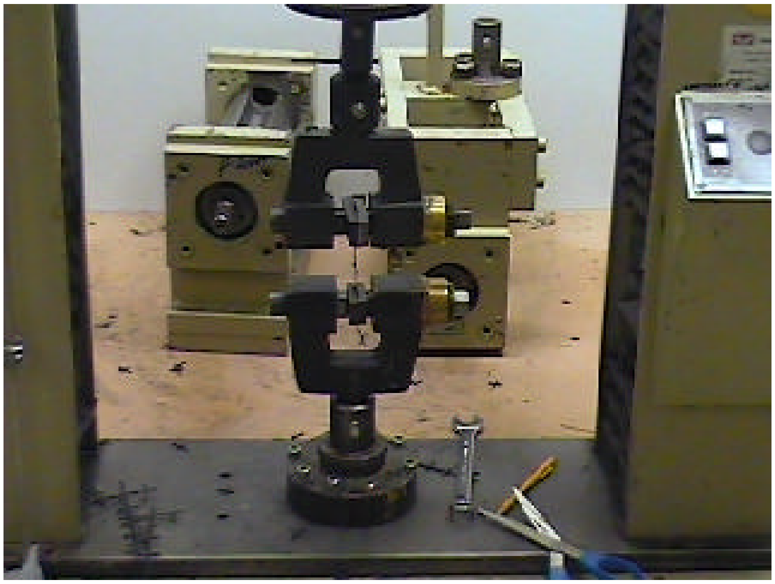


Figure 4.4. Single Rib Junction Clamps

Pullout test overall specimen length, with embedment and free board overhang from the front opening of the box, was approximately 2.75 m. All geogrid test specimens were 0.6 m wide with an embedment length of 0.6 m. The outer edge of the longitudinal ribs and the length of the longitudinal ribs defined these dimensions, respectively.

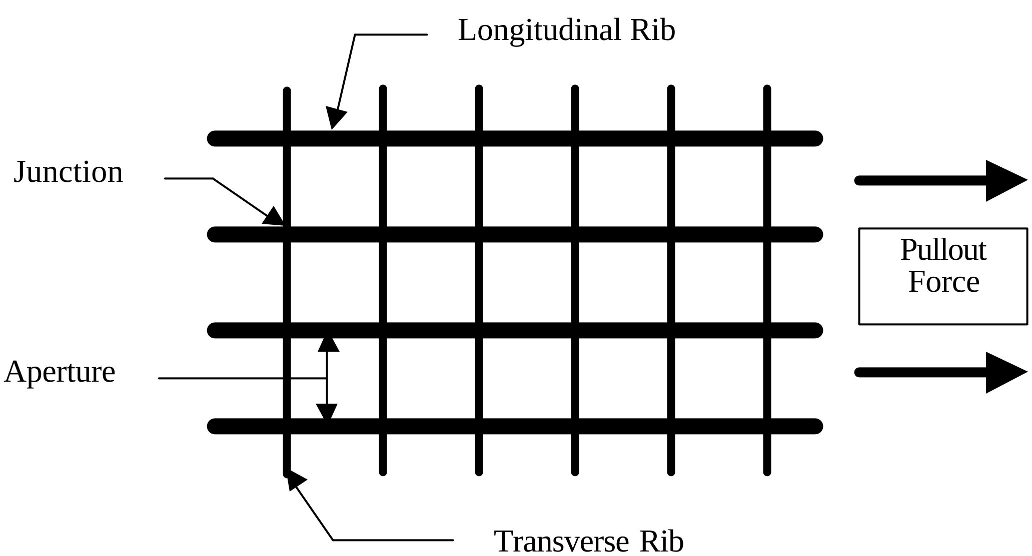


Figure 4.5. Flexible Geogrid Test Orientation

4.3 Rigid Geogrid

The rigid geogrid used in this research was Enkagrid MAX 20 donated by TC Mirafi. According to the manufacturer, Enkagrid MAX 20 is a rigid biaxial geogrid composed of highly oriented, extruded polypropylene strips. The polypropylene strips are bonded using laser technology that precisely controls the production process creating consistently rigid junctions. Enkagrid MAX 20 is inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids.

TC Mirafi has tested Enkagrid MAX 20, and the results are as shown in Table 4.2. As with the Huesker geogrid, test results are presented as Minimum Average Roll Values (MARV).

Table 4.2. Physical and Mechanical Properties of Enkagrid MAX 20 (TC Mirafi)

Physical / Mechanical Property	Test Method	Unit	Minimum Average Roll Value	
			MD	CD
Tensile Strength (at ultimate)	ASTM D 4595	kN/m (lbs/ft)	20.5 (1400)	20.5 (1400)
Elongation (at ultimate)	ASTM D 4595	%	10	10
Tensile Strength (at 2% strain)	ASTM D 4595	kN/m (lbs/ft)	8 (548)	8 (548)
Tensile Strength (at 5% strain)	ASTM D 4595	kN/m (lbs/ft)	15 (1028)	15 (1028)
Tensile Modulus (at 2% strain)	ASTM D 4595	kN/m (lbs/ft)	400 (27400)	400 (27400)
Tensile Modulus (at 5% strain)	ASTM D 4595	kN/m (lbs/ft)	300 (20560)	300 (20560)
Junction Strength per Rib, J_{rib}	GRI-GG2	N (lbs)	534 (120)	400 (90)
Ultimate Junction Strength, J_{grid}	GRI-GG2	kN/m (lbs/ft)	12.2 (840)	9.2 (630)
Flexural Rigidity	ASTM D 1388a	mg-cm	450000	
Percent Open Area	COE-22125-86	%	75	
UV Resistance (at 500 hours)	ASTM D 4355	% strength retained	70	

Notes: MD = machine direction
CD = cross-machine direction

4.4 Geotextiles

Two geotextile reinforcing materials, one medium strength and one high strength, were used in this research. The medium strength product was Geolon HS800; the high strength product was Geolon HS1150. Both products are manufactured by TC Mirafi and were donated to the PI for this research program. According to the manufacturer, both Geolon HS800 and HS1150 are composed of high tenacity polyester multifilament yarns which are woven into a stable network such that the yarns retain their relative position. Both products are inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids.

TC Mirafi has tested Geolon HS800 and HS1150, and the results are as shown in Table 4.3 and 4.4, respectively. As was true for the other geosynthetic products, test results are presented as Minimum Average Roll Values (MARV).

Table 4.3. Physical and Mechanical Properties of Geolon HS800 (TC Mirafi)

Physical / Mechanical Property	Test Method	Unit	Minimum Average Roll Value
			Machine Direction
Tensile Strength (at ultimate)	ASTM D 4595	kN/m (lbs/ft)	140.1 (9600)
Tensile Strength (at 5% strain)	ASTM D 4595	kN/m (lbs/ft)	52.5 (3600)
Tensile Strength (at 10% strain)	ASTM D 4595	kN/m (lbs/ft)	131.3 (9000)
Creep Reduced Strength	ASTM D 5262	kN/m (lbs/ft)	84.0 (5760)
Long Term Design Strength	GRI GT-7	kN/m (lbs/ft)	66.4 (4553)
Factory Seam Strength	ASTM D 4884	kN/m (lbs/ft)	35.0 (2400)
Permittivity	ASTM D 4491	sec^{-1}	0.20
Apparent Opening Size (AOS)	ASTM D 4751	mm (U.S. Sieve)	0.850 (20)
UV Resistance (at 500 hours)	ASTM D 4355	% strength retained	70

Table 4.4. Physical and Mechanical Properties of Geolon HS1150 (TC Mirafi)

Physical / Mechanical Property	Test Method	Unit	Minimum Average Roll Value
			Machine Direction
Tensile Strength (at ultimate)	ASTM D 4595	kN/m (lbs/ft)	201.4 (13800)
Tensile Strength (at 5% strain)	ASTM D 4595	kN/m (lbs/ft)	70.0 (4800)
Tensile Strength (at 10% strain)	ASTM D 4595	kN/m (lbs/ft)	175.1 (12000)
Creep Reduced Strength	ASTM D 5262	kN/m (lbs/ft)	120.8 (8280)
Long Term Design Strength	GRI GT-7	kN/m (lbs/ft)	95.5 (6545)
Factory Seam Strength	ASTM D 4884	kN/m (lbs/ft)	35.0 (2400)
Permittivity	ASTM D 4491	sec^{-1}	0.32
Apparent Opening Size (AOS)	ASTM D 4751	mm (U.S. Sieve)	0.600 (30)
UV Resistance (at 500 hours)	ASTM D 4355	% strength retained	70

A second trip to GeoSyntec Consultants in Atlanta, Georgia was made to verify the manufacturer-provided values for tensile strength (ASTM D 4595) of the rigid geogrid and the two geotextiles. In all cases, the test values were within ten percent of the manufacturer-provided values. Photographs of the rigid geogrid being tested are provided in Figures 4.6 and 4.7; photographs of the geotextile being tested are shown in Figures 4.8 and 4.9.

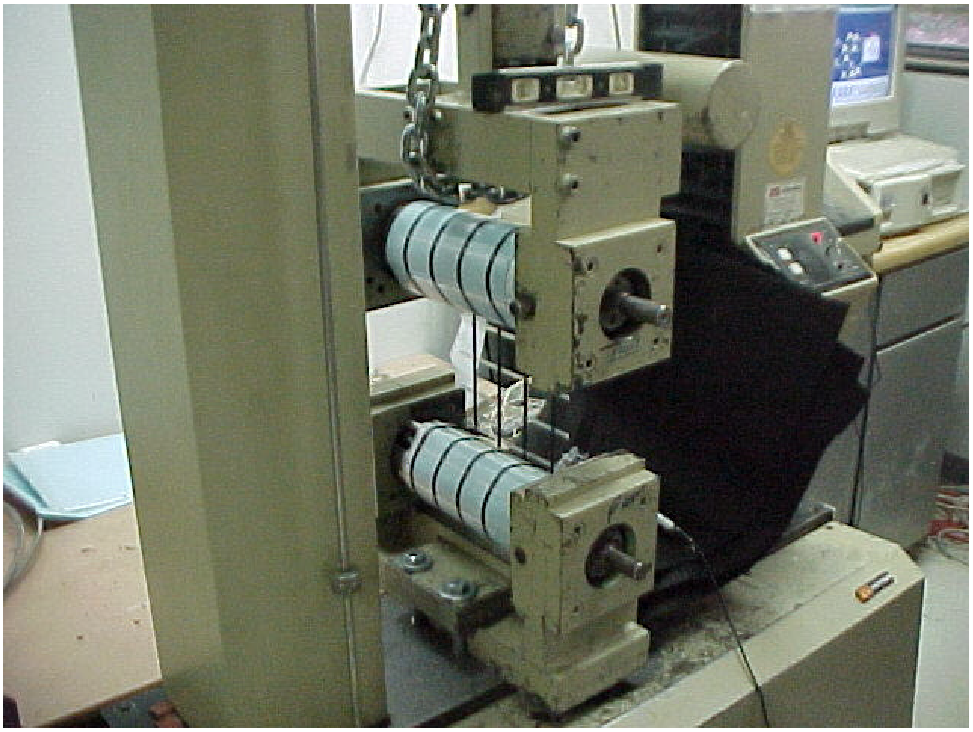


Figure 4.6. Wide Width Tensile Test on Enkagrid MAX 20 (before loading)

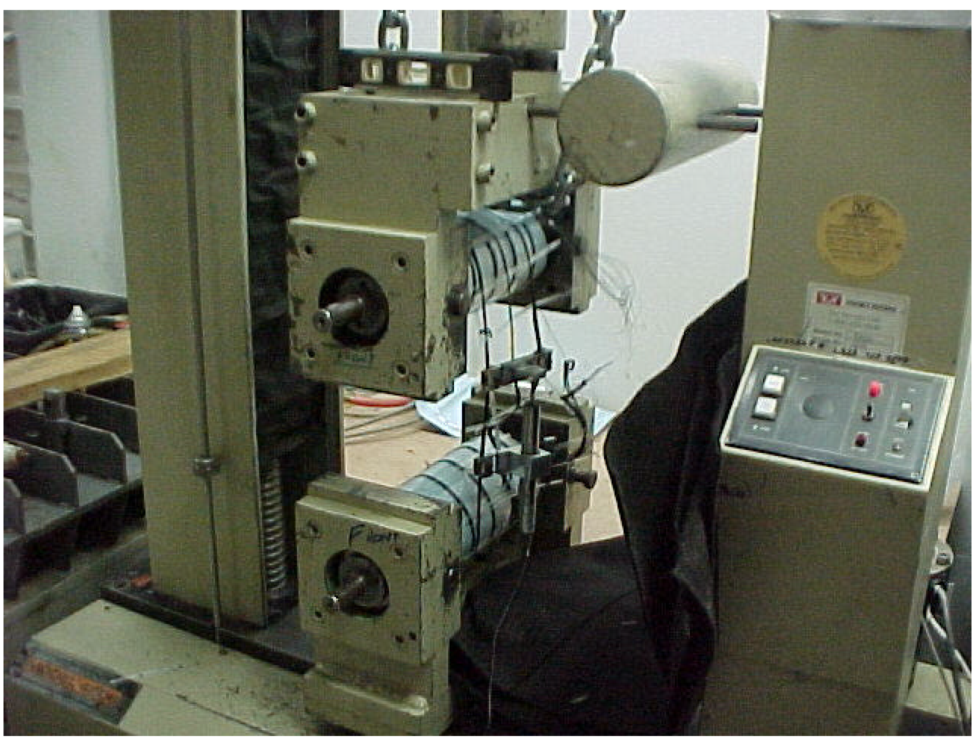


Figure 4.7. Wide Width Tensile Test on Enkagrid MAX 20 (after loading)

Figure 4.8. Wide Width Tensile Test on Geolon HS800 (before loading)

Figure 4.9. Wide Width Tensile Test on Geolon HS800 (after loading)

CHAPTER 5: SOILS

Two samples of Piedmont residual soil were used in this research program. The soil was obtained from ongoing NCDOT roadway construction projects in the Charlotte area. Mr. Clint Little of the NCDOT was instrumental in identifying appropriate sources of material and arranging delivery to the geotechnical laboratories at UNC Charlotte. The PI is indebted to Mr. Little and his associates for their willing cooperation in this vital aspect of this research project.

Once the two soils were delivered, they were minimally processed and appropriate material characterization tests were performed. These items are described for both soils in the following sections.

5.1. Soil Preparation

Processing of the soil proceeded as follows. Large soil clumps were broken down by hand and shovel, and any trash, organic material and rocks greater than 4” nominal diameter were removed by hand and discarded. The soil was passed through a large, 483 mm by 483 mm (19 in by 19 in), sieve with 19 mm (0.75 in) openings. This sieve was placed over a 55-gallon (208 L) drum. The soil was worked through the sieve to produce soil with a maximum particle size of 19 mm in the drum. Once a drum was partially filled with the processed soil, it was emptied into a 2 cubic yard, Wright self-dumping steel hopper. After all the soil obtained from the NCDOT had been processed and stored in the hopper, representative samples were taken for classification and other laboratory testing. One hopper was used for each soil type. Once filled with soil, the hoppers were covered and moved to a storage area located adjacent to the testing laboratory.

5.2 Soil #1 (A-2-4)

5.2.1 Classification

The processed soil was classified according to the American Association of State Highway and Transportation Officials (AASHTO), AASHTO M-145, as A-2-4. Classification was performed independently in the geotechnical engineering laboratories at UNC Charlotte and at the NCDOT Materials and Tests Unit.

5.2.2 Soil Properties

The grain size distribution was determined in general accordance with ASTM D 422. Results are shown in Table 5.1 and Figure 5.1.

Table 5.1. Soil#1 Grain Size Distribution

Sieve Size	Percent Passed
3/8”	99.4
#4	98.5
#10	95
#16	89.2
#40	62
#100	36.4
#200	22.1

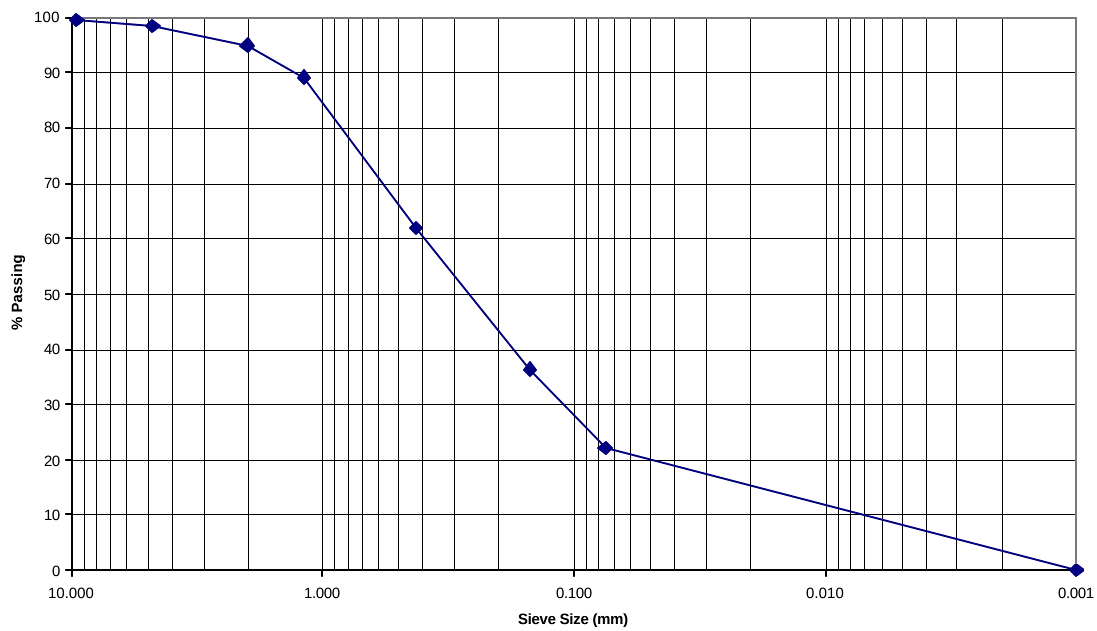


Figure 5.1. Soil #1 Grain Size Distribution Curve

The Atterberg Limits of the soil were determined in general accordance with ASTM D 4318. The Atterberg Limits establish states of consistency of the soil and are used in soil classification. The Liquid Limit of Soil #1 was 24% and the Plastic Limit was 28%. The difference between the Liquid Limit and the Plastic Limit is the Plasticity Index (PI) of the soil. In this case, since the Liquid Limit and Plastic Limit values are essentially the same, the soil is deemed non-plastic.

Specific gravity (G_s) relates the weight of soil solids to the weight of water and is a dimensionless value. The procedure for determining the specific gravity of a soil is ASTM D 854. The G_s of Soil #1 was found to be 2.79.

The moisture-unit weight relationship (compaction characteristics) of the soil was determined by using the standard Proctor test (ASTM D 698). The test results indicate a maximum dry density of 121.3 pcf at an optimum moisture content (OMC) of 11.8%. The standard Proctor curve is shown in Figure 5.2.

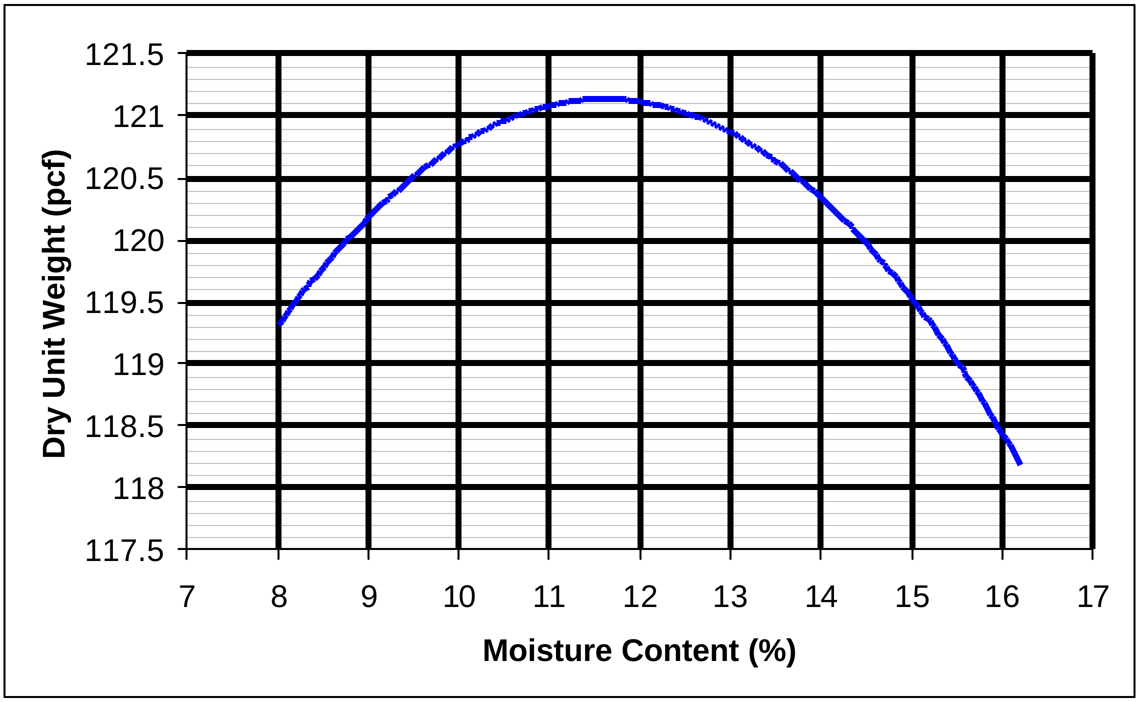


Figure 5.2. Soil #1 Standard Proctor Curve

Table 5.2. Summary of Soil #1 Properties

Soil Properties	
AASHTO Classification	A-2-4
% Fines	22.1
Liquid Limit (LL)	24%
Plastic Limit (PL)	28%
Plastic Index (PI)	NP
Specific Gravity (G_s)	2.79
Optimum Moisture Content	11.8% *
Maximum Dry Unit Weight ($?_{max}$)	121.3 pcf *

* based on Standard Proctor Test (ASTM D 698)

5.3 Soil #2 (A-4)

5.3.1 Classification

The processed soil was classified according to the American Association of State Highway and Transportation Officials (AASHTO), AASHTO M-145, as A-4. Classification was performed independently in the geotechnical engineering laboratories at UNC Charlotte and at the NCDOT Materials and Tests Unit.

5.3.2 Soil Properties

The grain size distribution was determined in general accordance with ASTM D 422. Results are shown in Table 5.3 and Figure 5.3.

Table 5.3 Soil#2 Grain Size Distribution

Sieve Size	Percent Passed
3/8”	100
#4	99.5
#10	99.4
#40	79.7
#100	48.9
#200	36.0

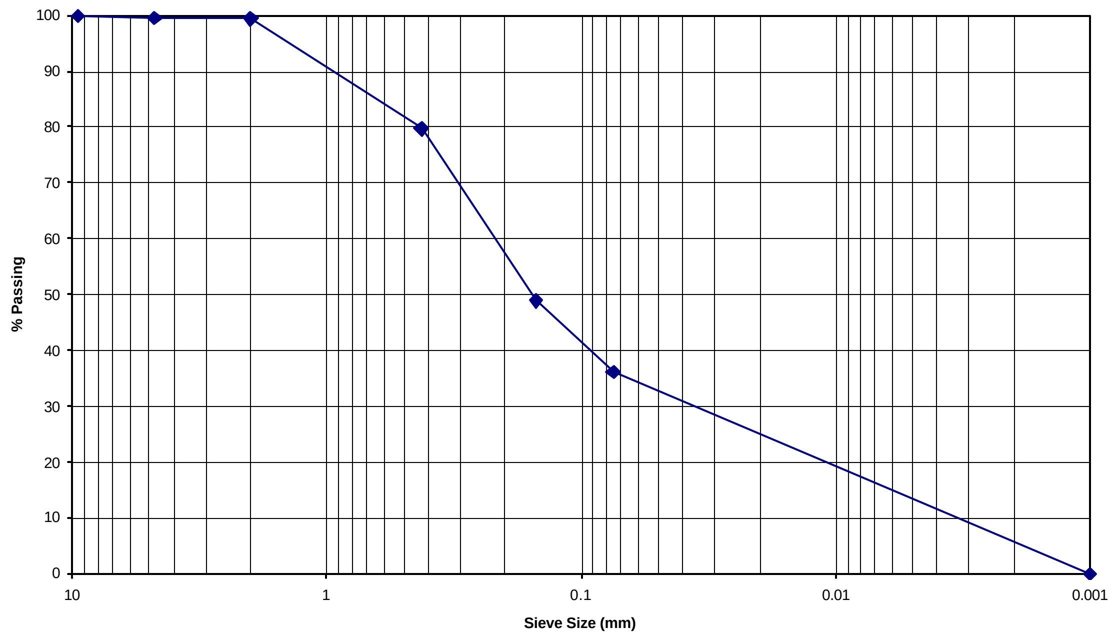


Figure 5.3. Soil #2 Grain Size Distribution Curve

The Atterberg Limits of the soil were determined in general accordance with ASTM D 4318. The Atterberg Limits establish states of consistency of the soil and are used in soil classification. The Liquid Limit of Soil #2 was 37% and the Plastic Limit was 31%. The difference between the Liquid Limit and the Plastic Limit is the Plasticity Index (PI) of the soil. In this case, since the Plasticity Index for Soil #2 is 6.

Specific gravity (G_s) relates the weight of soil solids to the weight of water and is a dimensionless value. The procedure for determining the specific gravity of a soil is ASTM D 854. The G_s of Soil #2 was found to be 2.78.

The moisture-unit weight relationship (compaction characteristics) of the soil was determined by using the standard Proctor test (ASTM D 698). The test results indicate a maximum dry density of 107.1 pcf at an optimum moisture content (OMC) of 17.1%.

The standard Proctor curve is shown in Figure 5.4.

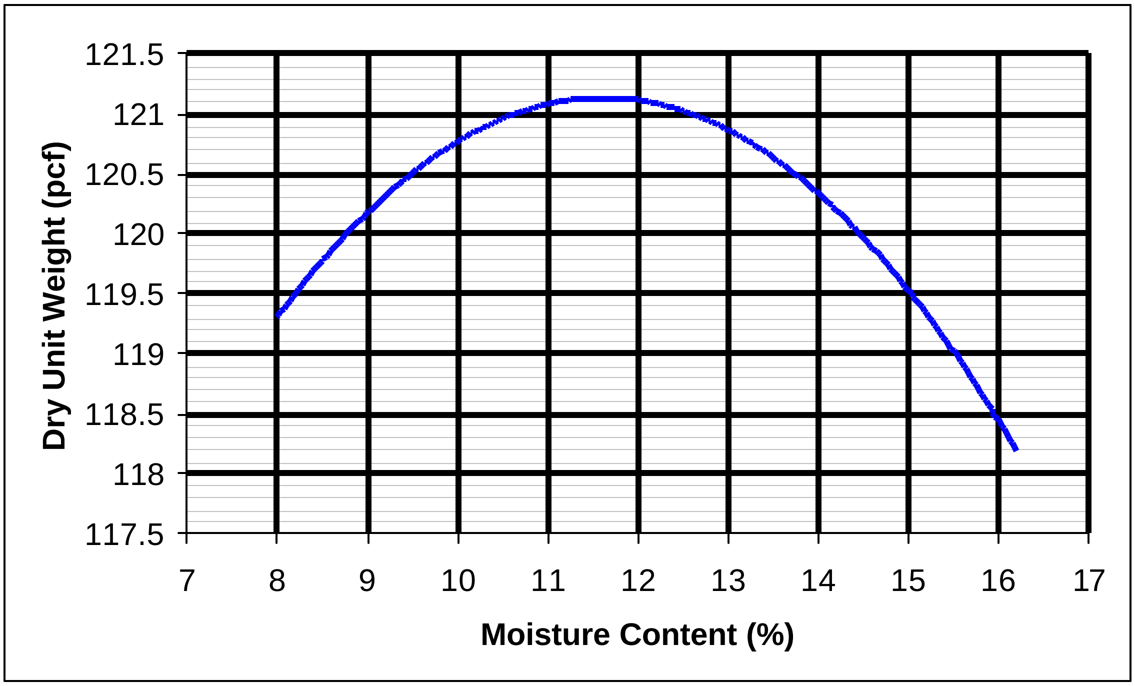


Figure 5.4. Soil #2 Standard Proctor Curve

Table 5.4. Summary of Soil #2 Properties

Soil Properties	
AASHTO Classification	A-4
% Fines	36.0
Liquid Limit (LL)	37%
Plastic Limit (PL)	31%
Plastic Index (PI)	6
Specific Gravity (G_s)	2.78
Optimum Moisture Content	17.1% *
Maximum Dry Unit Weight ($?_{max}$)	107.1 pcf *

* based on Standard Proctor Test (ASTM D 698)

5.4. Soil Handling and Compaction

In addition to the 2 cubic yard self-dumping hoppers, 55-gallon drums were used for both storage and delivery of the soil to the pullout box. Moving the soil filled drums was accomplished using a Clark forklift (model LPS, 1588 kg maximum capacity) and a Wesco drum lifter (363 kg maximum capacity) (Figure 5.5). The forklift and drum lifter combination was used to move and lift the soil filled drums over the rear wall of the pullout box.

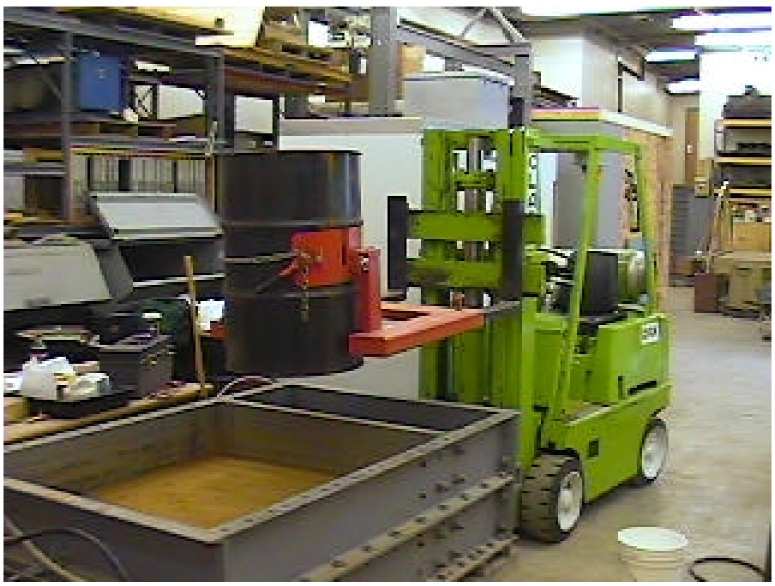


Figure 5.5. Clark Forklift and Wesco 55-gallon Drum Lifter

For testing, the moisture content of the soil in the drums was adjusted to within 2% of the OMC. When the target moisture content was achieved, the drums were sealed and moved to a staging and storage area.

The moisture and unit weight relationship of the soil was used to determine the weight of soil need to achieve 95% of the Standard Proctor maximum dry density when compacted. Before conducting a pullout test, the moisture content of the soil was checked and adjusted if necessary. The moisture content and lift volume were calculated to determine the weight needed for a particular lift.

A Whacker Packer jumping jack tamp and a 152 mm by 254 mm rectangular and tamp weighing 93 kg were used to compact the soil. The target dry density (95% of the Standard Proctor maximum dry density) was achieved when the soil lift was compacted to a final lift thickness of 6 cm.

The in-place density of the compacted soil was determined using the sand cone (ASTM D 1556). These tests were performed in one of four locations. The interior of the pullout box was separated into four equal quadrants. Each of the locations corresponded to one of the four quadrants. Test locations were: 1 rear left, 2 rear right, 3 front right, and 4 front left (Figure 5.6).

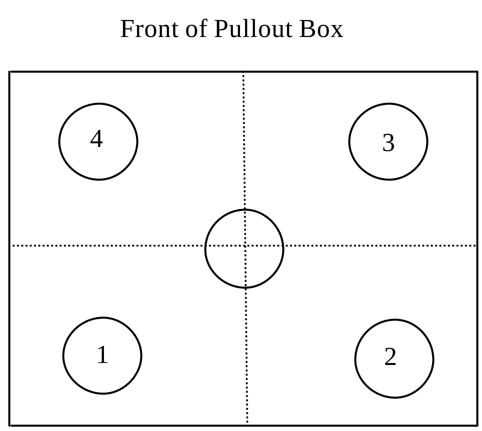


Figure 5.6. Sand Cone Density Test Locations

CHAPTER 6: TEST EQUIPMENT AND PROCEDURES

6.1. Pullout Box

The overall outside dimensions of the pullout box are 2.3 m (L) X 1.4 m (W) X 0.7 m (D). The inside length (embedment) dimension is adjustable from 0.95 m to 1.5 m by use of an interior wall insert. For the test program, the inside dimensions were 1.2 m (W) X 1.5 m (L) X 0.6 m (D).

The pullout box is constructed of hot-rolled A36 structural steel sections including steel channels, tubes, and angles (Figure 6.1). The sections are connected with 19 mm A325 structural bolts and beveled washers.

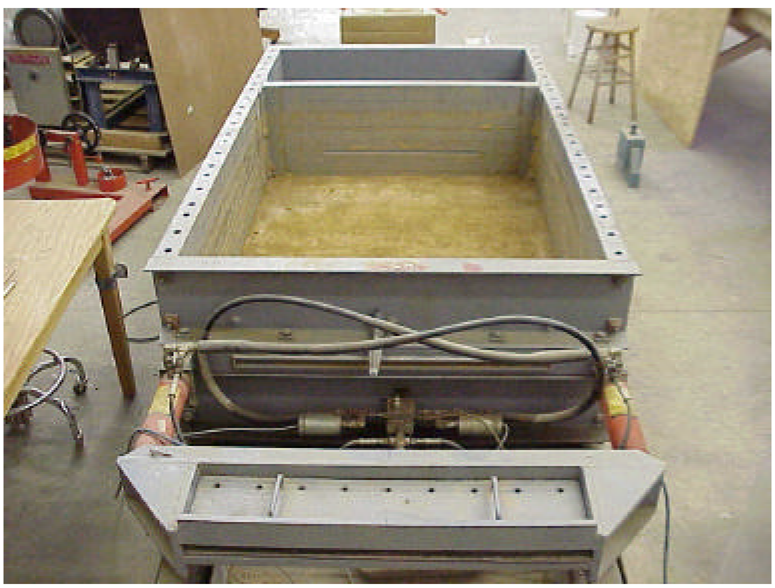


Figure 6.1. Front View of Pullout Box

The A36 steel component sections of the pullout box are as follows:

- The two sidewalls are constructed of two 2.13 m long C12X25 channels stacked on their sides and bolted together (Figure 6.2). Two 13 mm thick A36 steel plates are welded on the inside ends of each channel (Figure 6.3).
- The cover and floor are constructed of seven 1.37 m long C12X25 channels each individually bolted to the sidewalls (Figure 6.4).
- The front and rear walls are constructed of one 1.37 m long 102 mm by 76 mm steel rectangular tube bolted between two 1.37 m long C10x30 channels (Figure 6.5).
- The wall insert is constructed of three 1.22 m long C8x11.5 channels stacked and supported by four 0.6 m long L3X3X0.25 angles bolted to the sidewalls (Figure 6.6).

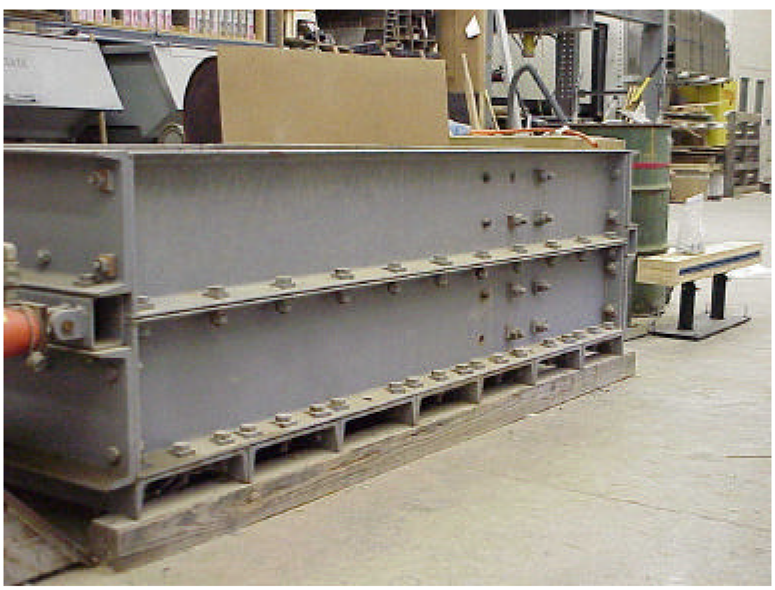


Figure 6.2. Pullout Box Sidewalls

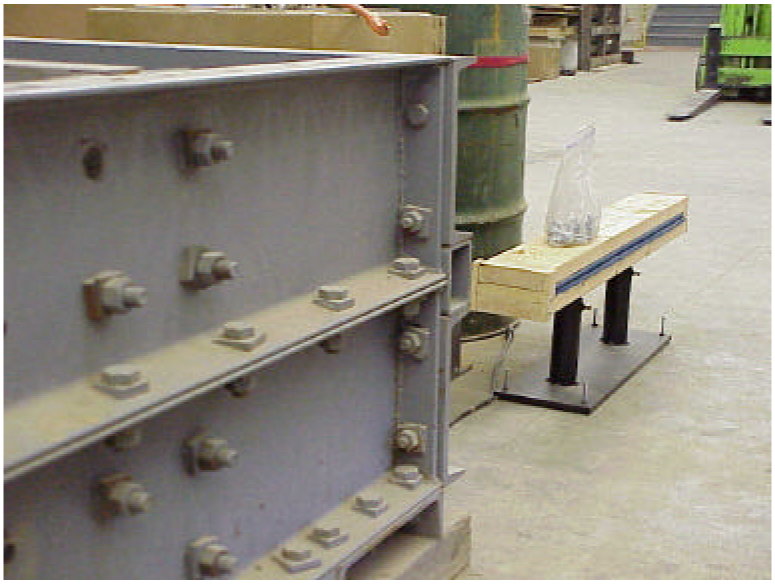


Figure 6.3. Steel Plates on Inside Ends of Each Sidewall

Figure 6.4. Cover and Floor Individually Bolted to the Sidewalls

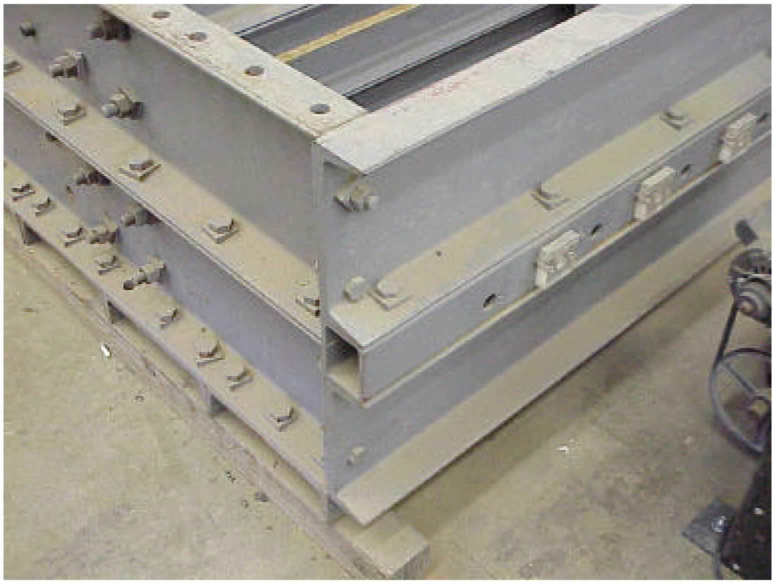


Figure 6.5. Rear and Sidewall

Figure 6.6. Stacked Rear Wall Insert

The interior wall insert is slotted to allow telltale wires to attach to the rear LVDT instrumentation located outside of the confining soil mass. The 0.9 m wide by 25 mm high slot is positioned 0.3 m above the pullout box's interior bottom and is the same elevation as the geosynthetic test specimen. A custom clamp device (constructed with steel legs and a wooden jaw) held the rear LVDTs in place during testing (Figure 6.7)

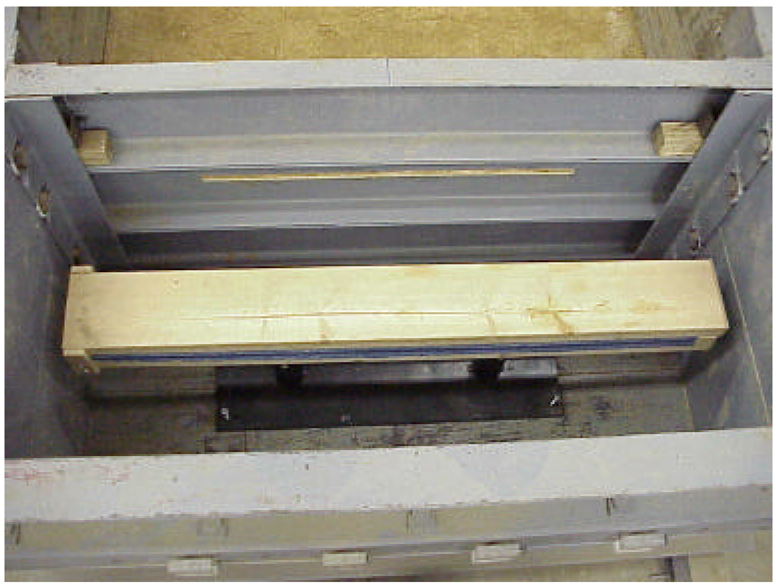


Figure 6.7. Rear LVDT Clamp Device

The front wall's steel rectangular tube is constructed with a centered 0.9 m wide by 25 mm high slot. This slot has four A36 steel plates welded perpendicular to the tubing walls to provide a continuous and level slot into the pullout box interior. An interior sleeve is formed by two 0.15 m long by 1.22 m wide by 3.2 mm thick cold-rolled steel angles. One leg of an angle is welded to the interior front wall face above the slot. The other angle is welded similarly but it is below the front wall slot to form the sleeve. For strength, both legs of the angles forming the sleeve are built-up with two additional 3.2 mm thick cold-rolled steel plates welded in a stepwise fashion (Figure 6.8).

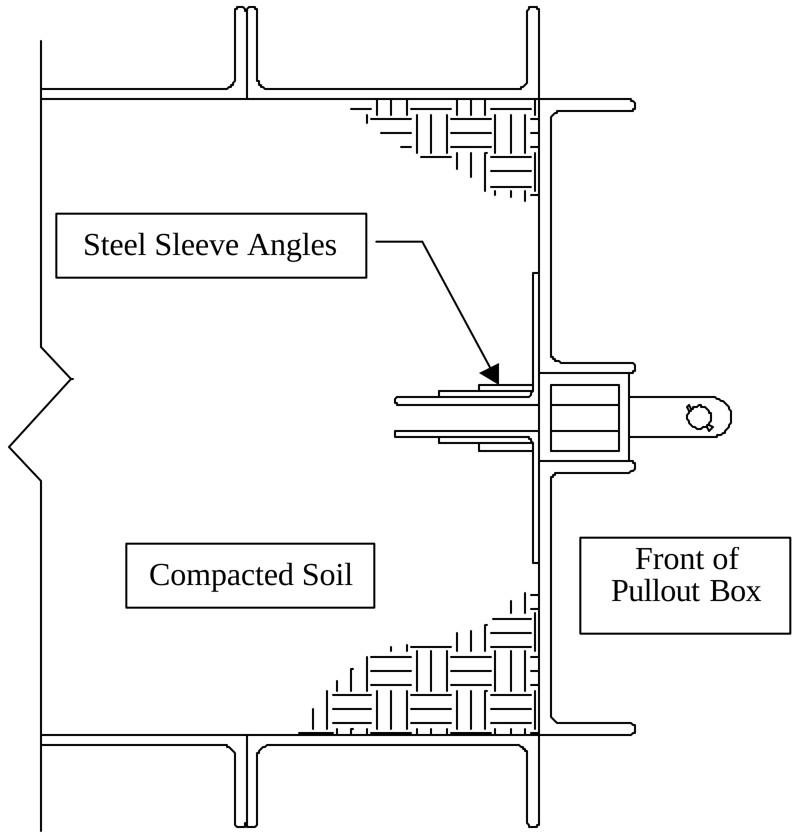


Figure 6.8. Cross Section Through Front of Pullout Box (not to scale)

6.2. Pullout Bar

The pullout bar transfers the applied tensile force to the entire width of the geosynthetic test specimen. The geosynthetic sample is kept horizontal when it is attached to the pullout bar., which ensures planar application of tensile force.

The pullout bar is constructed of hot-rolled A36 structural steel, rubber inserts, rubber lined plywood, bolts, and wheel stands (Figure 6.9). The pullout bar clamp consists of various welded A36 steel plates. The front and rear housing walls are constructed with 91 cm long by 2.5 cm high slots to allow for placement of the geosynthetic test specimen through the pullout bar's housing.

Figure 6.9. Pullout Bar

A rubber sheet was placed on the bottom of the housing followed by a plywood insert wrapped in two layers of rubber. A final rubber sheet and steel plate were placed on top of the preceding inserts. The geosynthetic test specimen was locked between the inserts and secured to the pullout bar by nine 9 mm diameter bolts through the steel cover plate, inserts, and the bottom of the housing (Figure 6.10).

The housing rests on wheel stands to allow for free movement of the pullout bar away from the pullout box. A 102 cm long by 30.5 cm wide by 2.5 cm high channel is used under each wheel stand to ensure a level and smooth rolling surface. Each wheel stand is constructed of a 50 mm diameter pipe welded to the midpoint of a 0.6 m long by 25 mm wide by 50 mm high rectangular steel pipe. Two 50 mm diameter wheel casters are welded to the underside of each wheel stand. The wheel caster positioned closest to the pullout box is fixed in the direction of the tensile force being applied. However, the wheel caster farthest from the pullout box is free to rotate.

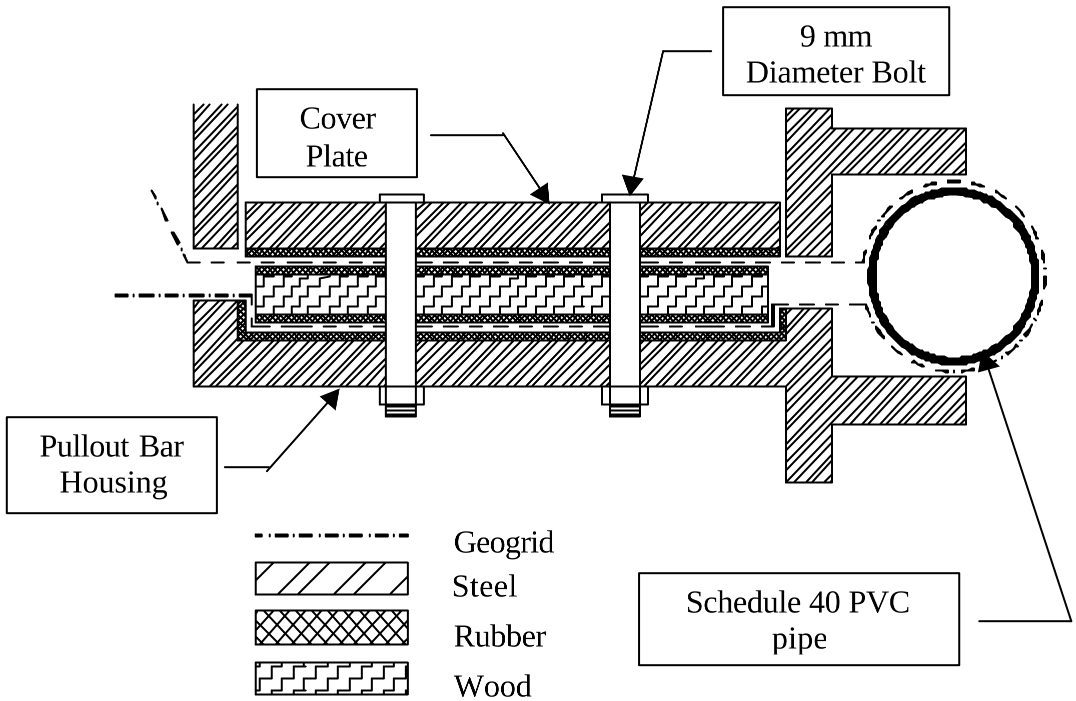


Figure 6.10. Cut Away of Pullout Bar (not to scale)

6.3. Pullout Force

Tensile force is required to pull the geosynthetic specimen out of the confining soil mass within the pullout box. The tensile force is applied to the pullout bar (and gripped geosynthetic) by two spring return hydraulic rams. Each hydraulic ram has an output capacity of over 220 KN and a stroke of 260 mm.

One hydraulic ram is connected to the pullout box on each side of the front wall sleeved slot. Each ram is connected to the pullout box via a hinge connection with a 25 mm pin. The hydraulic ram piston is secured to the pullout bar by a 13 mm bolt screwed directly into the piston through a hole in the pullout bar steel plate housing (Figure 6.11).

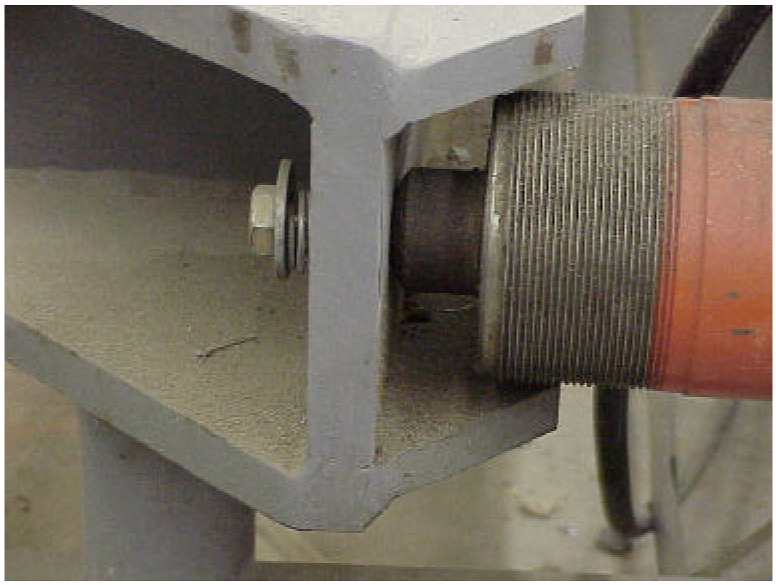


Figure 6.11. Hydraulic Ram Connection to Pullout Bar Housing

The system supplying hydraulic pressure to the rams includes an OTC electric-powered hydraulic pump and a four position, three way, metered, temperature, and pressure compensated flow control valve. The pump and control valve are connected to the rams by various hydraulic hoses and fittings.

6.4. Surcharge Pressure

A vertical surcharge pressure, normal to the geosynthetic plane of embedment, is provided to test the geosynthetic under a range of confining pressures. Increasing the surcharge pressure inside the pullout box increases the geosynthetic's confining pressure.

Increasing the vertical surcharge pressure in the pullout box is accomplished by pressurizing the 1.6 m long by 1.2 m wide, 90-mil PVC flexible air bladder (Figure 6.12). An air valve with a three-meter long pressure hose is attached to the air bladder. The three-meter long pressure hose is attached to the laboratory supplied constant pressurized air line. The air bladder is maintained at a constant pressure by using an inline pressure regulator (Porter model 8290) and a pressure dial gauge (USG) (Figure 6.13).

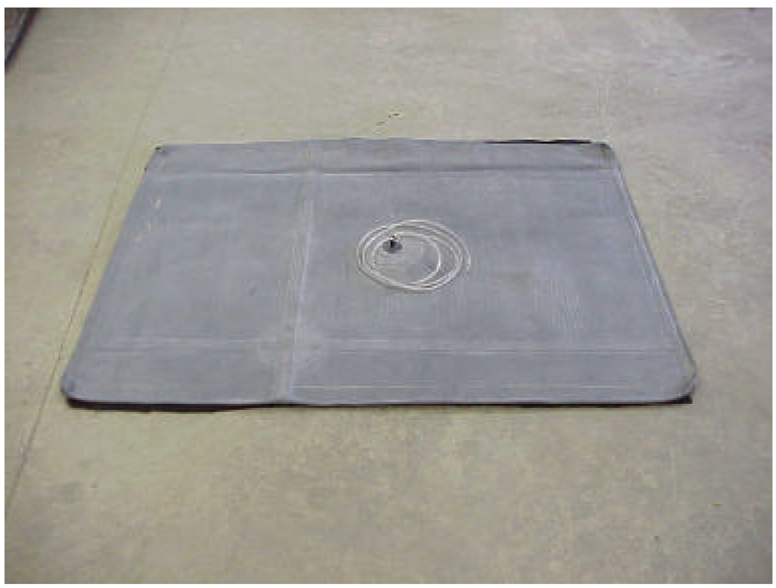


Figure 6.12. Flexible Air Bladder

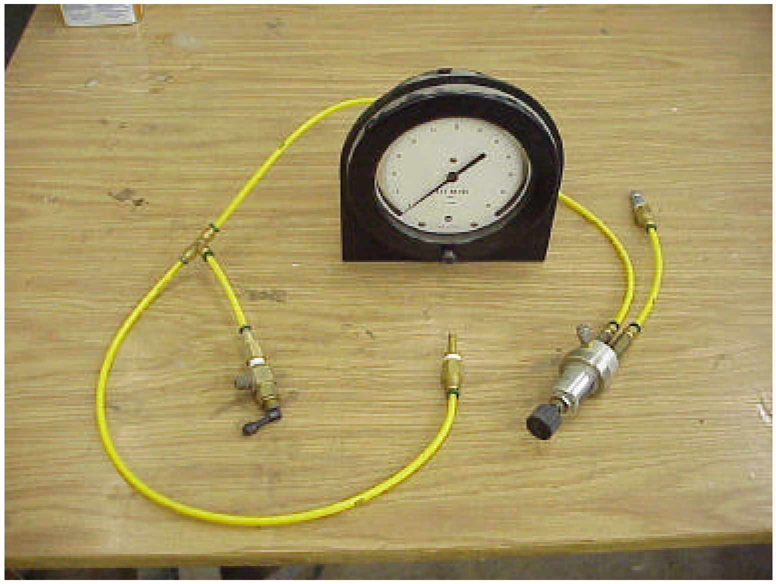


Figure 6.13. Inline Pressure Regulator and Pressure Dial Gauge

6.5. Data Acquisition System

A computerized data acquisition system (DAQ) was employed to record and store data from the pullout tests. The DAQ can display data in real-time from multiple input channels. The DAQ used in this research consisted of four components:

1. Computer hardware and software
2. Data acquisition device hardware
3. Data acquisition software
4. Electronic Transducers

Each of these components is described in the following paragraphs. The computer and data acquisition device hardware are shown in Figure 6.14.

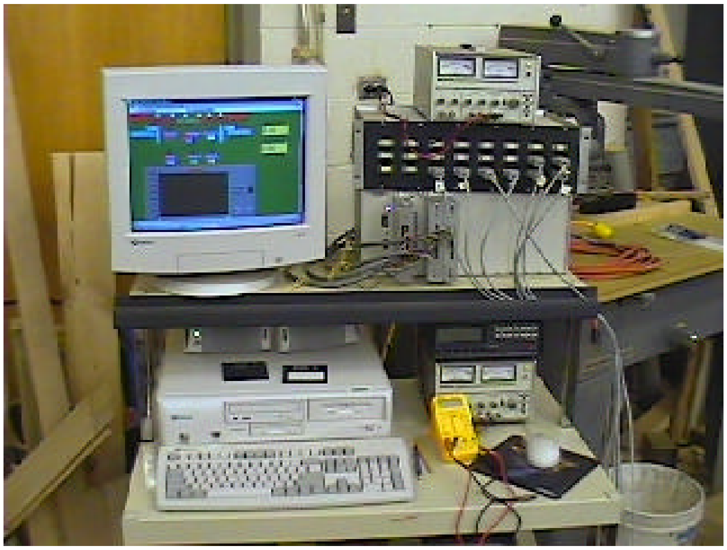


Figure 6.14. Computer and Data Acquisition Hardware

The computer was an IBM compatible Gateway with a Pentium II 333 MHz MMX processor. The computer was equipped with 64 Megabytes of RAM and operated under the Windows NT Version 4.0 Build 1381 with Service Pack 3 operating system.

Most of the data acquisition device hardware consisted of National Instruments products except for the custom constructed electronic instrumentation relay-box (EIRB). The National Instruments hardware components consisted of the following:

- PCI DAQ board in the computer (model PCI-MIO-16E-4)
- A 12 slot chassis (model SCXI-1001)
- Two four channel isolation amplifier with excitation modules (model SCXI-1121)
- Two terminal block connectors for the SCXI-1121 (model SCXI-1321)
- A 32 channel differential multiplexer/amplifier module (model SCXI-1100)
- A terminal block connector for the SCXI-1100 (model SCXI 1303)

The EIRB was designed and fabricated at UNC-Charlotte. It is the multi-port component in the center of Figure 6.15. This device allows for the application of the proper excitation voltage to individual transducers. The EIRB also allowed quick connect and disconnect of instrumentation to the DAQ by use of standard 9 pin (DB9) connectors. This was accomplished by hardwiring all necessary connections for the transducers to each DB9 connector. The transducer was wired to a “male” BD9 and connected to a “female” BD9 on the EIRB. The “female” DB9 represents a specific channel in the DAQ. The channels were configured through the National Instruments software (i.e., gain and calibration). This allows the same channel to perform data acquisition from various transducer types (strain gage, pressure transducer, LVDT, etc).

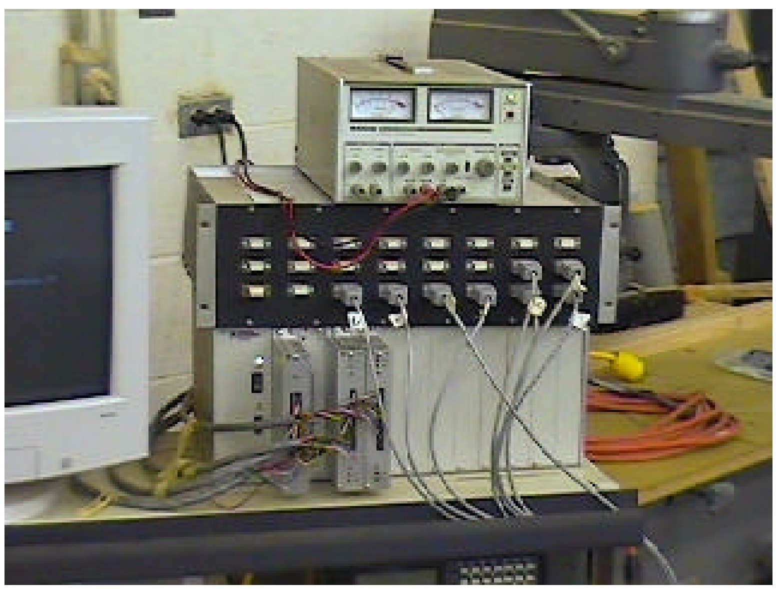


Figure 6.15. Electronic Instrumentation Relay-Box

The data acquisition software was National Instruments LabVIEW Version. 5.1. LabVIEW allows for interfacing the electronic transducers through the DAQ hardware by means of the LabVIEW graphic language, G. Specific DAQ programs called virtual instruments (VIs) were written in G by the user to allow for a totally user definable DAQ system. The user defines and writes the VI to perform the specific data acquisition required.

The electronic transducers consisted of two pressure transducers (Setra Systems Inc., model 280E) and six LVDTs (Schlumberger Industries models DC25 and DC50). The electronic transducers were excited by two variable power supply units (Leader model LPS-151). These two units were used to excite the LVDTs and pressure transducers by 10 and 24 volts, respectively.

The pressure transducers measured the hydraulic pressure applied to the pullout bar rams, and ultimately, the force applied to the geosynthetic. Two LVDTs (model DC50) were used to measure the displacement of the pullout bar. One LVDT was attached to each hydraulic ram. These LVDTs were monitored during testing to ensure a constant strain rate of 1 mm/min was maintained. Three LVDTs (model DC25) were attached within the embedded portion of the geosynthetic by telltales. The sixth LVDT (model DC25) was attached to the geosynthetic at the front opening of the pullout box.

6.6. Instrument Calibration

Calibration of the electronic transducers correlates the electronic signals they produce (volts) to physical measurement units (mm and kPa). The pressure transducers were calibrated using a hand calibrating hydraulic pump (Ralston Instruments) and a voltmeter. A change in applied pressures was correlated with the change in voltage. The linear function was graphed and inputted into LabVIEW to calibrate the pressure traducers' voltage change to pressure change. The LVDTs were calibrated using a custom fabricated calibration device (Figure 6.16) and the DAQ.

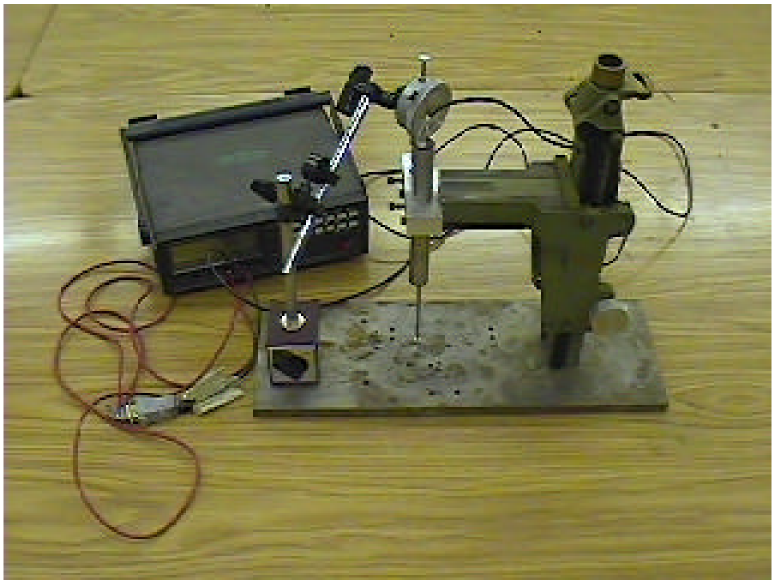


Figure 6.16. LVDT Calibration Device

A calibration VI was written in LabVIEW to facilitate the use of the DAQ in the calibration process. Each of the LVDTs was placed vertically in the calibration device. The device held the LVDT in place and allowed the extension rod to extend and retract from the LVDT housing in uniform increments. The LVDT was excited and wired to the DAQ. The increase or decrease in displacement was correlated with the change in voltage. The linear range for each LVDT was established and the associated calibration file was saved for each instrument. Because a VI was written to calibrate the LVDTs, the calibration data was input directly into LabVIEW for future VIs using these instruments. An example LVDT calibration curve is shown in Figure 6.17.

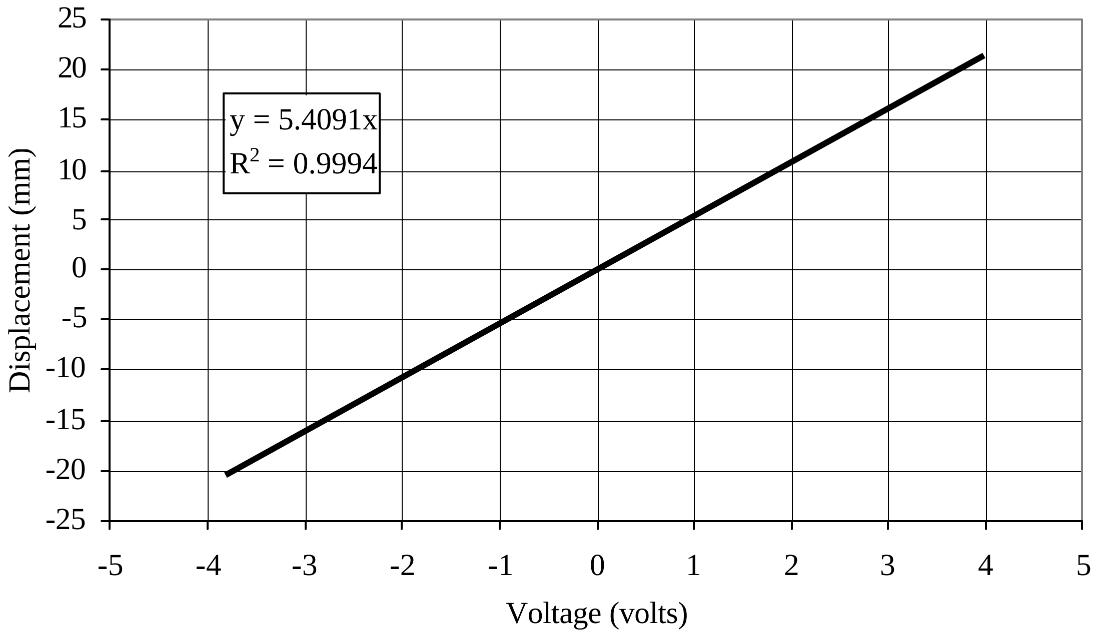


Figure 6.17. LVDT Calibration Chart

6.7. Test Procedure

The tasks performed for each test included: soil preparation, geosynthetic preparation, soil placement and compaction, geosynthetic placement, instrumentation setup, pullout box and pullout bar setup, test performance, and post test tasks.

The soil preparation first consisted of determining the moisture content of the soil. The weight of soil needed for each lift was placed in one 55 gal drum. After the soil lifts were sealed into drums the geosynthetic specimen was prepared.

The geosynthetic was cut to test dimensions and three telltale wires were connected to the geosynthetic. The wires were connected to the geosynthetic by twisting steel fishing leader at the desired locations (i.e., ¼, ½, and ¾ of the embedment depth and width) (Figure 6.18).

After the geosynthetic test specimen and soil were prepared the placement and compaction of the lower lift commenced. The lower 180 mm of the pullout box (same elevation as the front wall opening) was filled with soil in loose lifts, leveled across the test area, and compacted by a combination of mechanical and hand tamping to a final depth of 60 mm. Each lift was compacted to a target density of 95% of the Standard Proctor maximum dry density. A chalk line was laid on the last of the lower lifts to form a centered construction line for placement of the test specimen.

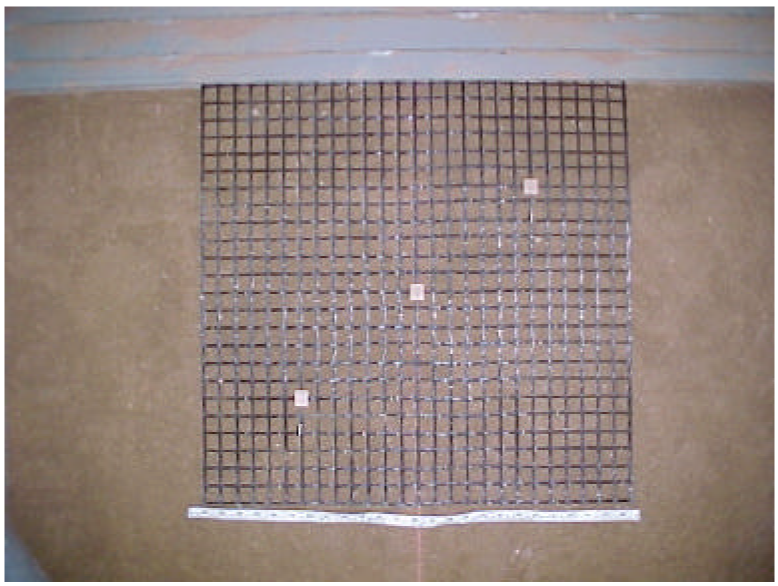


Figure 6.18. Typical Telltale Connections

The prepared geosynthetic test specimen was placed inside the pullout box and drawn through the front wall opening. The geosynthetic was aligned with the chalk line and measurements made to ensure proper embedment depth. The telltale wires were threaded through 3 mm diameter aluminum tubing to ensure the surcharge loads would not hinder the wire’s movement. Finally, pretest photographs where taken of the geosynthetic as it appeared in the pullout box.

After the placement of the geosynthetic, soil was hand placed over the geosynthetic to ensure no movement of the test specimen during the placement of the first overburden soil layer. The upper 180 mm of the pullout box was filled with soil and compacted in the same manner as the lower half of the pullout box.

Following the soil and geosynthetic placement, the pullout box was readied for testing. A non-woven needle punched geotextile cut to the pullout box inside plan dimensions was placed over the last soil lift to protect the air bladder from the soil layer. Next, an air bladder, also constructed to the pullout box inside plan dimensions, was placed over the geotextile. The air bladder was followed by a plywood spacer cut to the same inside dimensions for protection from the steel channels. A hose was connected to the air valve of the air bladder and fed through a drilled hole in the plywood spacer. The top of the box was comprised of six channels placed across the pullout box from sidewall to sidewall. The channels were attached to the sidewalls with bolts tightened with an impact wrench. The hose attached to the bladder's air valve was fed through a drilled hole in the center channel.

The freeboard portion of the geosynthetic extending out of the pullout box's front opening was connected to the pullout bar. The geosynthetic was drawn through the front opening of the pullout bar over a rubber insert and out the rear opening. The geosynthetic was pulled taut, wrapped over a piece of Schedule 40 PVC pipe, and fed back through the rear opening of the pullout bar. This process was conducted to remove slack from the test specimen. A plywood insert lined with a rubber sheet on both sides was placed over the geosynthetic. The geosynthetic was laid over the rubber lined plywood insert and through the front opening of the pullout bar. A final rubber insert was placed over the geosynthetic test specimen. This process was conducted while maintaining tension on the test specimen. A steel cover plate insert was placed over the final rubber insert and attached to the pullout bar with two equally spaced rows of 9 mm diameter bolts.

The pullout box and pullout bar were set for the test. The DAQ instrumentation was attached to the in-place geosynthetic test specimen and pullout bar:

1. One LVDT was attached to each of the two pressure rams.
2. An LVDT was attached to the freeboard portion of the geosynthetic, approximately 10 mm from the front pullout box opening.
3. The three telltales from the geosynthetic were connected to the three rear pullout box LVDTs.

Next, the DAQ was prepared for testing. The front and rear LVDTs and pressure transducer lead wires were connected to their respective channels on the DAQ relay box. The DAQ computer and hardware warm up was initiated. The DAQ software was started and software and hardware pretest checks performed. The LVDTs were adjusted to the linear range limit to achieve the maximum range of data acquisition.

The test was ready to begin after the air bladder was inflated and maintained at the desired surcharge pressure for a minimum of one hour. After the hour of airbladder seating time, the DAQ software program was initiated and the hydraulic pump was turned on to begin application of the pullout force. The loading rate was maintained at 1 mm/min for results relative to near static testing. Test termination occurred when the load decreased relative to an increase in time and displacement. Test termination occurred approximately 3 hours after initiation of the test.

After test termination, the hydraulic pump was turned off and the load removed from the test specimen. The air bladder was depressurized to remove the surcharge. The data acquired by the DAQ was saved and the files immediately backed up. DAQ hardware and software were shut down and the LVDT and pressure transducer lead wires were unplugged from the EIRB. The LVDTs were removed from the pullout box and stored. The pullout box lid channels, geotextile, air bladder, plywood spacer, and overburden soil were removed down to a level 30 mm above the geosynthetic. The 30 mm of soil over the geosynthetic was removed by careful excavation using brushes and small tools so as not to disturb post-test geosynthetic orientation. After excavation of the geosynthetic, post-test photographs where taken of the geosynthetic as it appeared in the pullout box. The geosynthetic and remaining soil were removed from the pullout box. All soil removed was placed in random 55-gallon drums to disperse the material throughout the next soil lifts. Images portraying typical testing procedures are given in Figures 6.19 to 6.33.

Figure 6.19. Fork Lift & 2-Cubic Yard Hopper of Soil

Figure 6.20. Spreading Moisture-Conditioned Soil

Figure 6.21. Mechanical Compaction of Soil

Figure 6.22. Mechanical Compaction

Figure 6.23. Sand Cone Density Test

Figure 6.24. Deployment of Geotextile Specimen

Figure 6.25. Smoothing Embedded Portion of Geotextile Specimen

Figure 6.26. LVDT Telltales in Protective Tubing

Figure 6.27. Hand Tamping Directly Over Geosynthetic Specimen

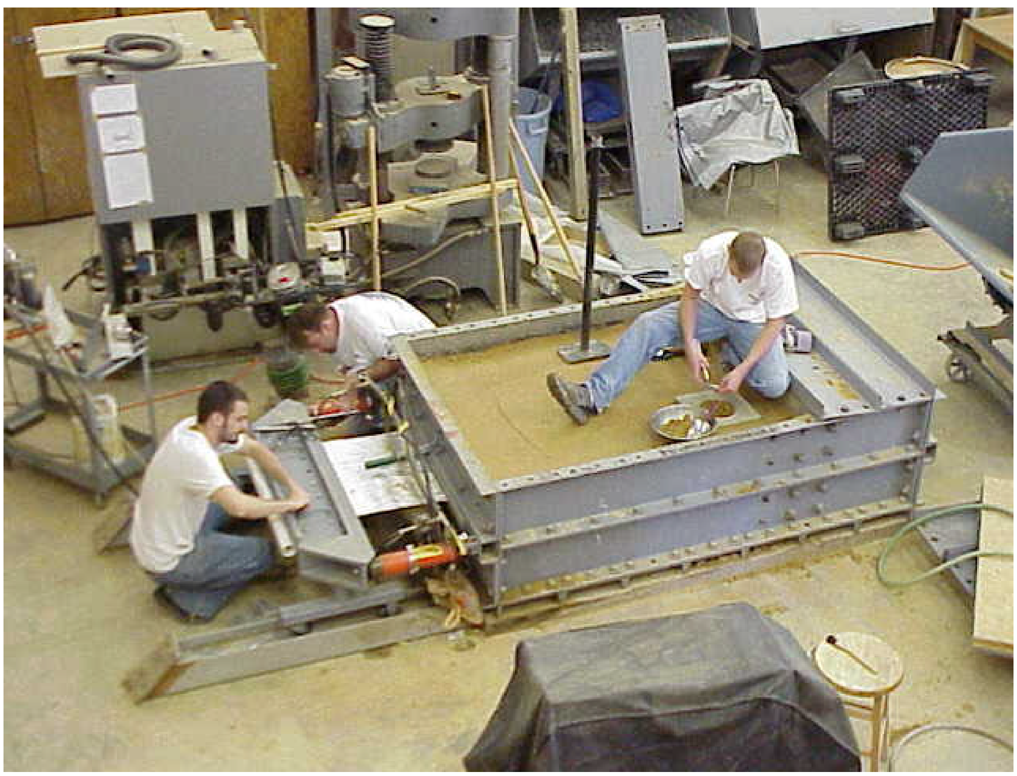


Figure 6.28. Final Stages of Sample Preparation

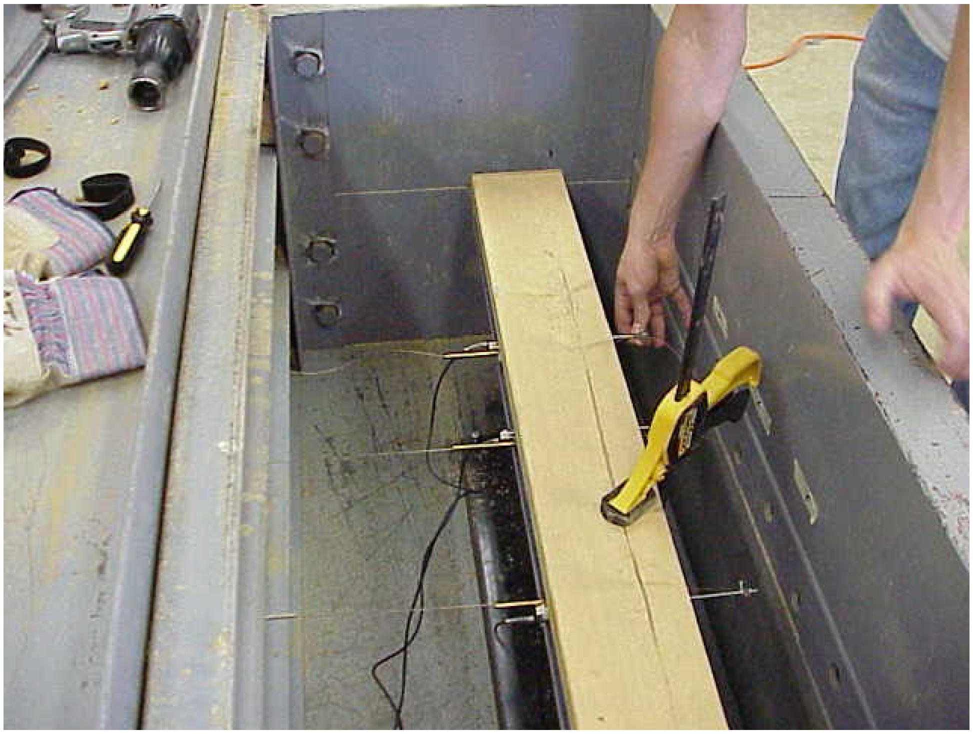


Figure 6.29. Rear LVDT Clamping Mechanism

Figure 6.30. Securing Geotextile Specimen into Pullbar

Figure 6.31. Attaching LVDT to Hydraulic Ram

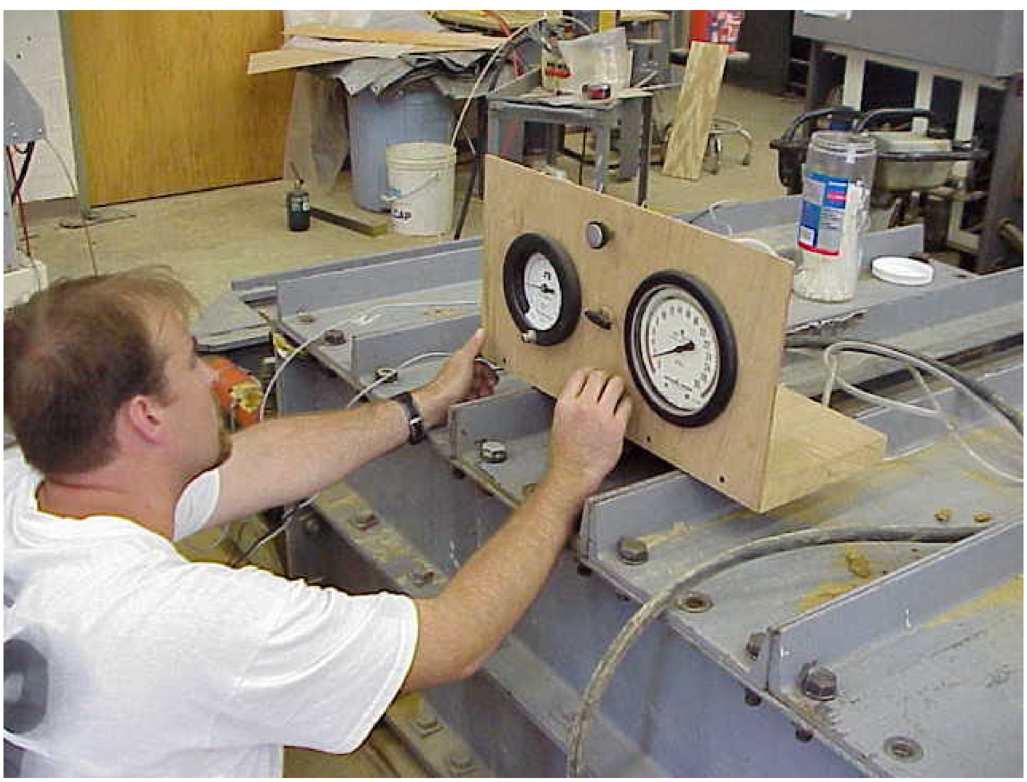


Figure 6.32. Setting Confining Pressure

Figure 6.33. Post-Test Excavation

CHAPTER 7: TEST RESULTS

The individual test results are presented in this chapter. A total of 25 different tests were performed in this project. The complete test matrix is shown in Table 7.1. Table 7.1 contains five columns of information: project test number, test name, soil type, reinforcement type, and confining pressure. The project test numbers were assigned chronologically, so the first test is No. 1 and the last test is No. 25. The test name consists of four parts: the date performed (month and day), the soil type, the reinforcement type, and the confining pressure. The soil classified as A-2-4 was deemed Soil No. 1; the soil classified as A-4 was deemed Soil No. 2. The shorthand notation for the geosynthetic reinforcement is as follows:

FGG Flexible Geogrid

HSGT High Strength Geotextile

LSGT Low Strength Geotextile

RGG Rigid Geogrid

The confining pressure is given in pounds per square inch (psi) and is either 1, 2, 4, or 8 psi, depending on the geosynthetic product being tested.

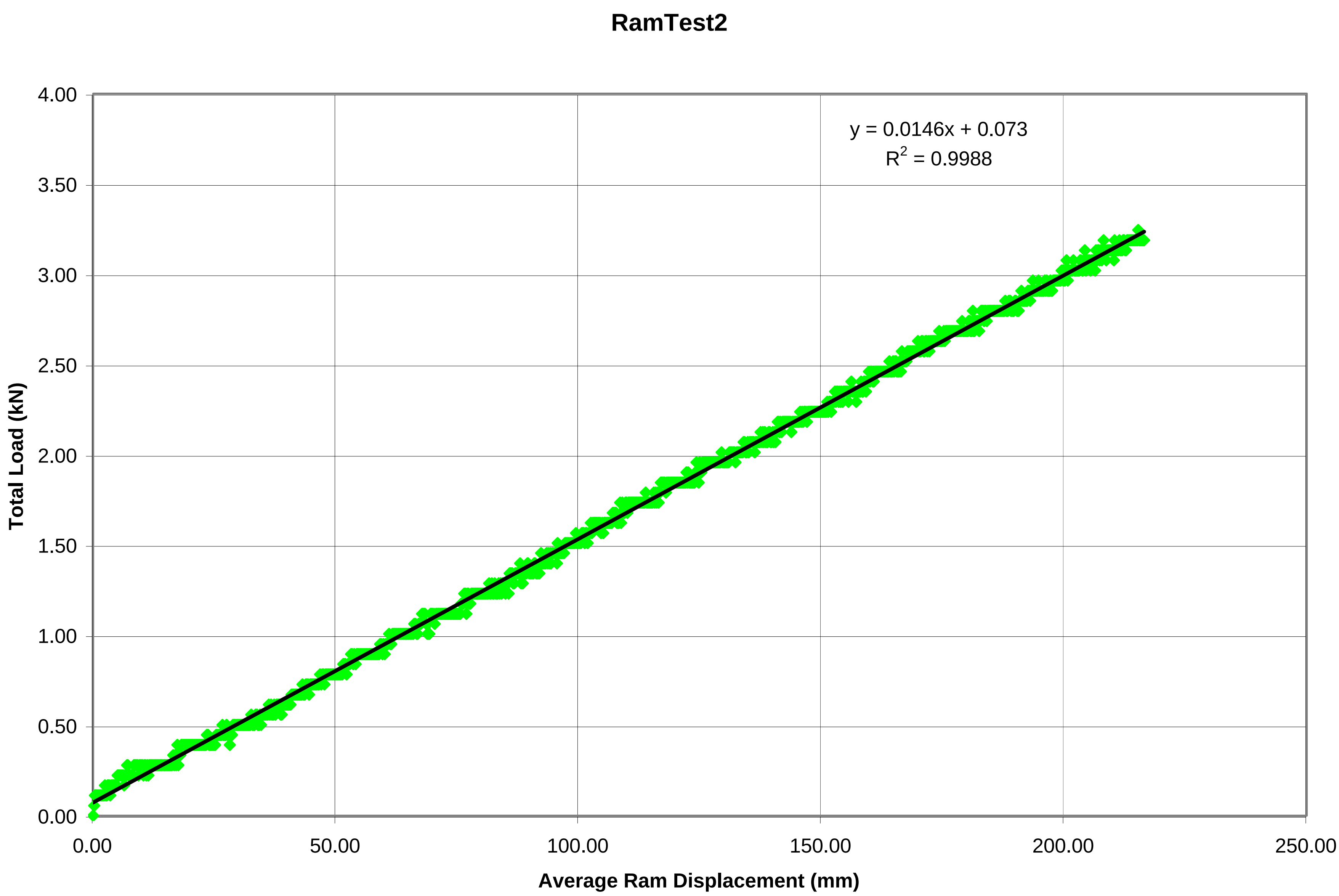
Two of the twenty-five tests were "empty box" tests used to calibrate the hydraulic loading system (Project Test Nos. 19 and 25). Representative results from this type of test are shown immediately following Table 7.1. This figure present load versus displacement data for the hydraulic loading system with no geosynthetic attached to the pullbar. This represents, among other things, both the static and "rolling" friction inherent in the system during any test. These system loads were taken into account when presenting results from actual geosynthetic pullout tests.

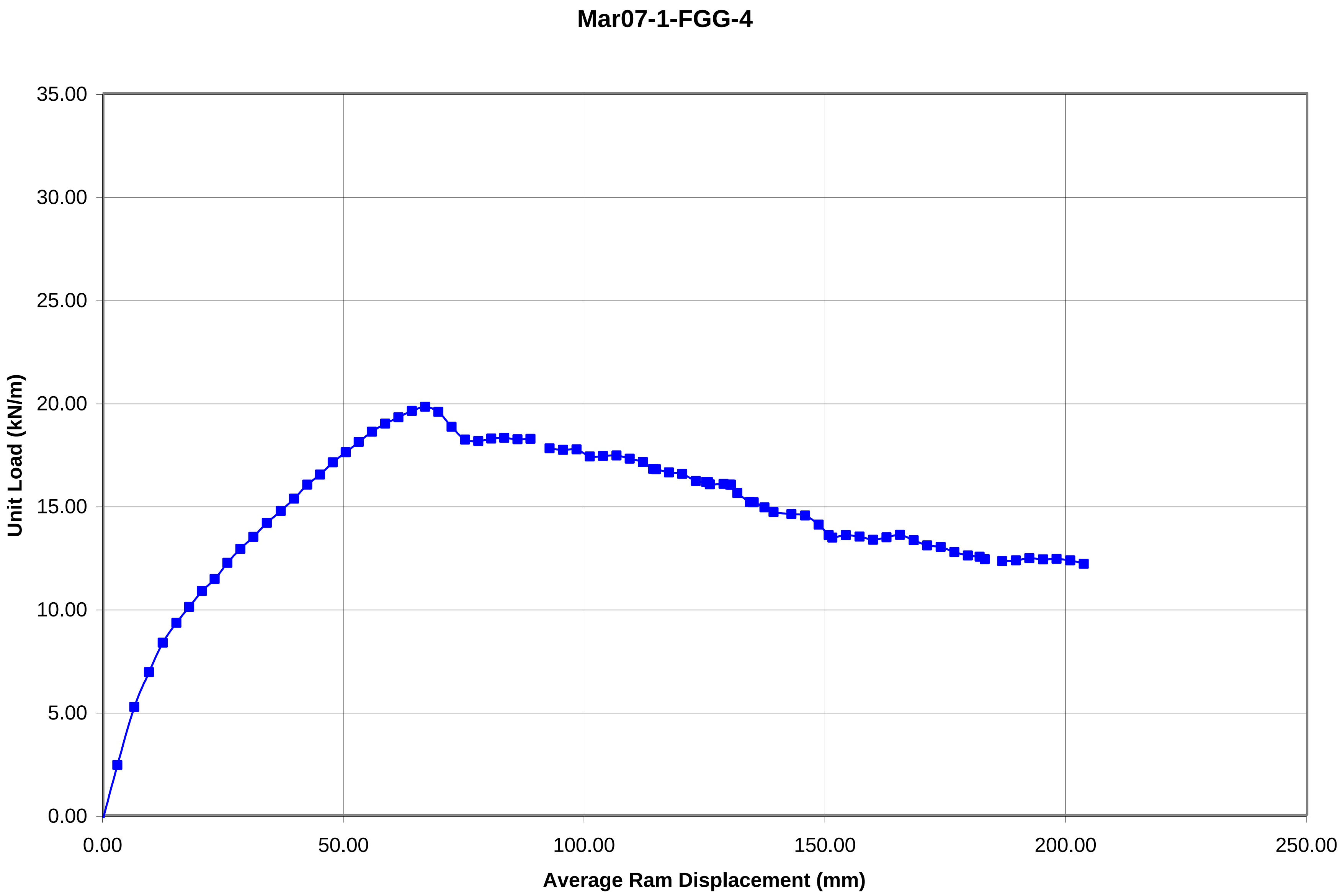
Two test configurations were repeated. Project Test Nos. 1 and 23 both tested the A-2-4 soil with the flexible geogrid under a 4 psi confining pressure. Project Test Nos. 7 and 24 both tested the A-2-4 soil with the low strength geotextile under a 4 psi confining pressure. These tests were repeated to improve the overall quality of the data collected.

In the remainder of this chapter, figures of individual test results are presented. Pairs of plots are presented for twenty-one tests (Project Test Nos. 3-18 and 20-24). The first of the paired figures presents a plot of applied load verses ram displacement; the second figure presents geosynthetic displacements verses ram displacement. Geosynthetic displacements were measured at three locations embedded in the soil and at the front of the pullbox. The embedded locations are deemed, Rear Right, Rear Middle, and Rear Left. The Rear Right location is at the ¼ point closest to the front of the pullbox (15 cm from the front of the box). The Rear Middle location is at the center of the embedded sample (30 cm from the front of the box), and the Rear Left location is at the ¾ point on the embedded sample (45 cm from the front of the box). These figures are arranged chronologically and can be identified by the Test Name as listed in Table 7.1.

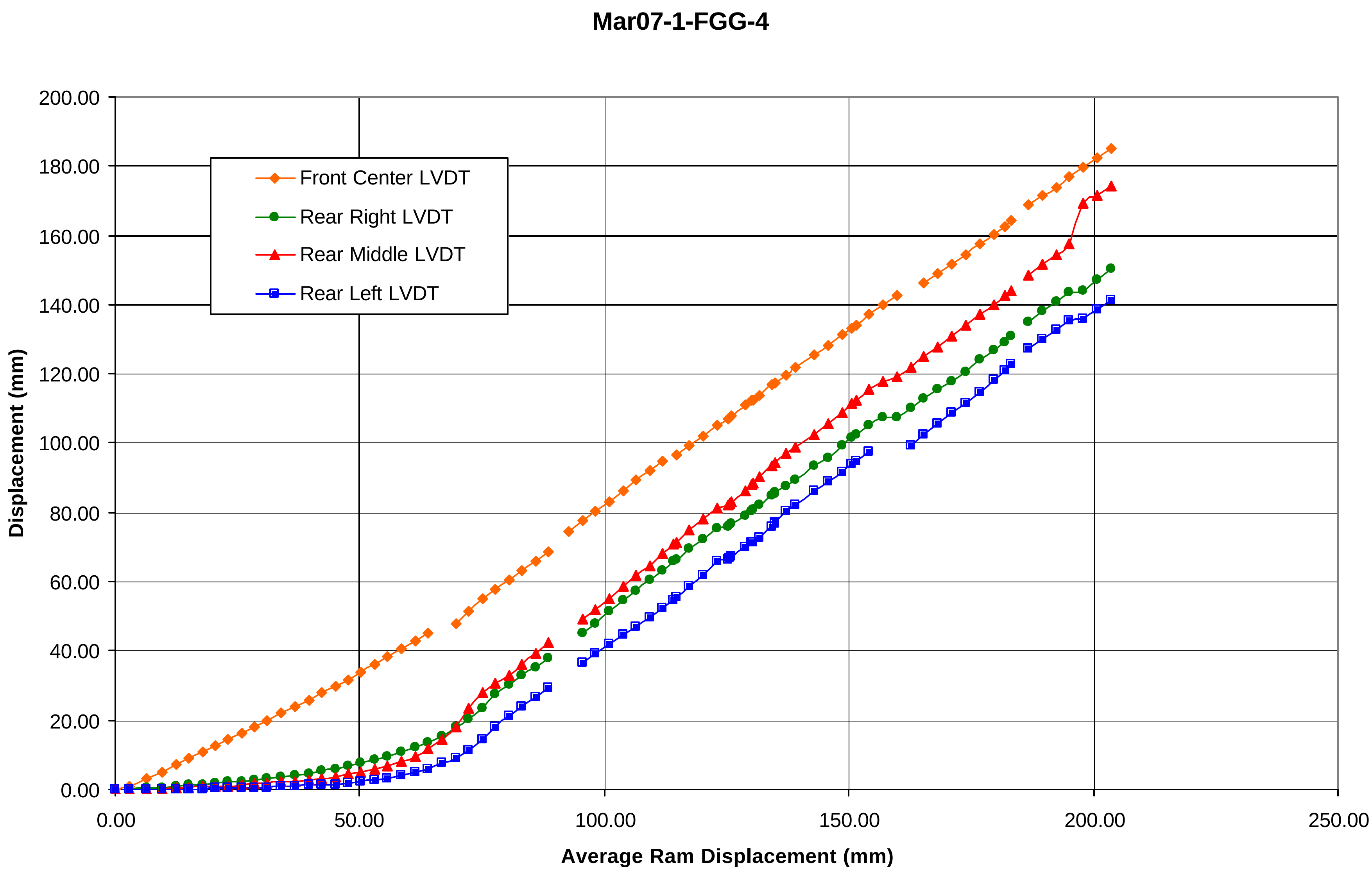
Table 7.1. Pullout Test Matrix

Project Test Number	Test Name	Soil Type (Class II, Type 2)	Reinforcement Type	Confining Pressure (psi)
1	Feb28-1-FGG-4	A-2-4	Flexible Geogrid	4
2	Mar06-1-FGG-8	A-2-4	Flexible Geogrid	8
3	Mar07-1-FGG-4	A-2-4	Flexible Geogrid	4
4	Mar21-1-FGG-2	A-2-4	Flexible Geogrid	2
5	Apr12-1-HSGT-4	A-2-4	High Strength Geotextile	4
6	Apr16-1-HSGT-8	A-2-4	High Strength Geotextile	8
7	Apr20-1-LSGT-4	A-2-4	Low Strength Geotextile	4
8	Apr27-1-RGG-4	A-2-4	Rigid Geogrid	4
9	May06-1-RGG-2	A-2-4	Rigid Geogrid	2
10	May10-1-RGG-1	A-2-4	Rigid Geogrid	1
11	May15-2-FGG-4	A-4	Flexible Geogrid	4
12	May17-2-FGG-2	A-4	Flexible Geogrid	2
13	May18-2-RGG-4	A-4	Rigid Geogrid	4
14	May22-2-RGG-2	A-4	Rigid Geogrid	2
15	May23-2-LSGT-4	A-4	Low Strength Geotextile	4
16	May24-2-HSGT-4	A-4	High Strength Geotextile	4
17	May29-2-HSGT-2	A-4	High Strength Geotextile	2
18	May30-2-LSGT-2	A-4	Low Strength Geotextile	2
19	Empty box 1	none	none	none
20	Jun04-2-FGG-4	A-4	Flexible Geogrid	4
21	Jun06-2-FGG-2	A-4	Flexible Geogrid	2
22	Jun8-2-FGG-1	A-4	Flexible Geogrid	1
23	Jun25-1-FGG-4	A-4	Flexible Geogrid	4
24	Jun27-1-LSGT-4	A-4	Low Strength Geotextile	4
25	Empty box 2	none	none	none

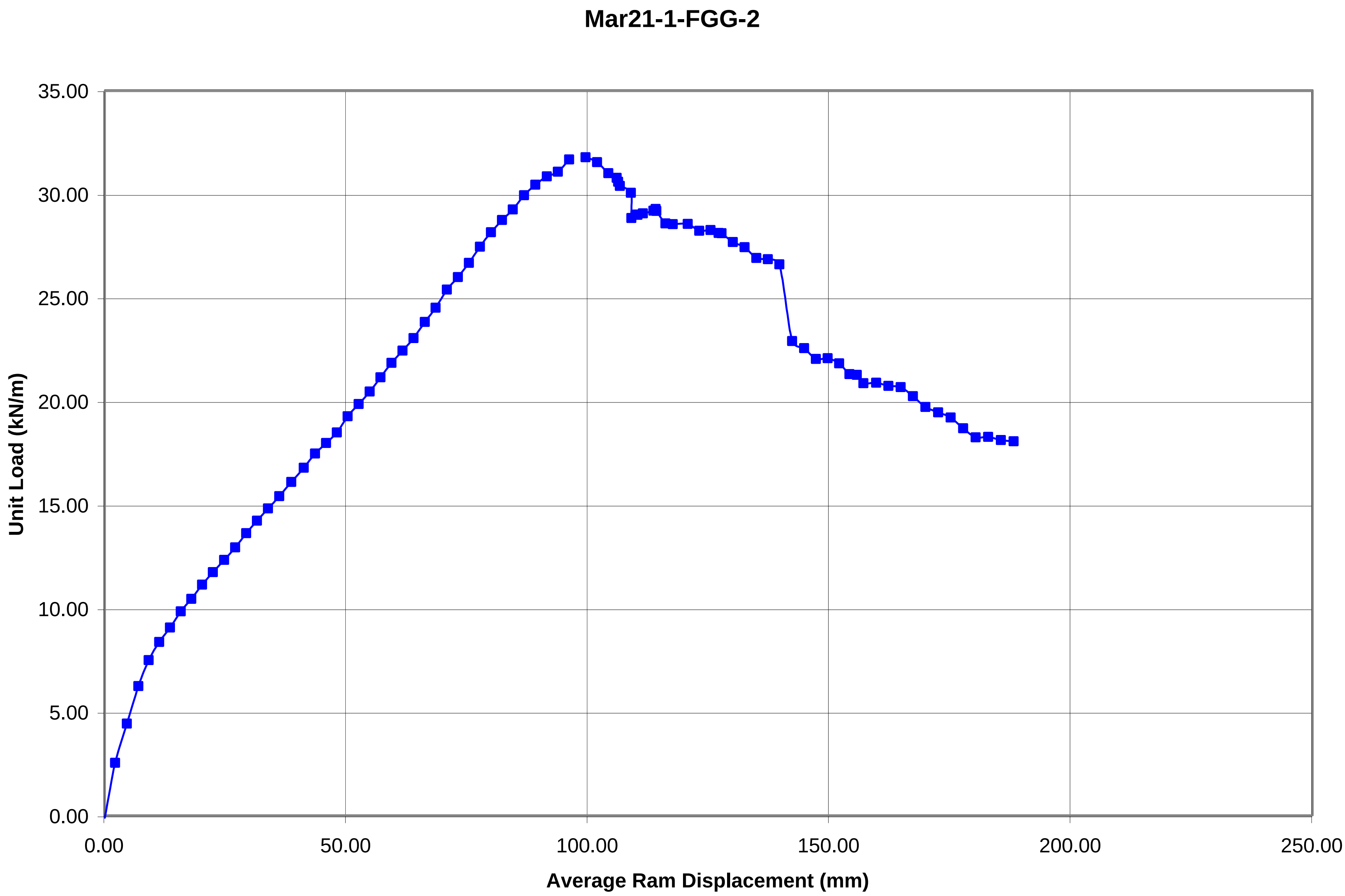

RamTest2
y = 0.0146x + 0.073
R^2 = 0.9988
Total Load (kN)
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



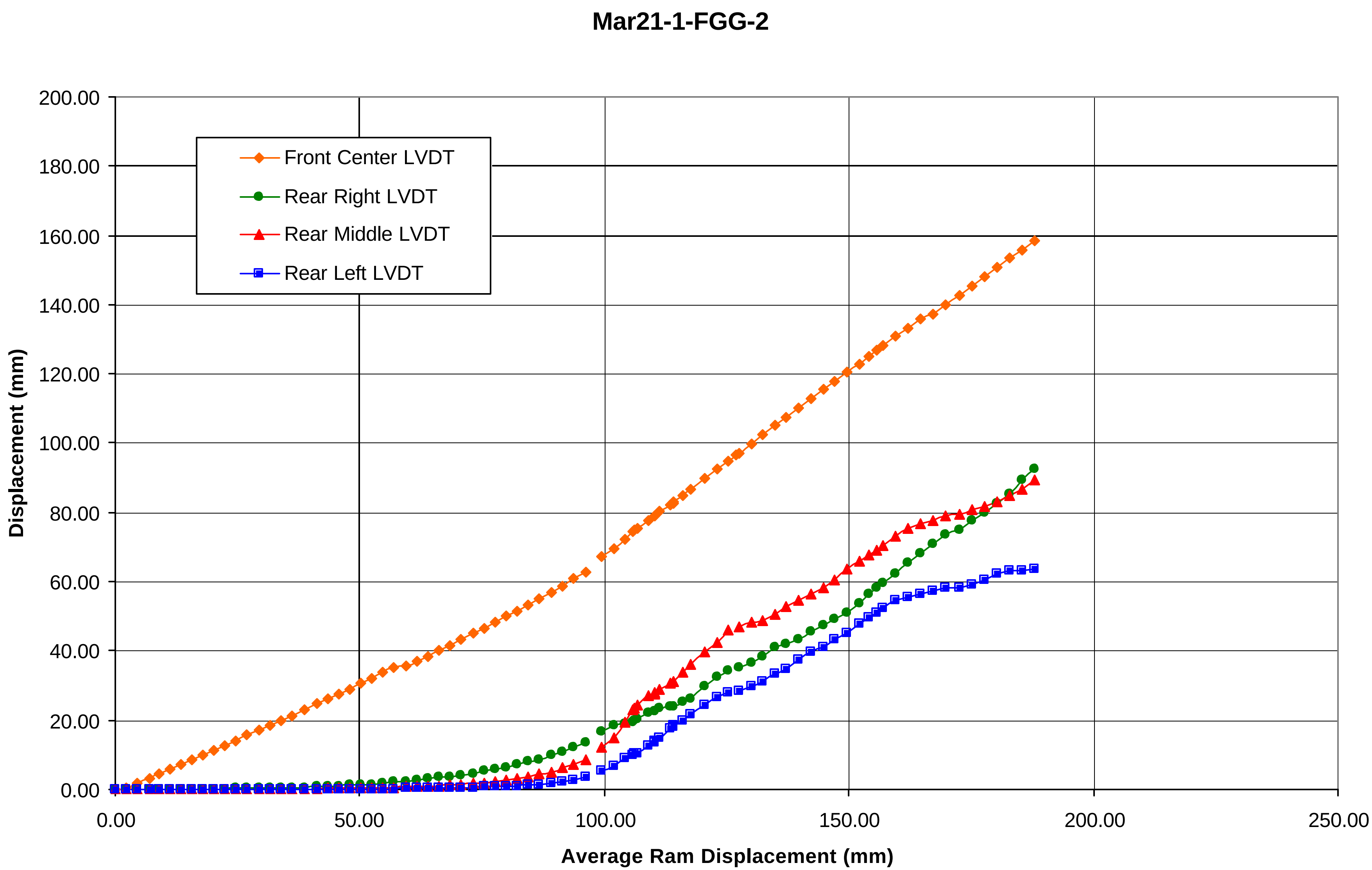
Mar07-1-FGG-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



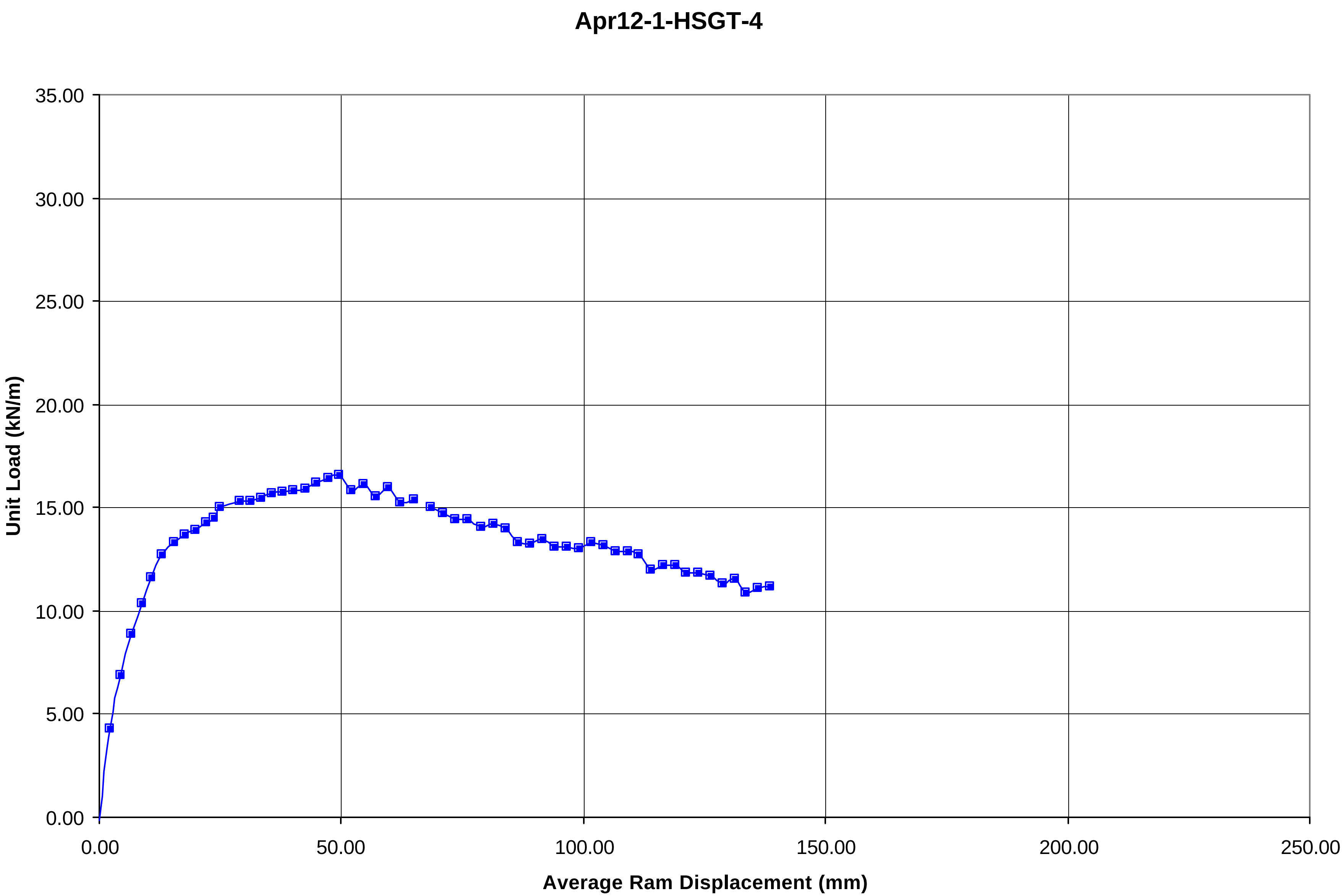
Mar07-1-FGG-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

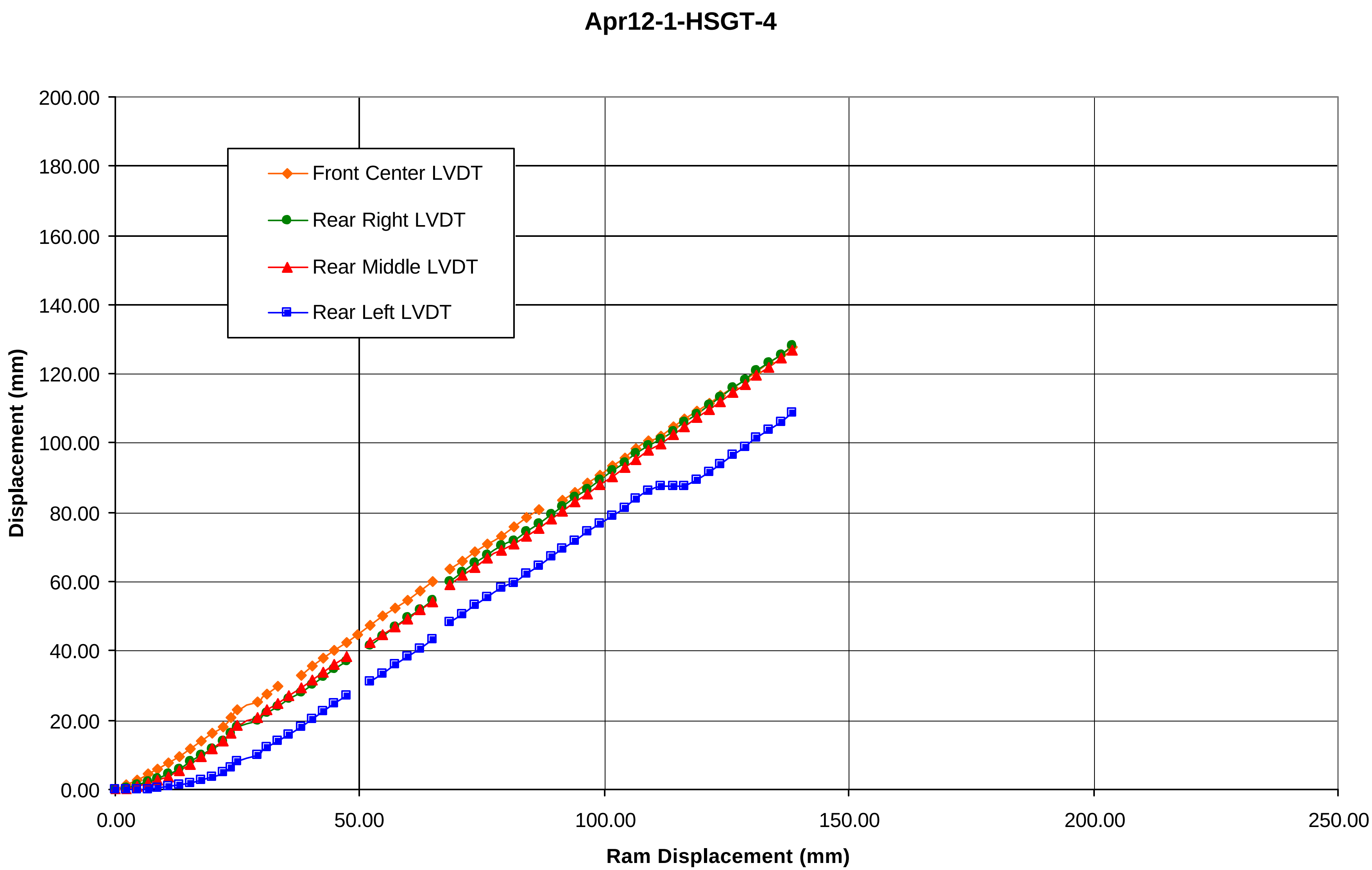


Mar21-1-FGG-2
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

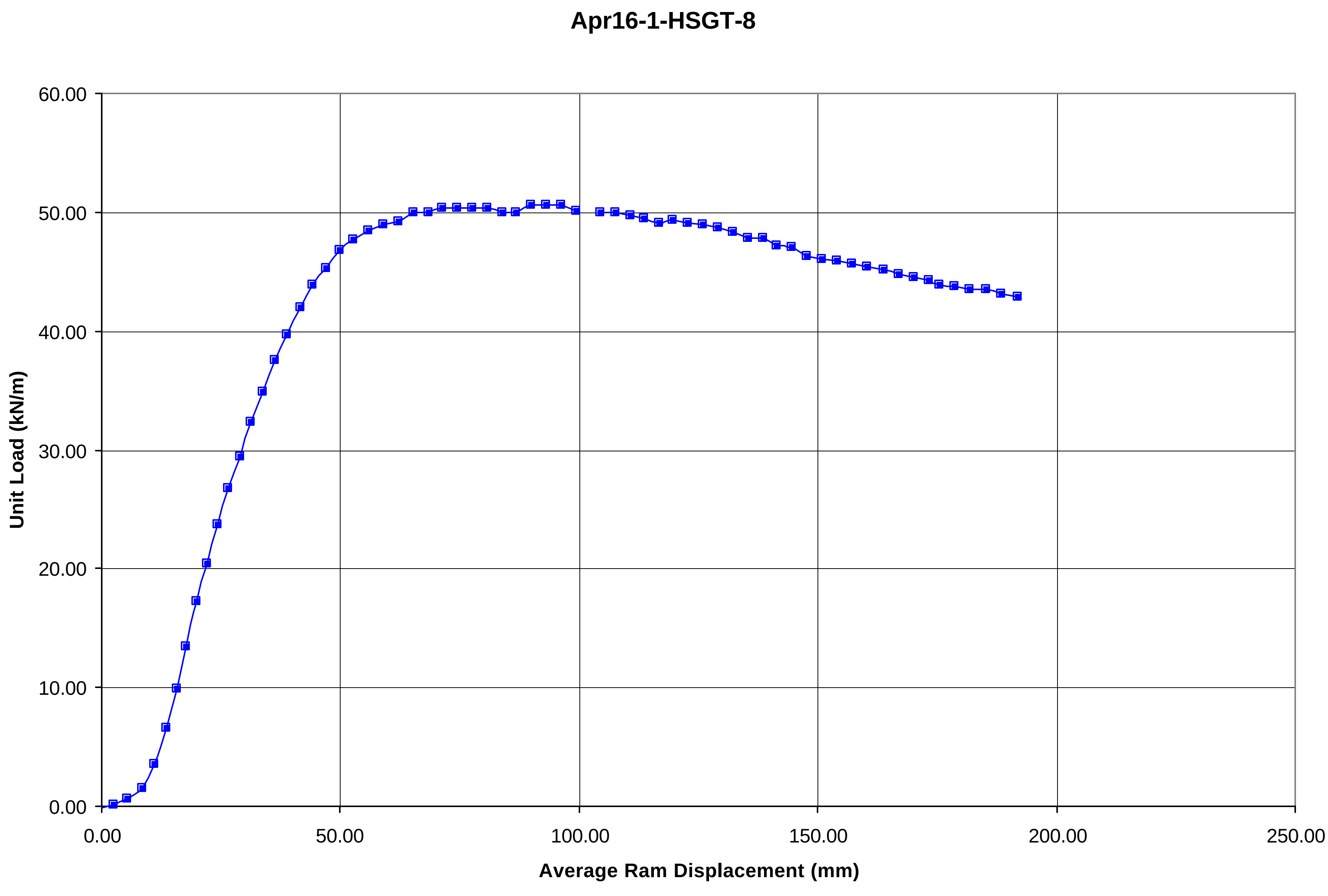


Mar21-1-FGG-2
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

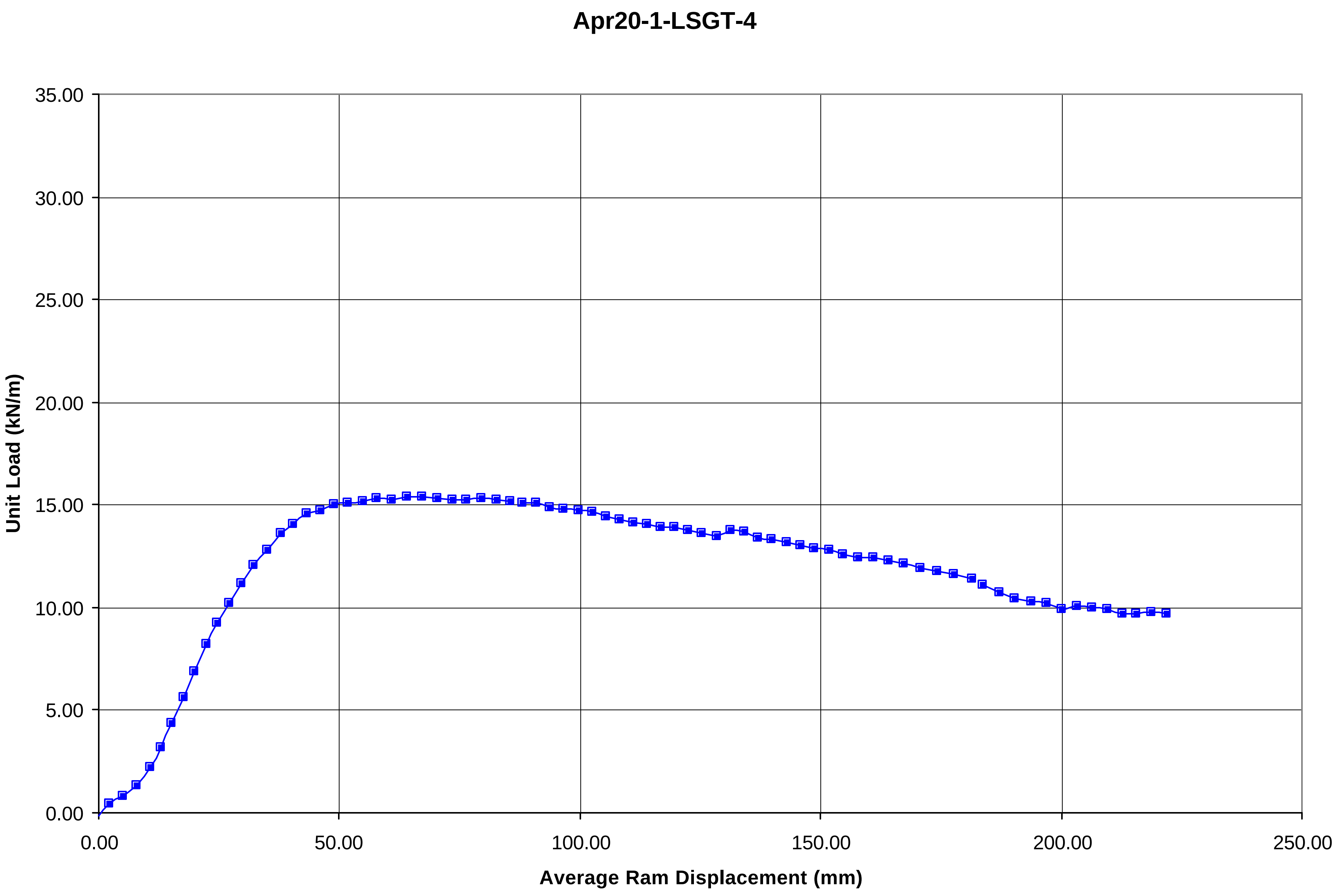

Apr12-1-HSGT-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

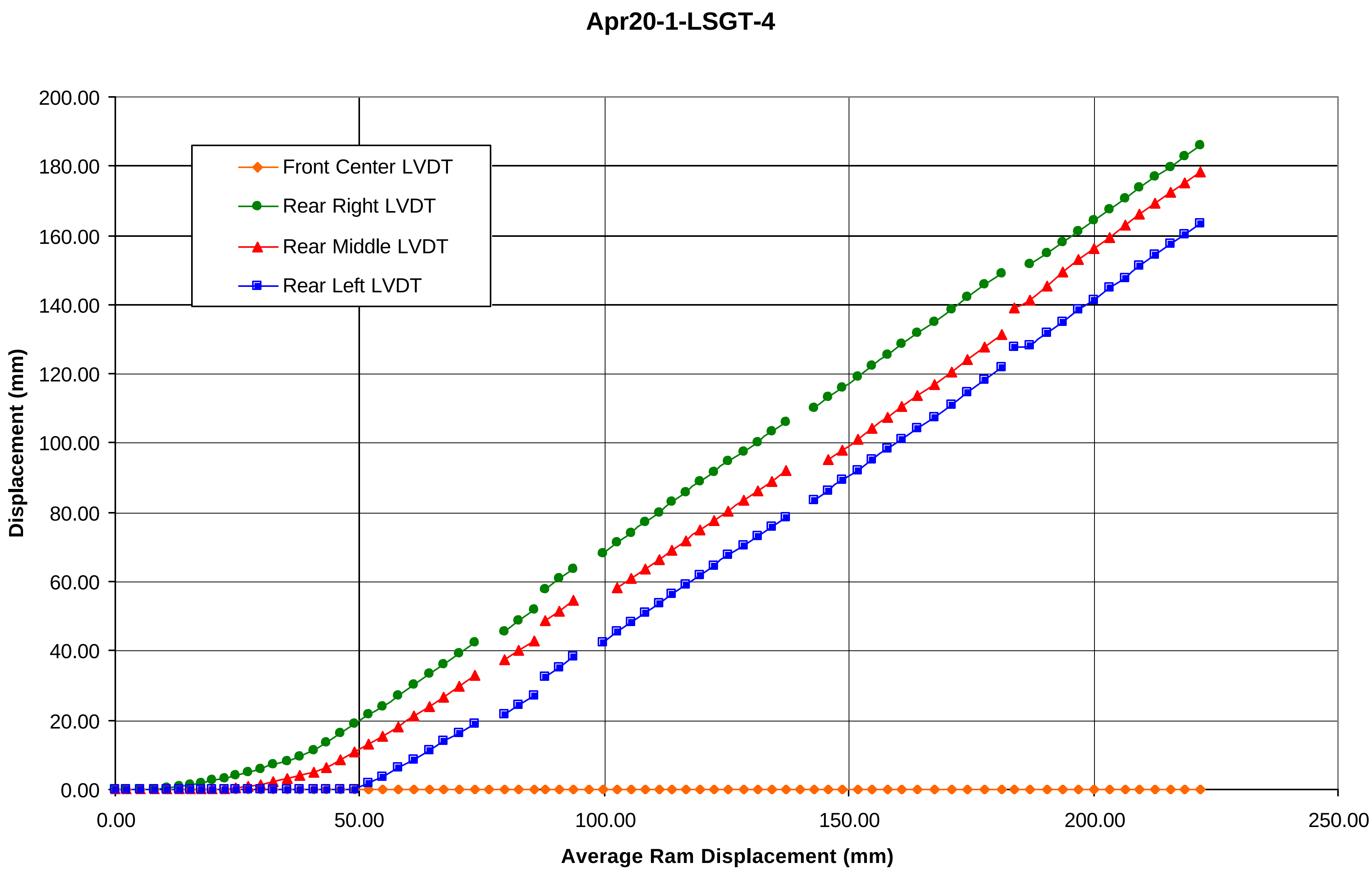


Apr12-1-HSGT-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Ram Displacement (mm)

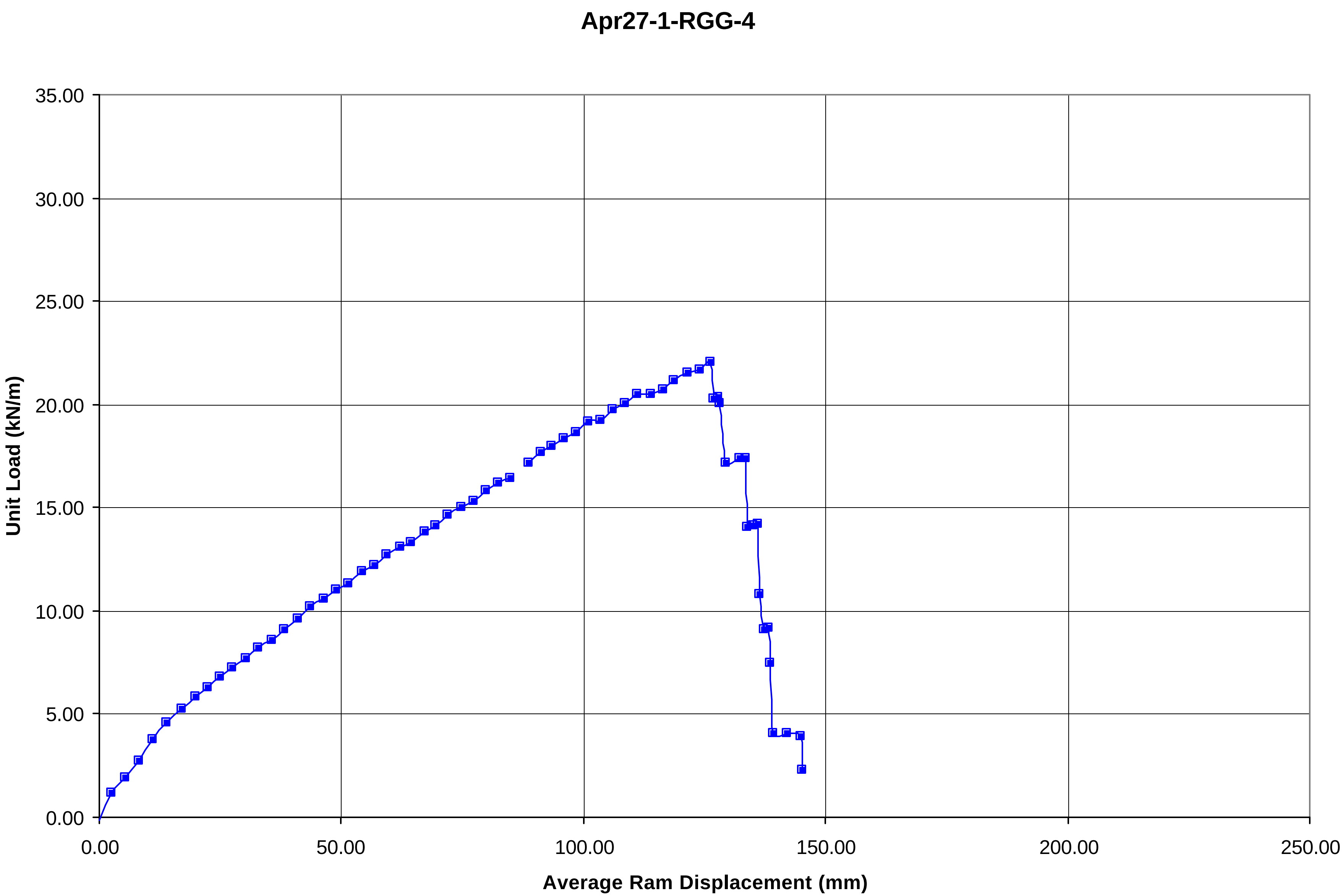

Apr16-1-HSGT-8
Unit Load (kN/m)
60.00
50.00
40.00
30.00
20.00
10.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

Apr16-1-HSGT-8
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
Displacement (mm)
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

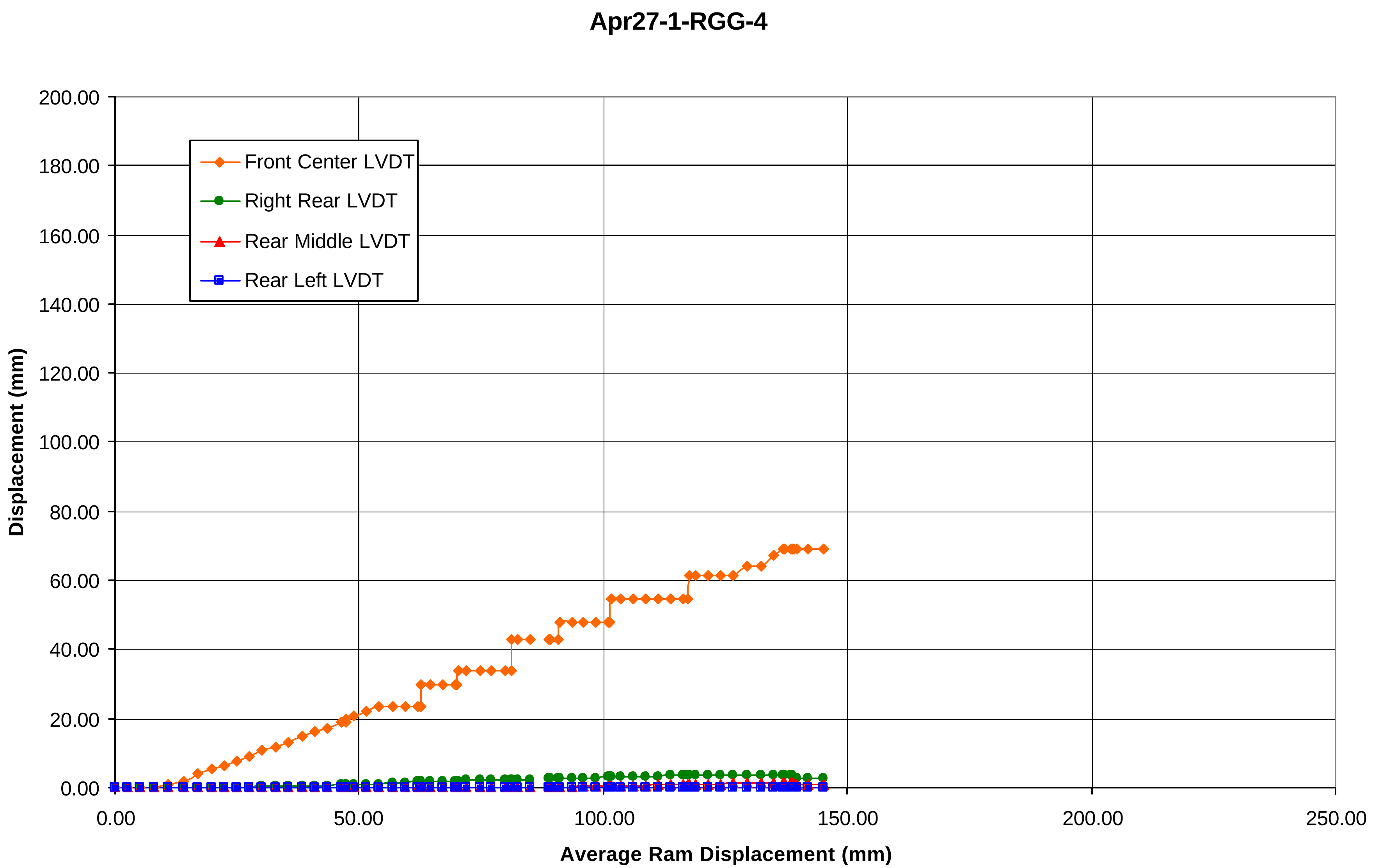

Apr20-1-LSGT-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



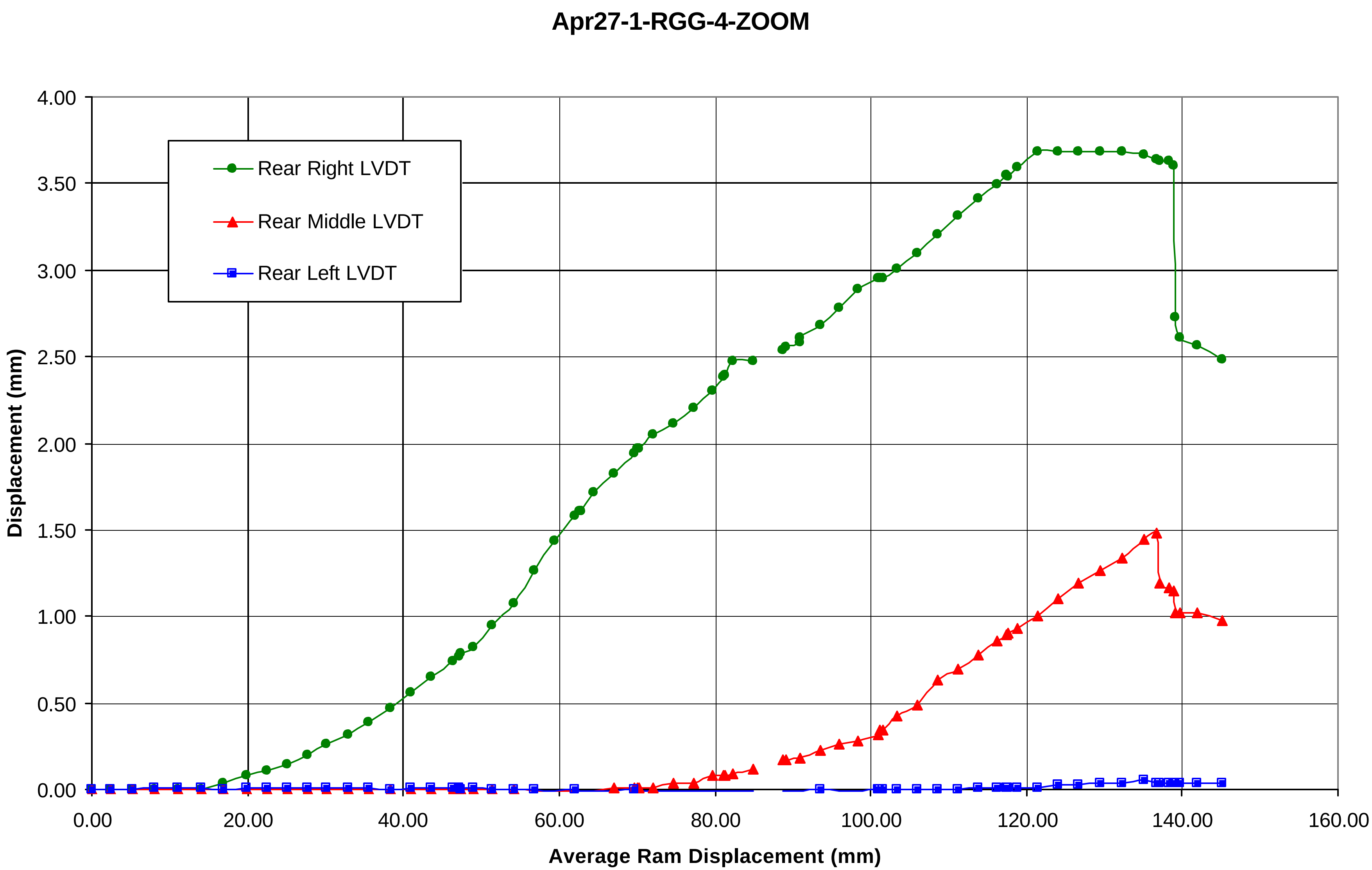
Apr20-1-LSGT-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



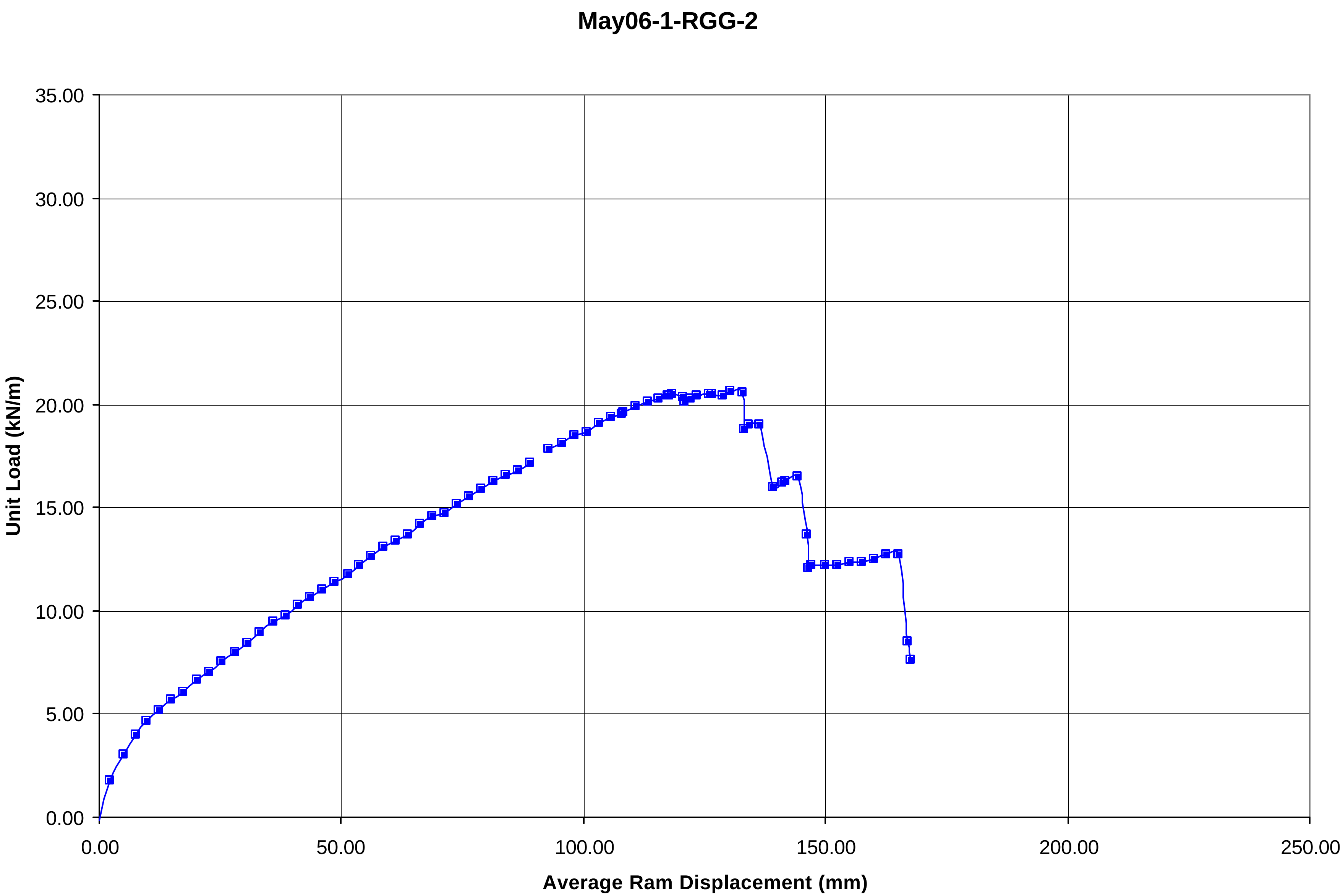
Apr27-1-RGG-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

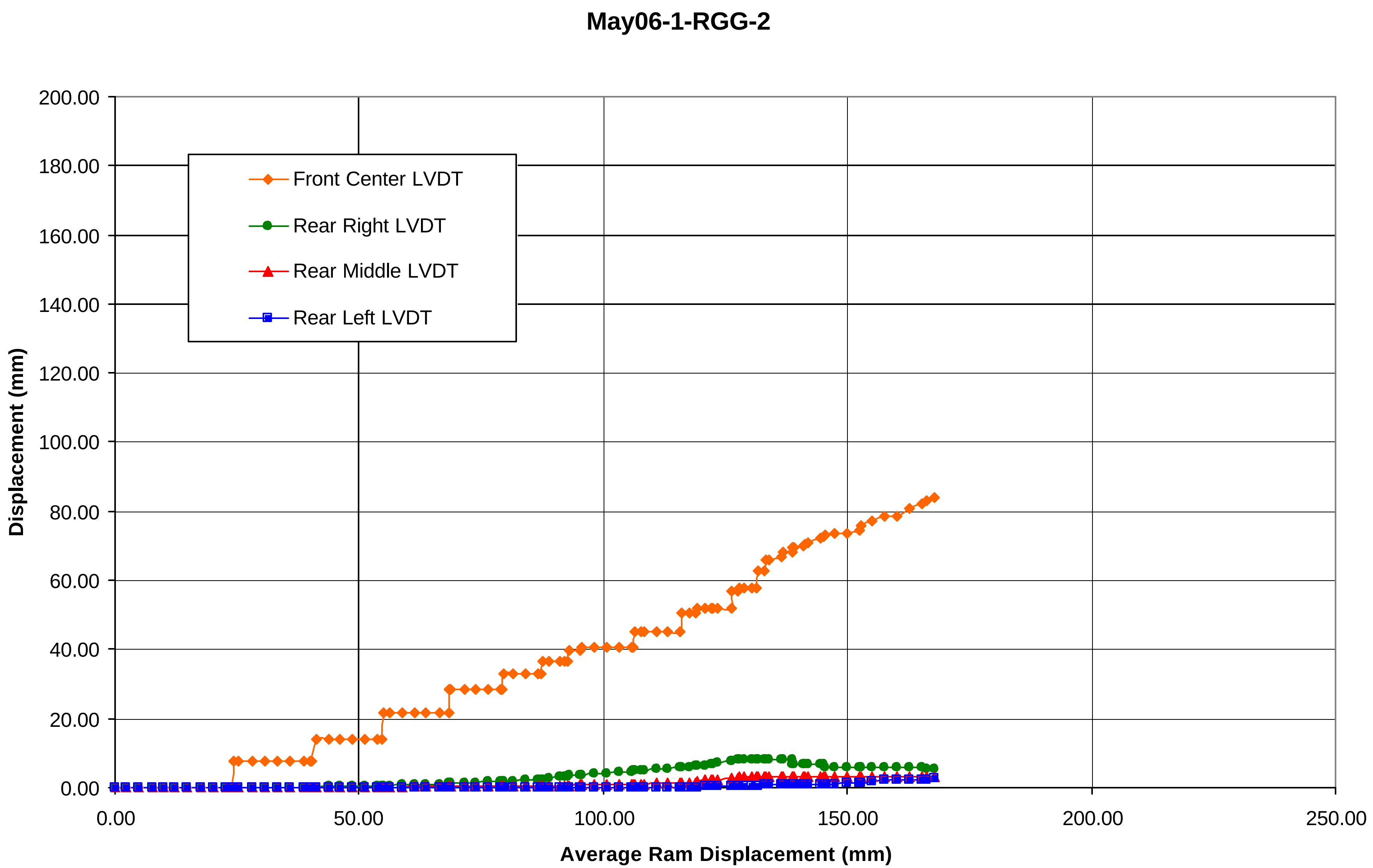


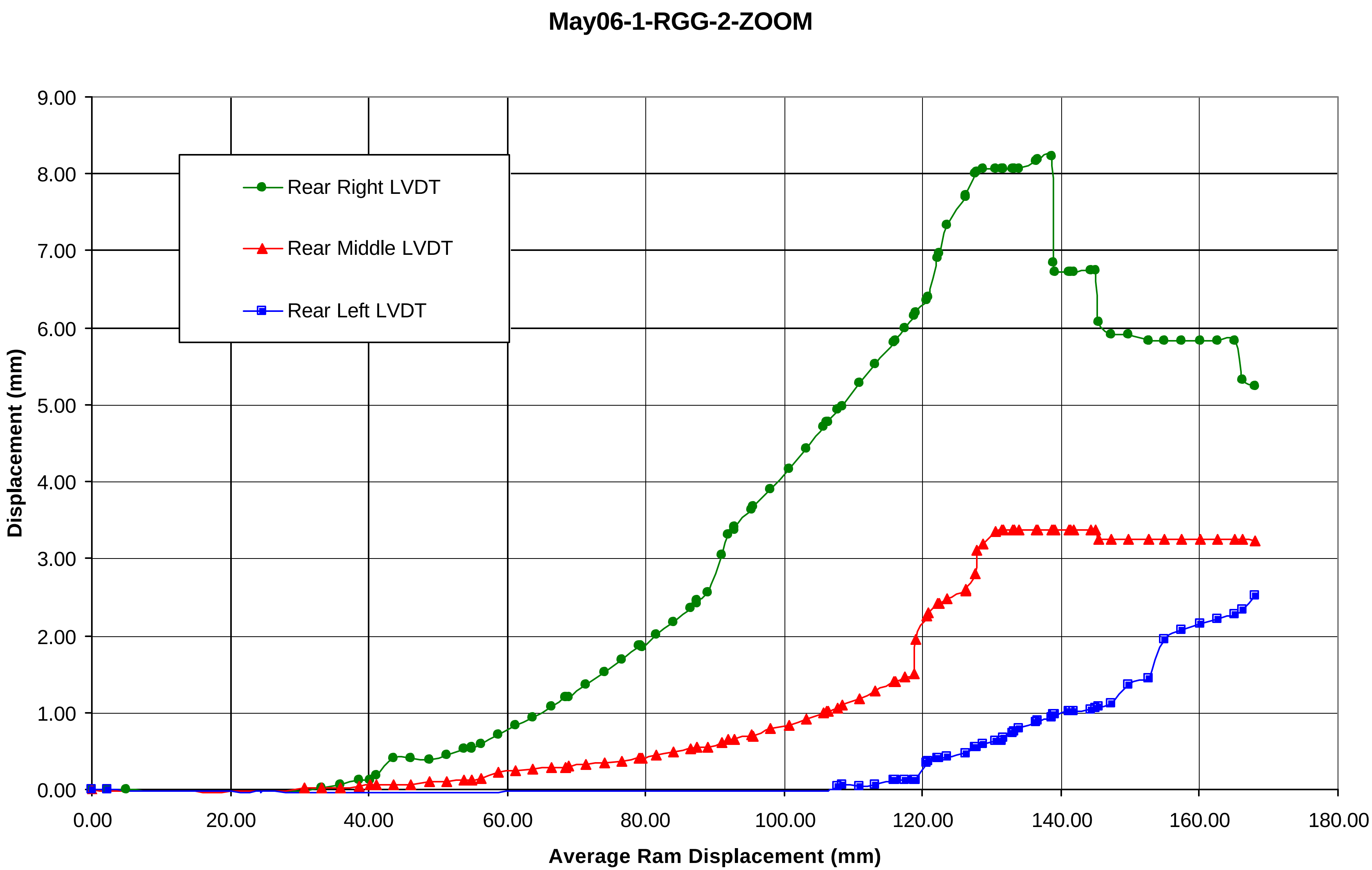
Apr27-1-RGG-4
Front Center LVDT
Right Rear LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



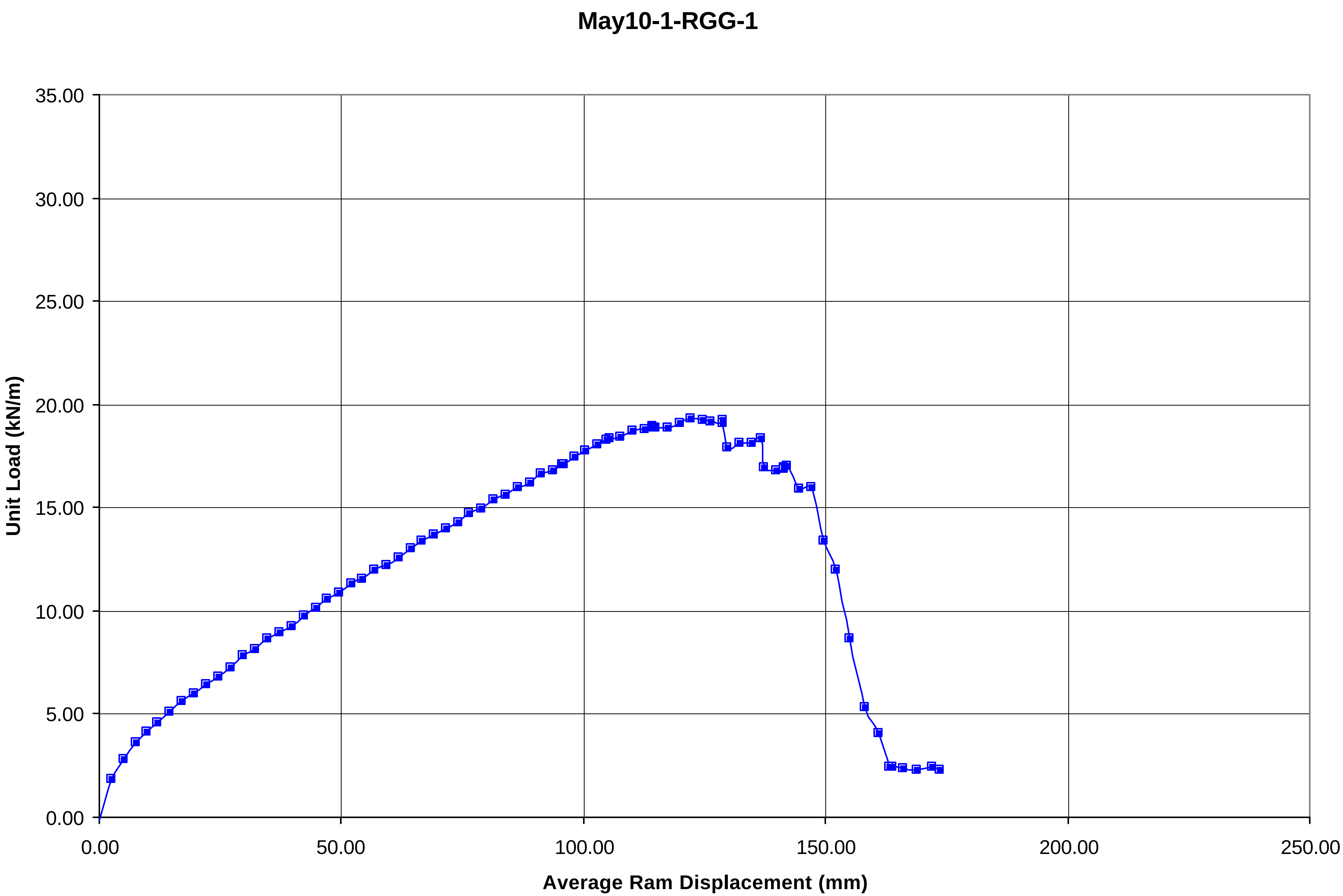
Apr27-1-RGG-4-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
Average Ram Displacement (mm)

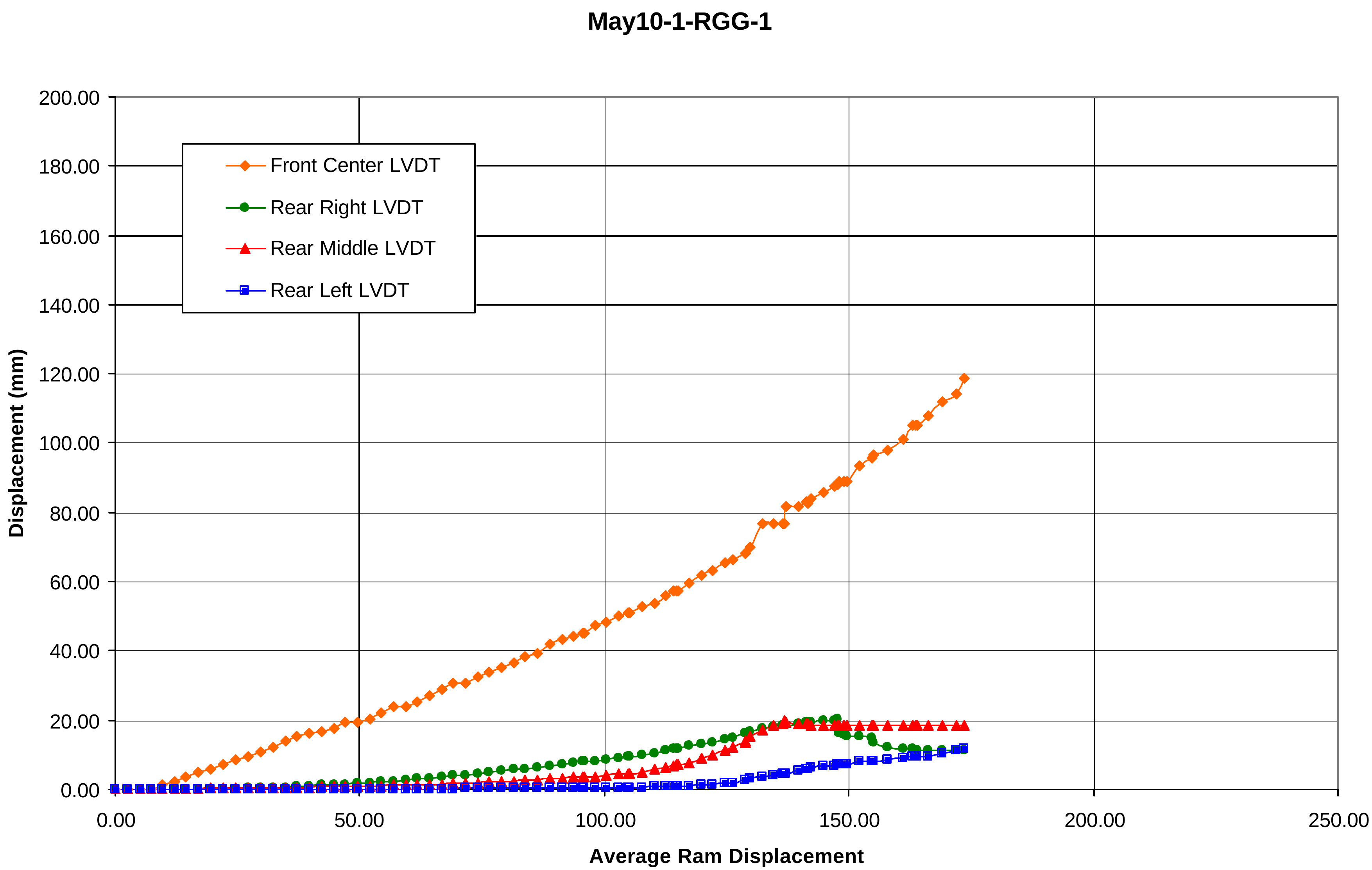

May06-1-RGG-2
Unit Load (kN/m)
Average Ram Displacement (mm)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00


May06-1-RGG-2
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

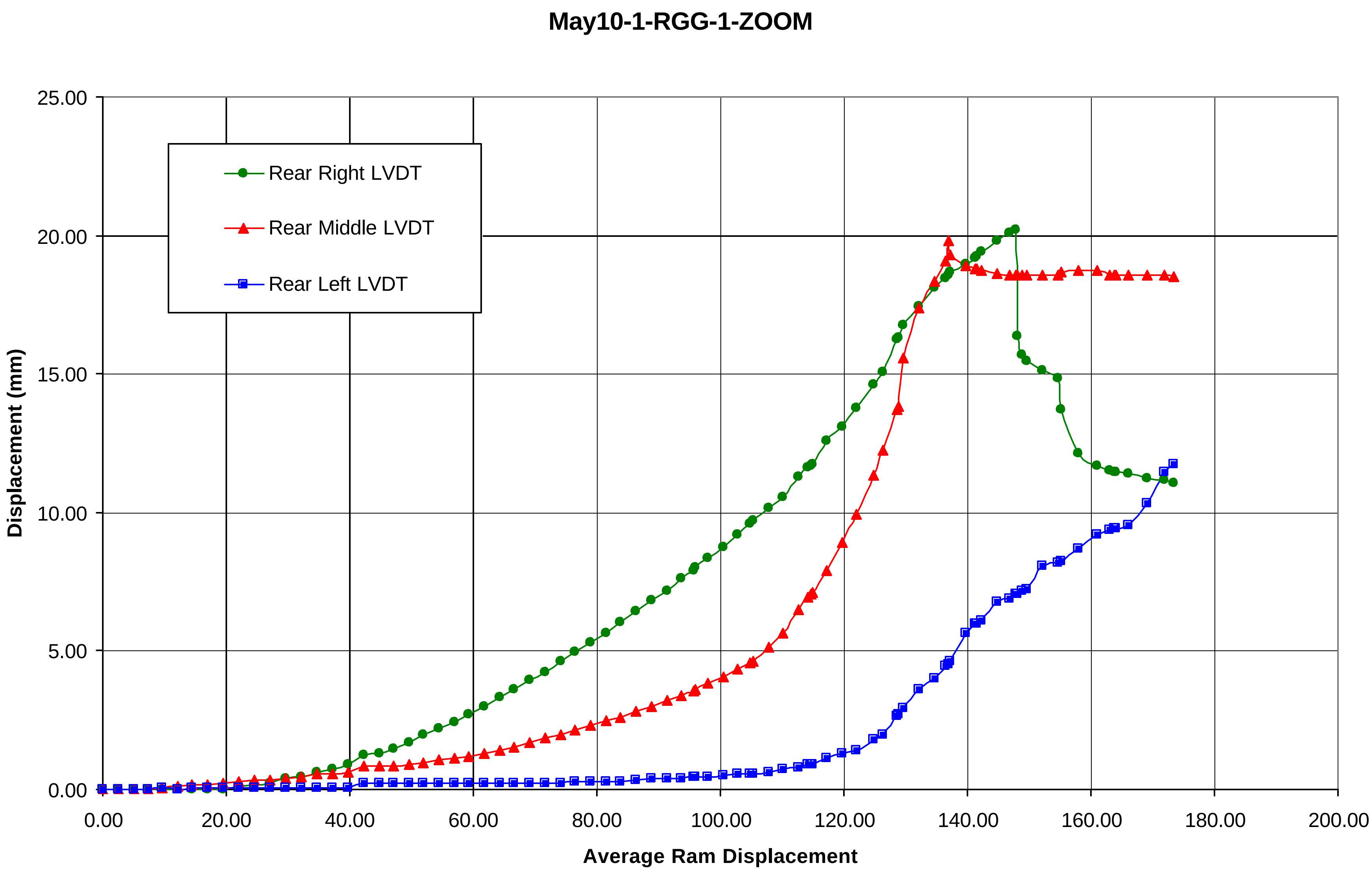


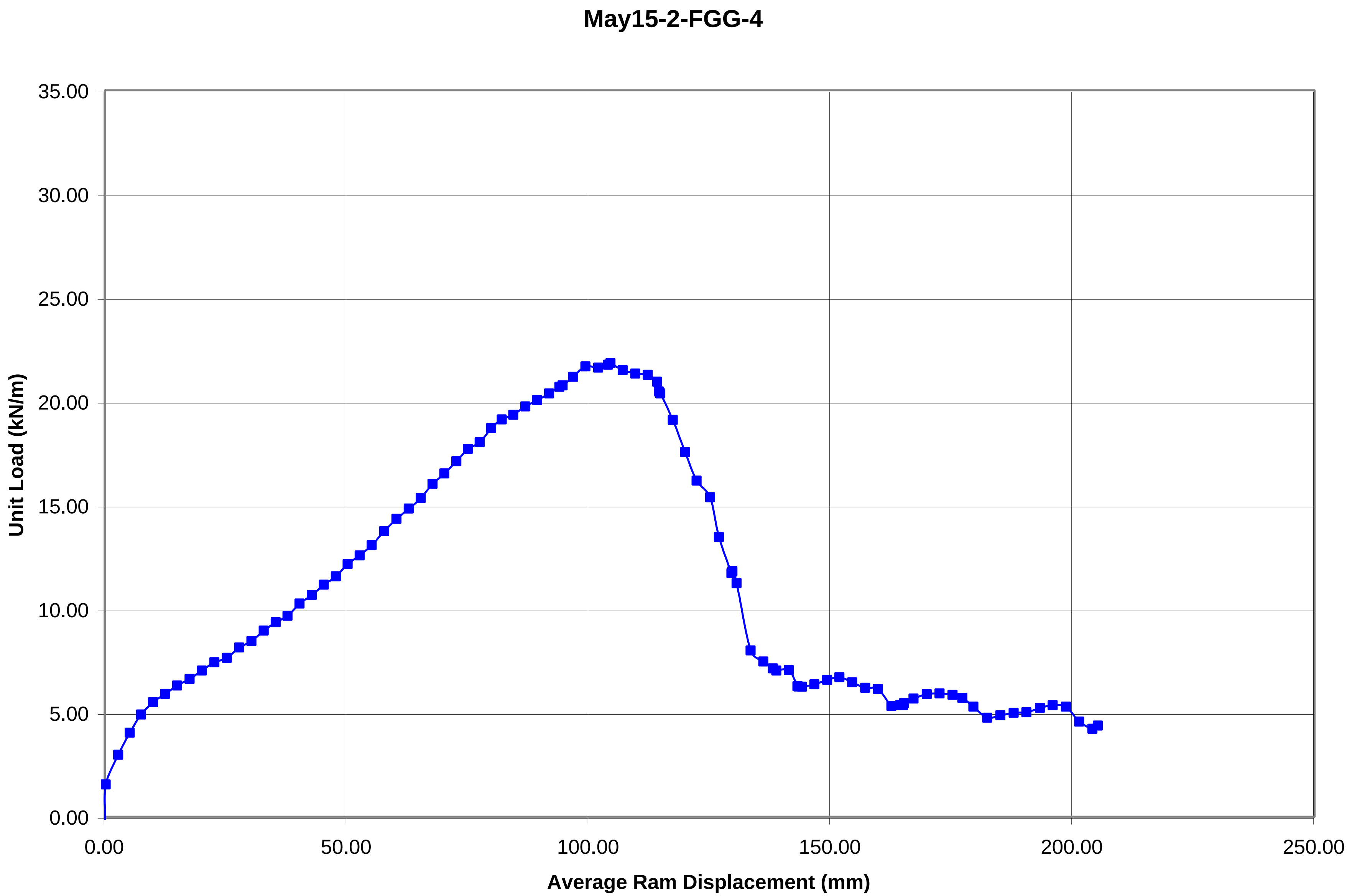
May06-1-RGG-2-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
Average Ram Displacement (mm)
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00

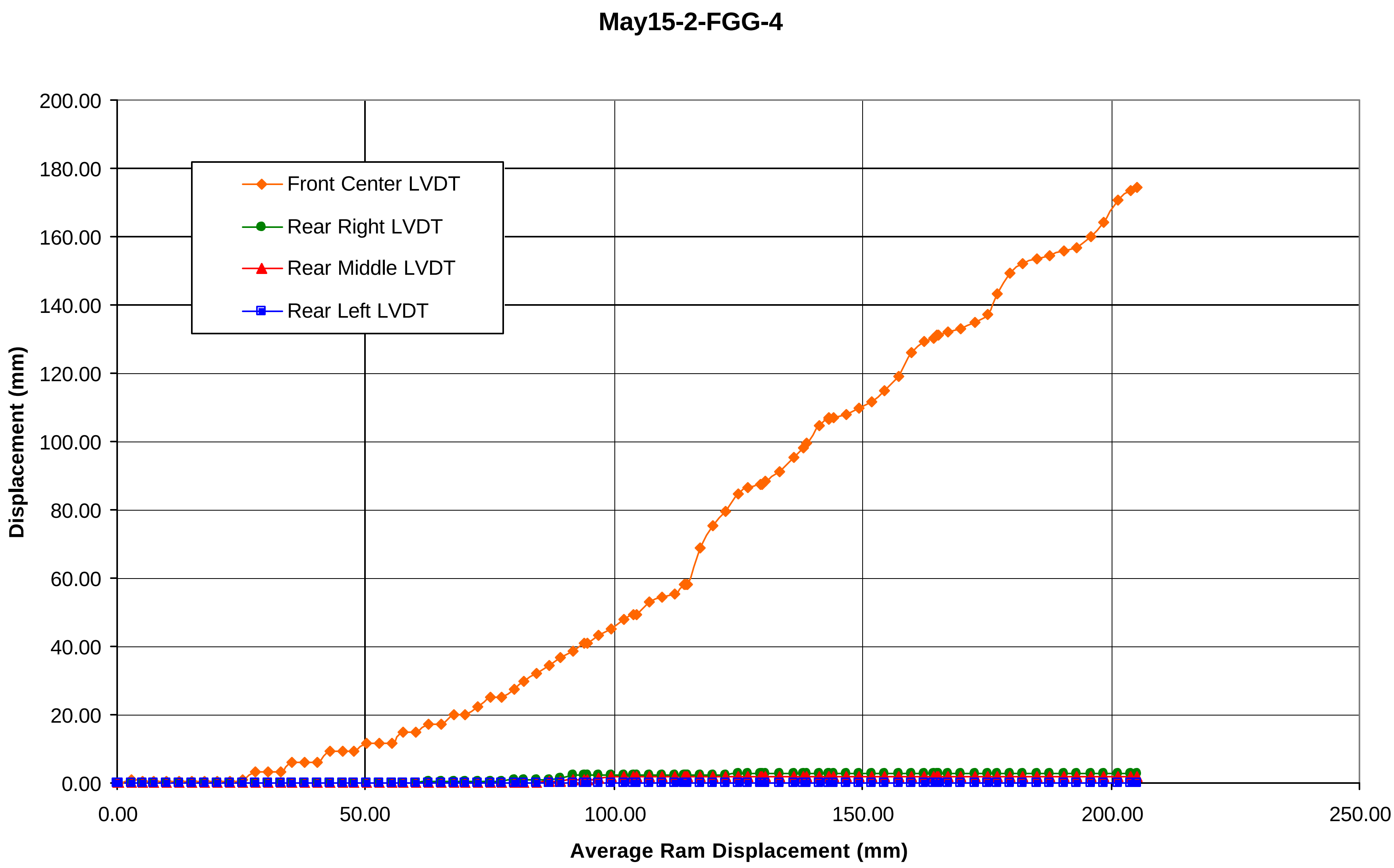

May10-1-RGG-1
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

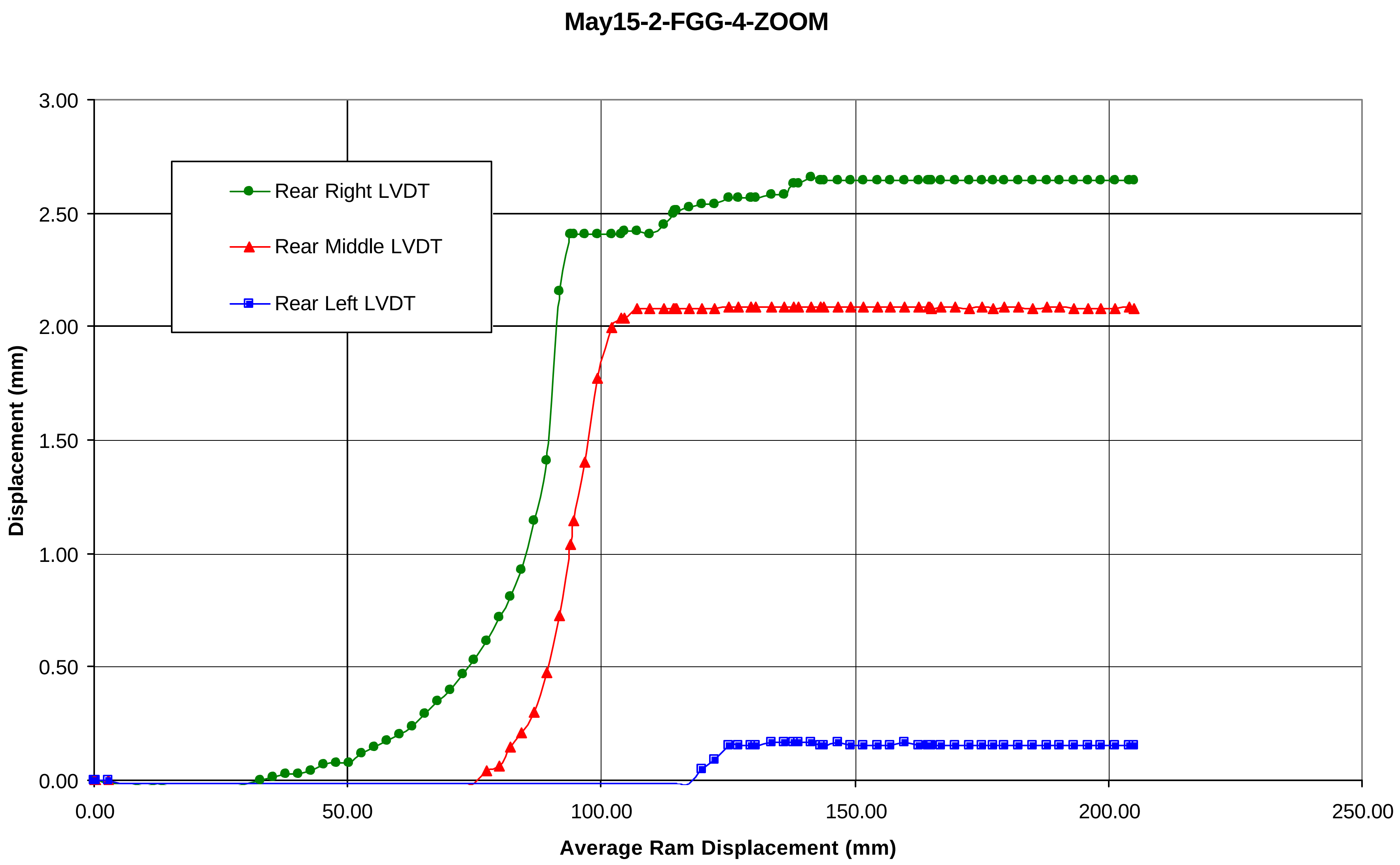


May10-1-RGG-1
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement

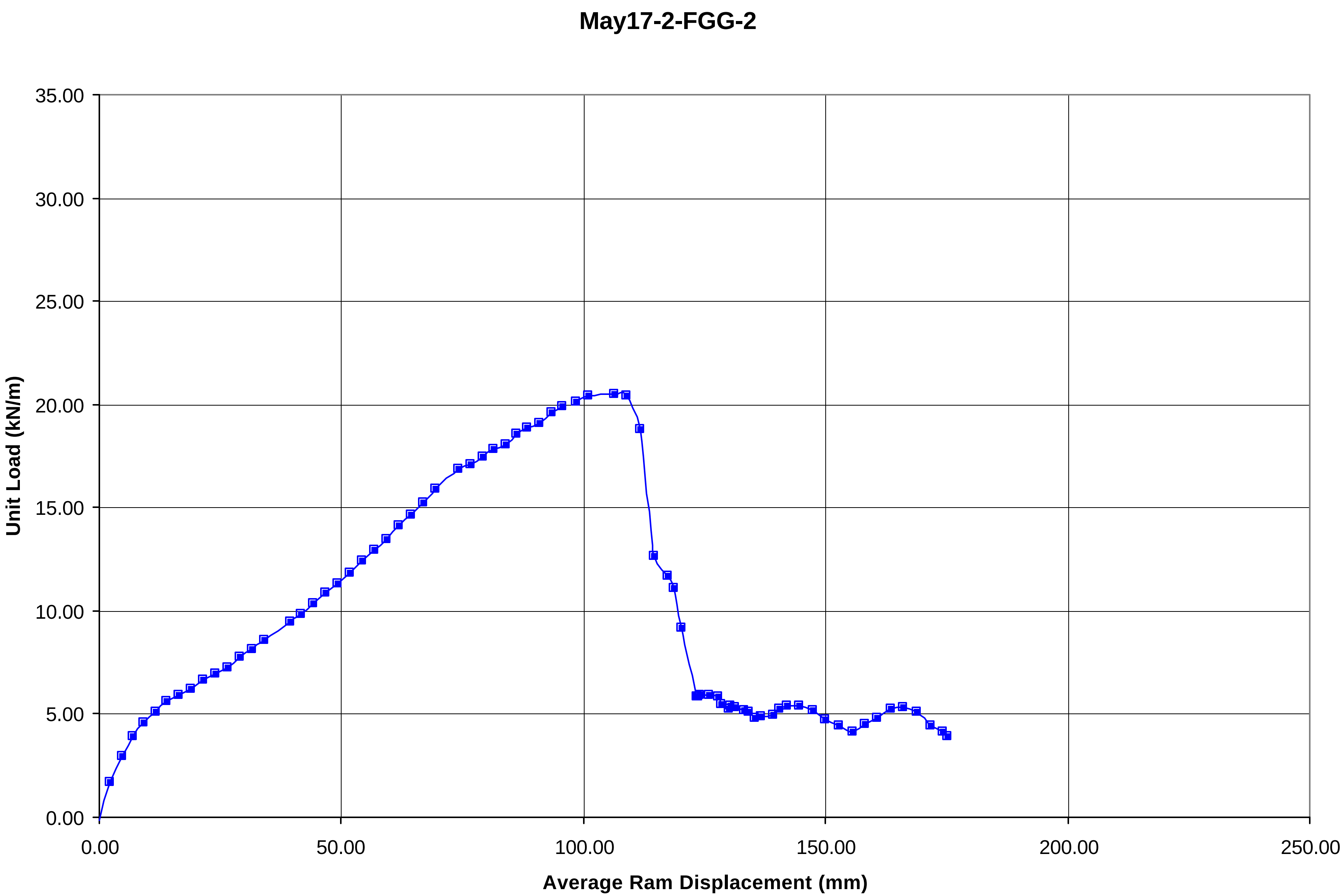

May10-1-RGG-1-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
Average Ram Displacement
0.00
5.00
10.00
15.00
20.00
25.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00

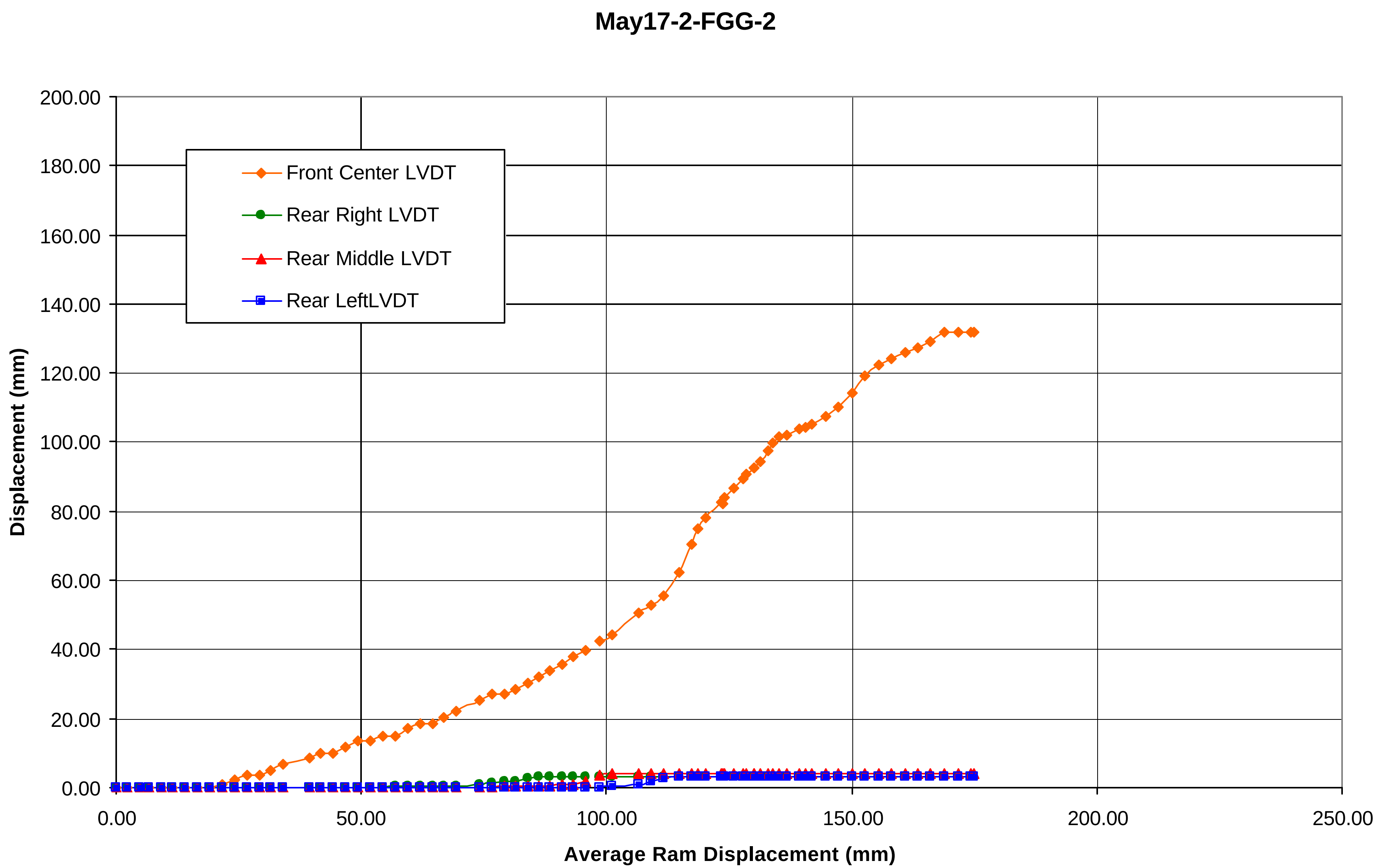

May15-2-FGG-4
Unit Load (kN/m)
Average Ram Displacement (mm)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00

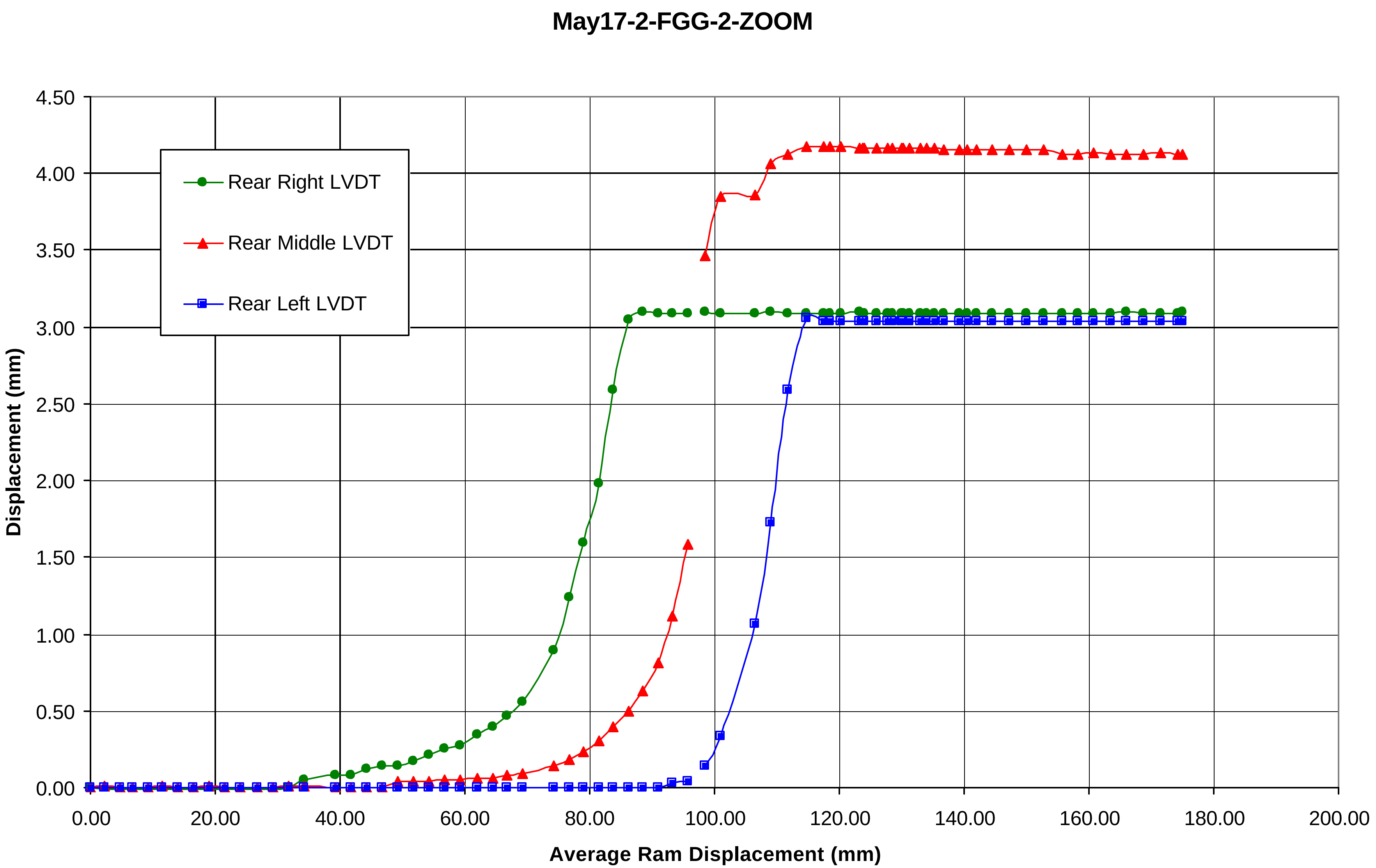

May15-2-FGG-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



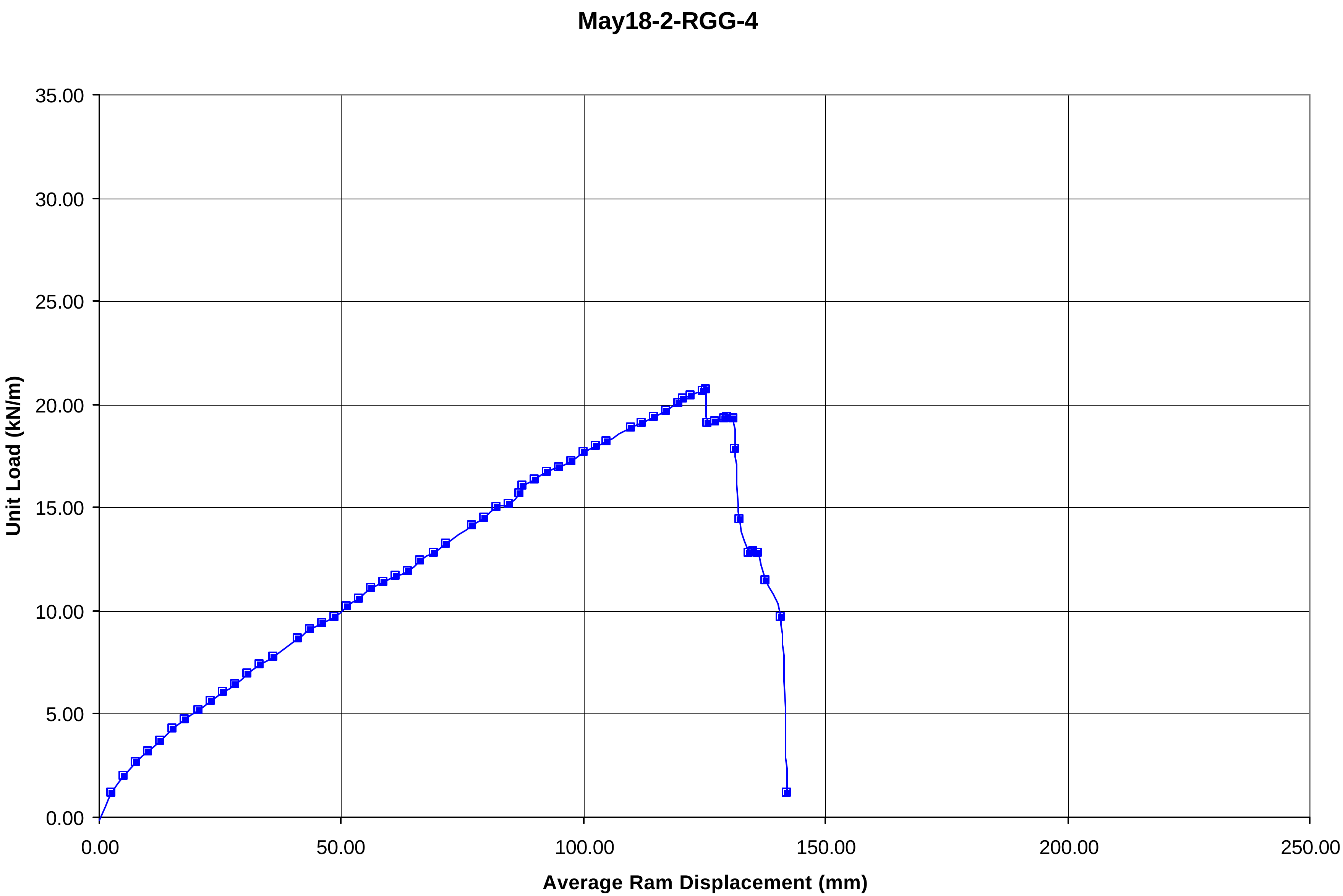
May15-2-FGG-4-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

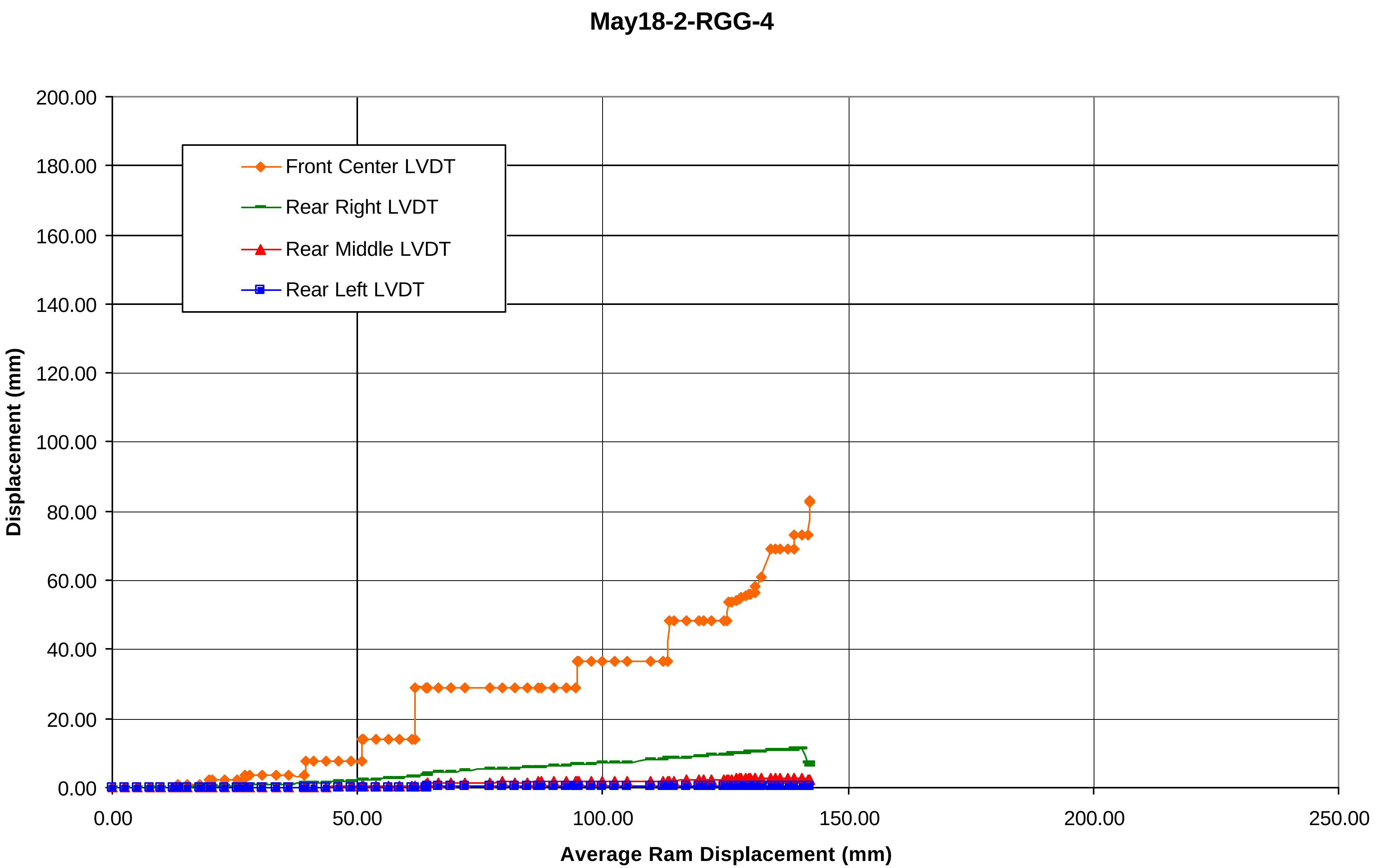

May17-2-FGG-2
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)


May17-2-FGG-2
Displacement (mm)
Average Ram Displacement (mm)
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear LeftLVDT
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00

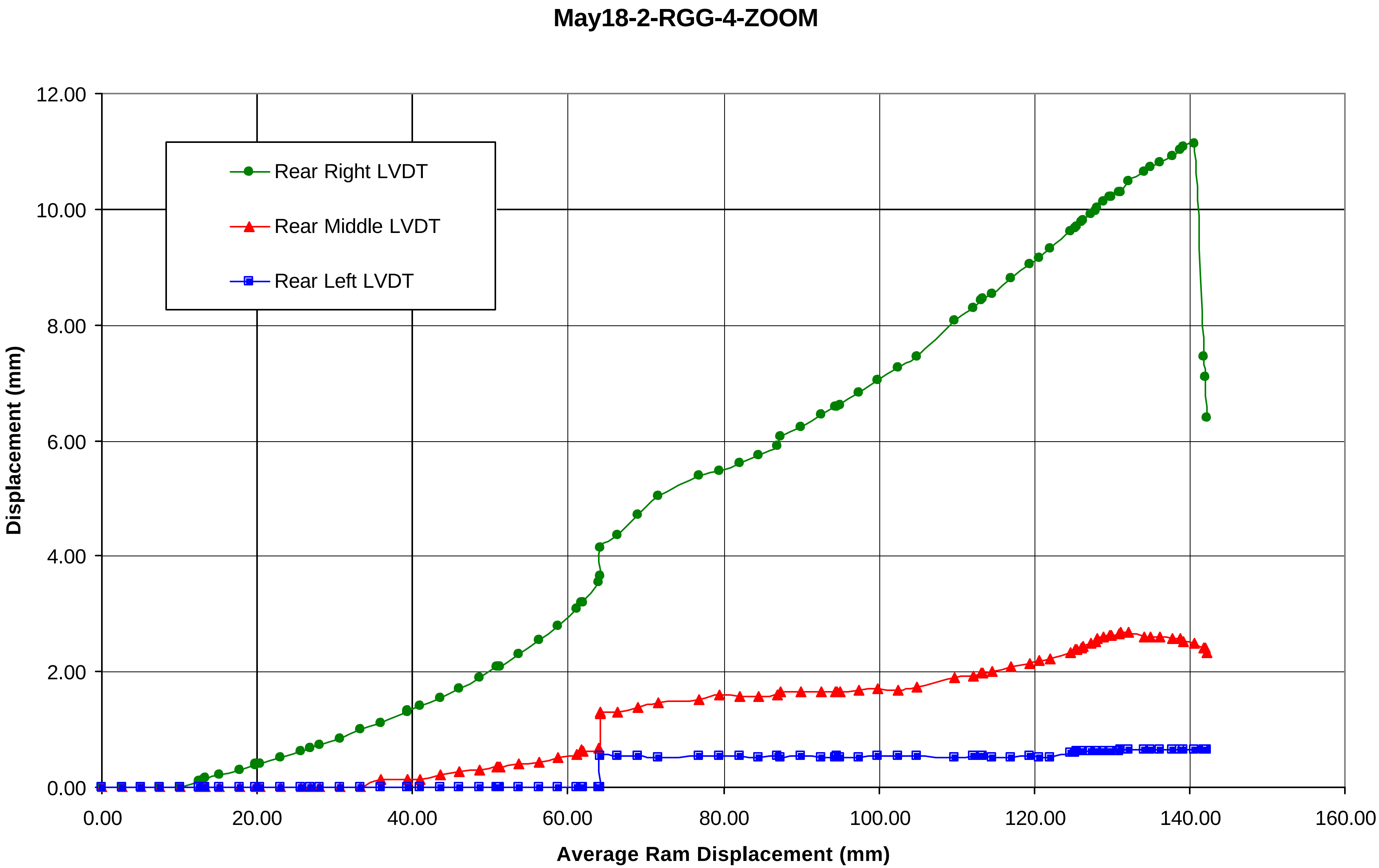


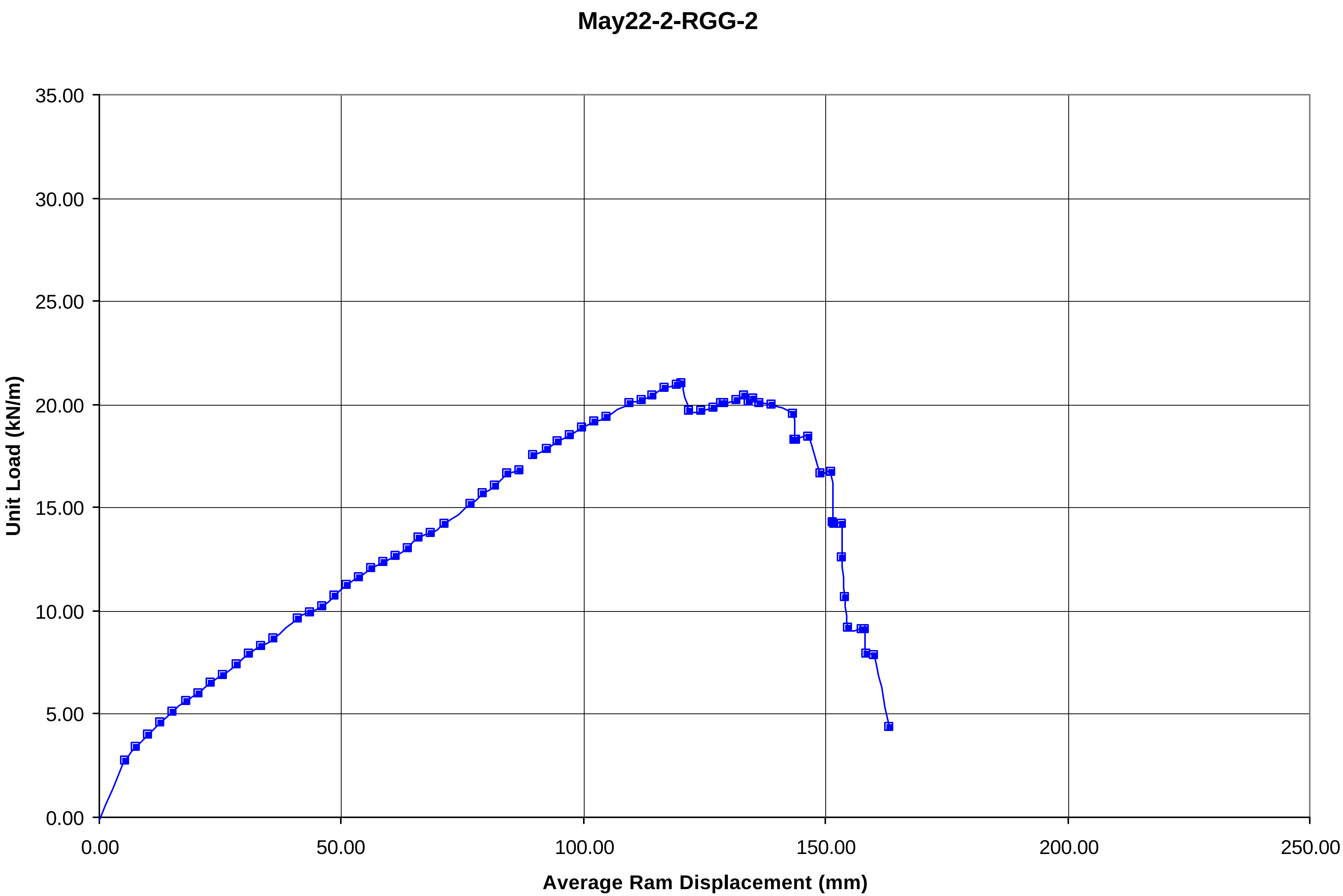
May17-2-FGG-2-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
Average Ram Displacement (mm)
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00

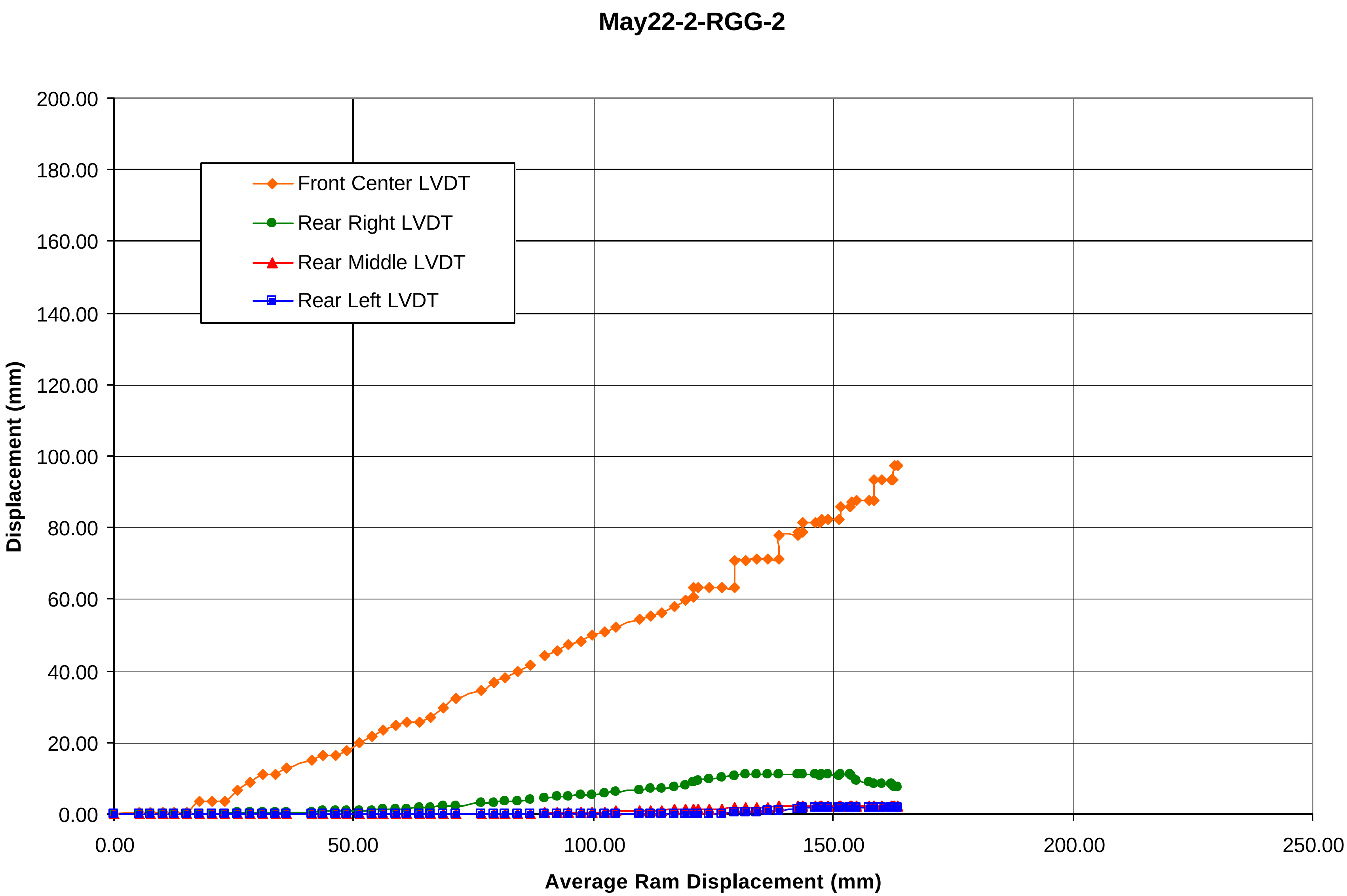

May18-2-RGG-4
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Unit Load (kN/m)
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



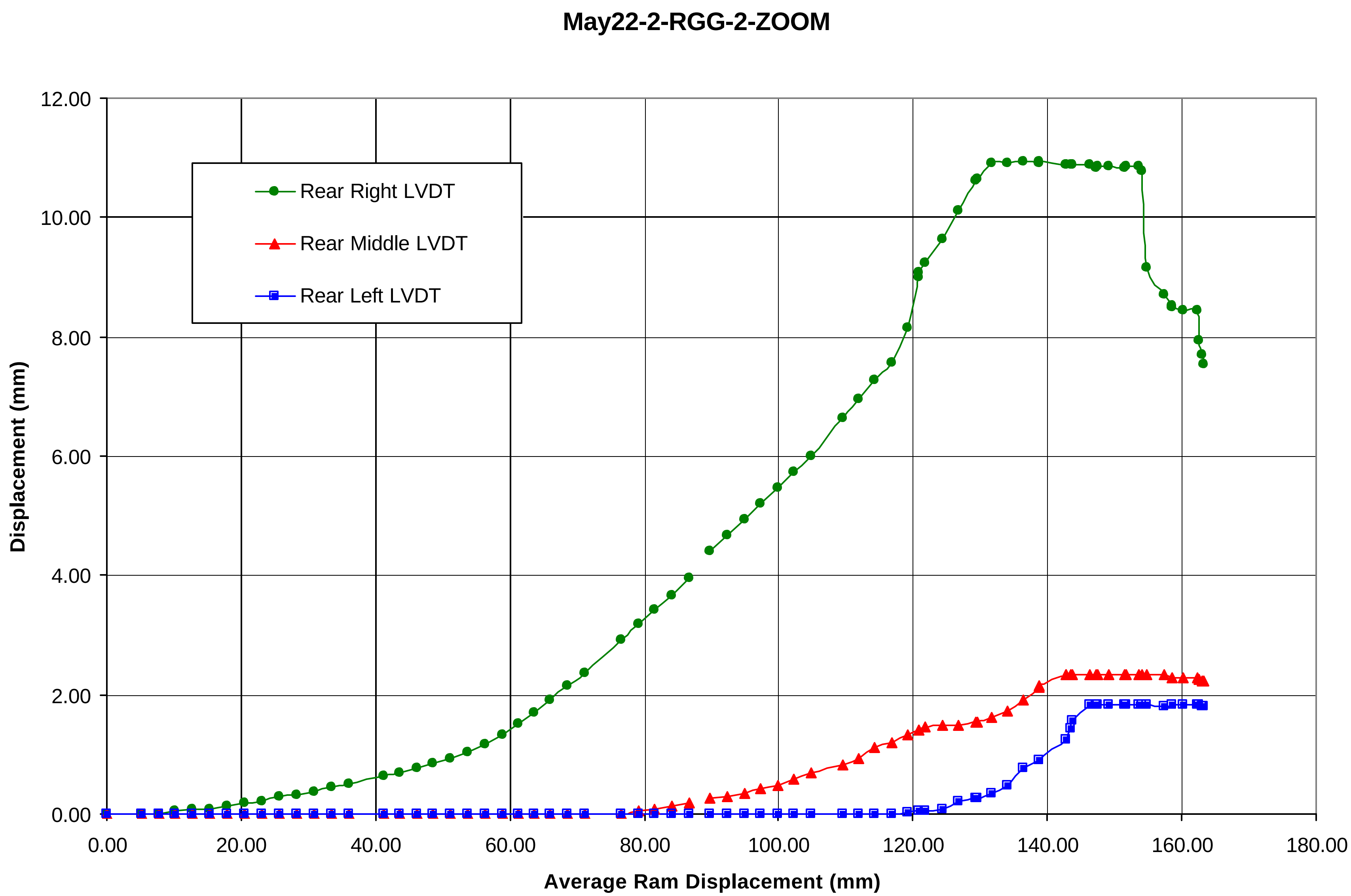
May18-2-RGG-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

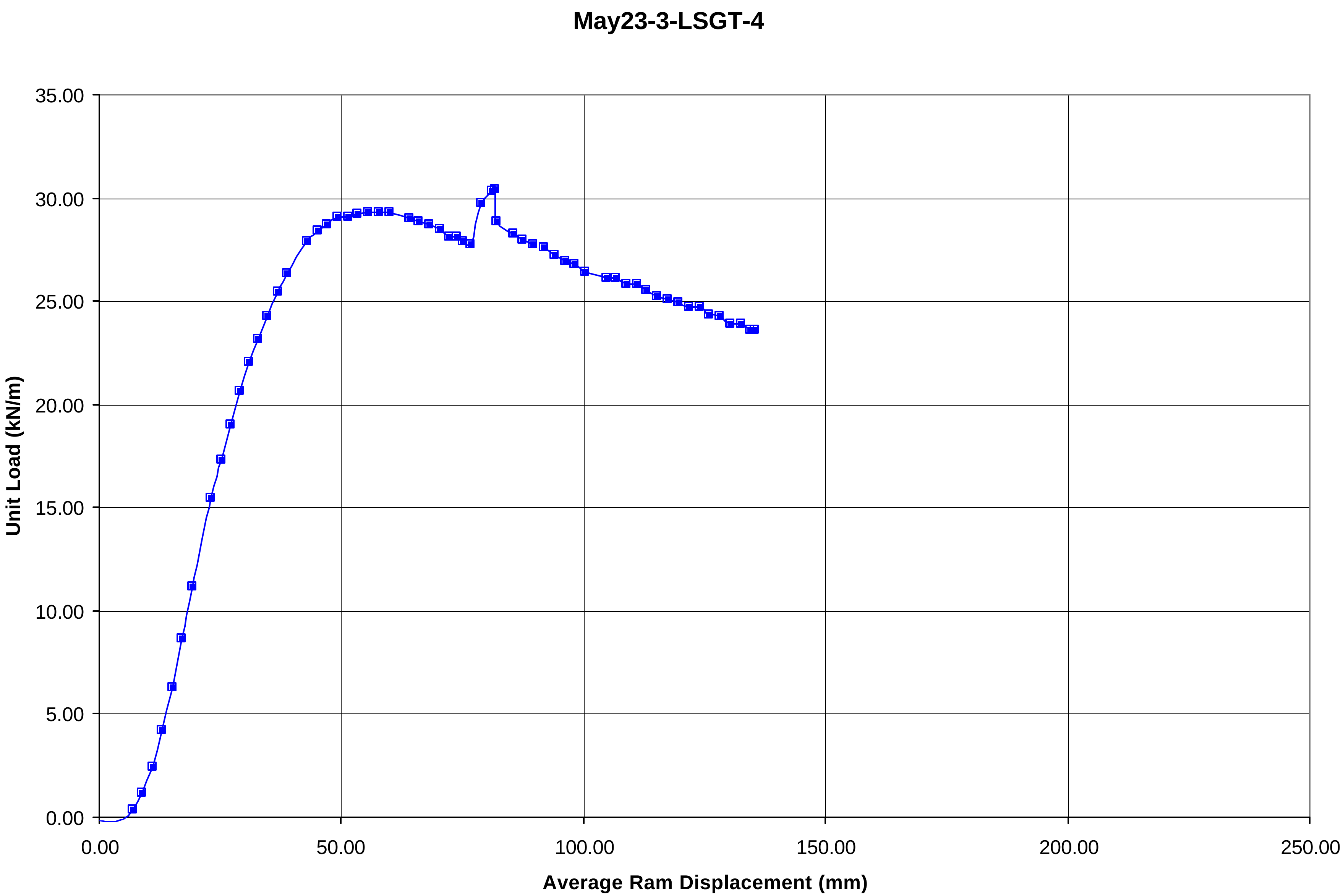

May18-2-RGG-4-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
Average Ram Displacement (mm)


May22-2-RGG-2
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

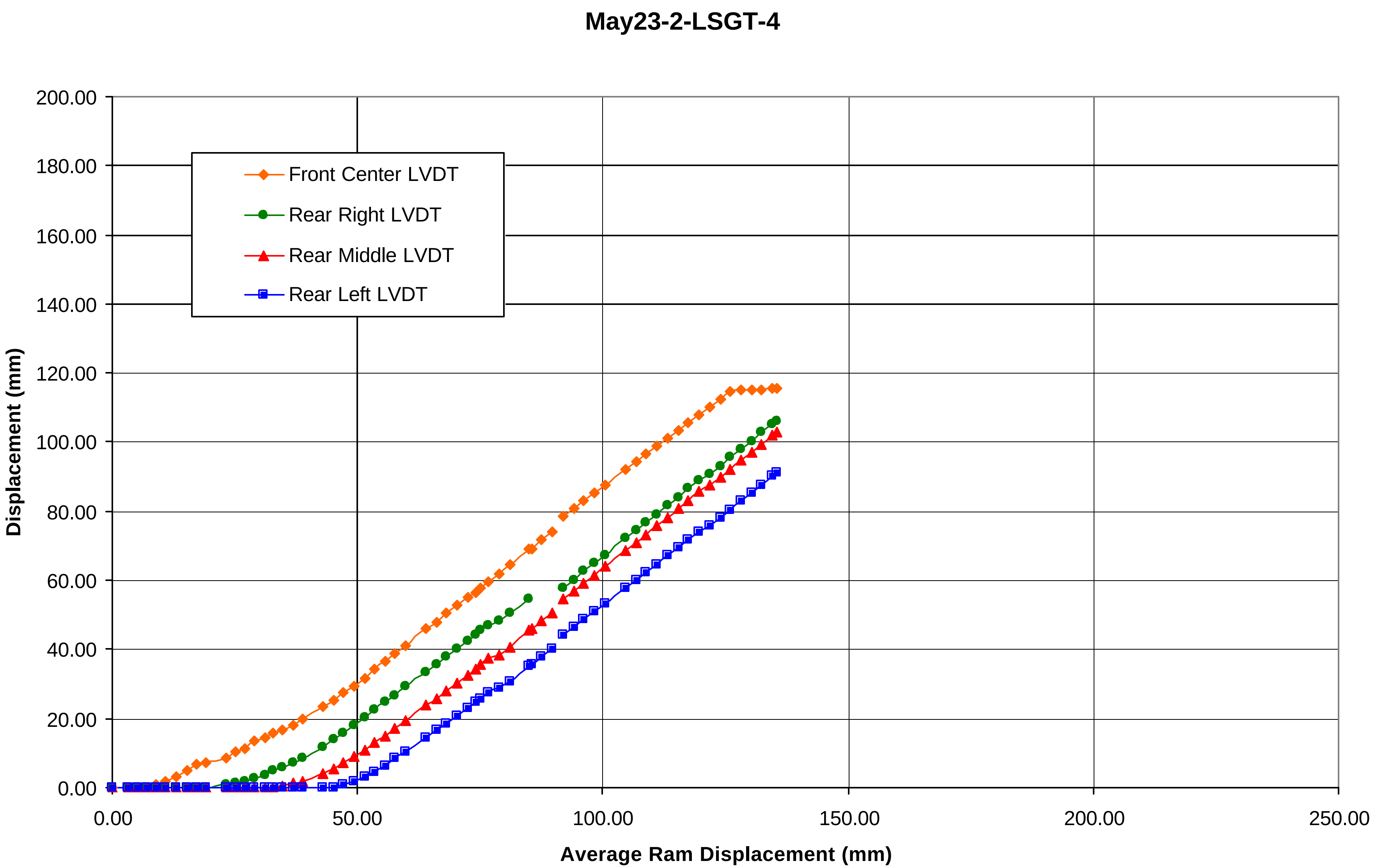


May22-2-RGG-2
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

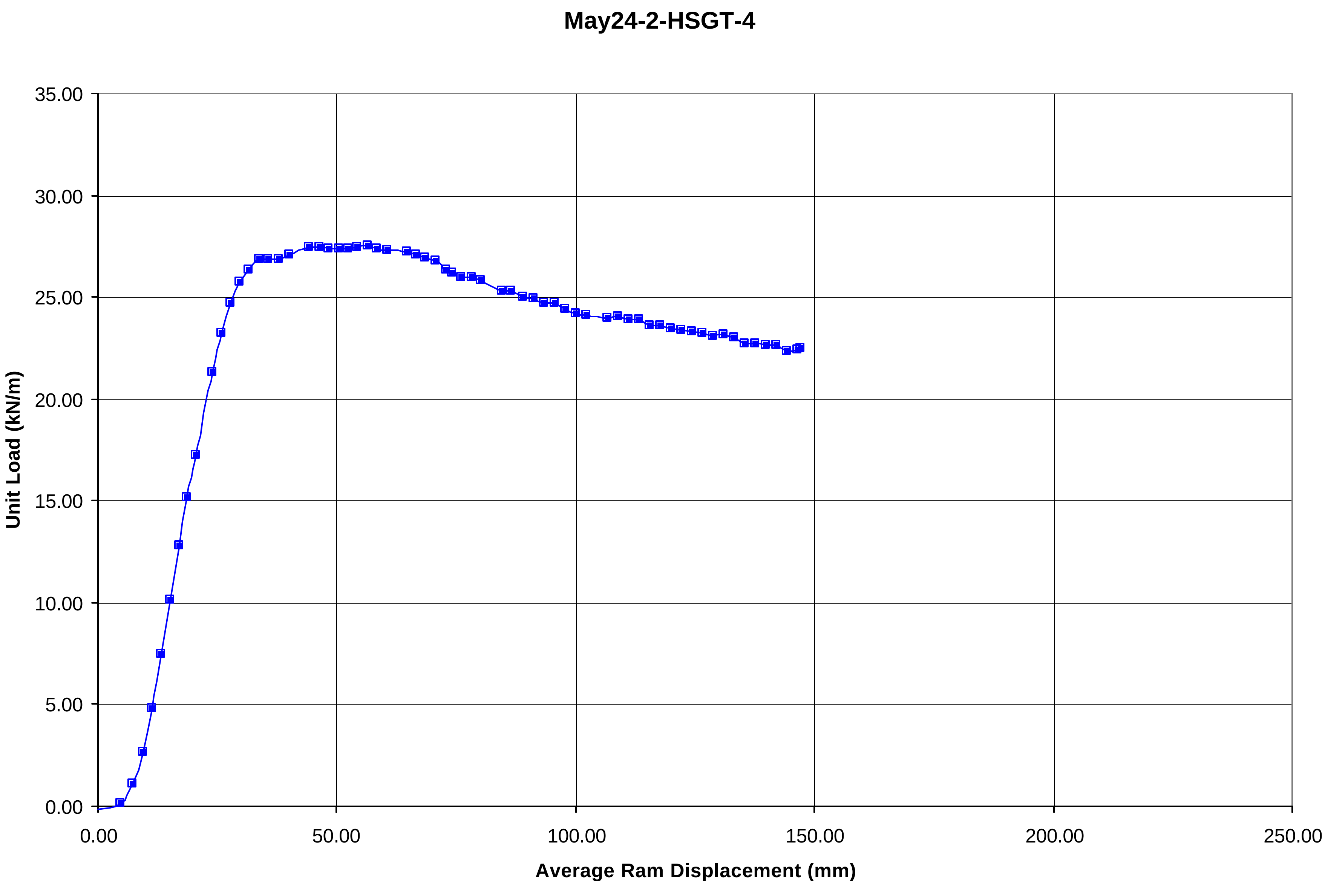

May22-2-RGG-2-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
Average Ram Displacement (mm)



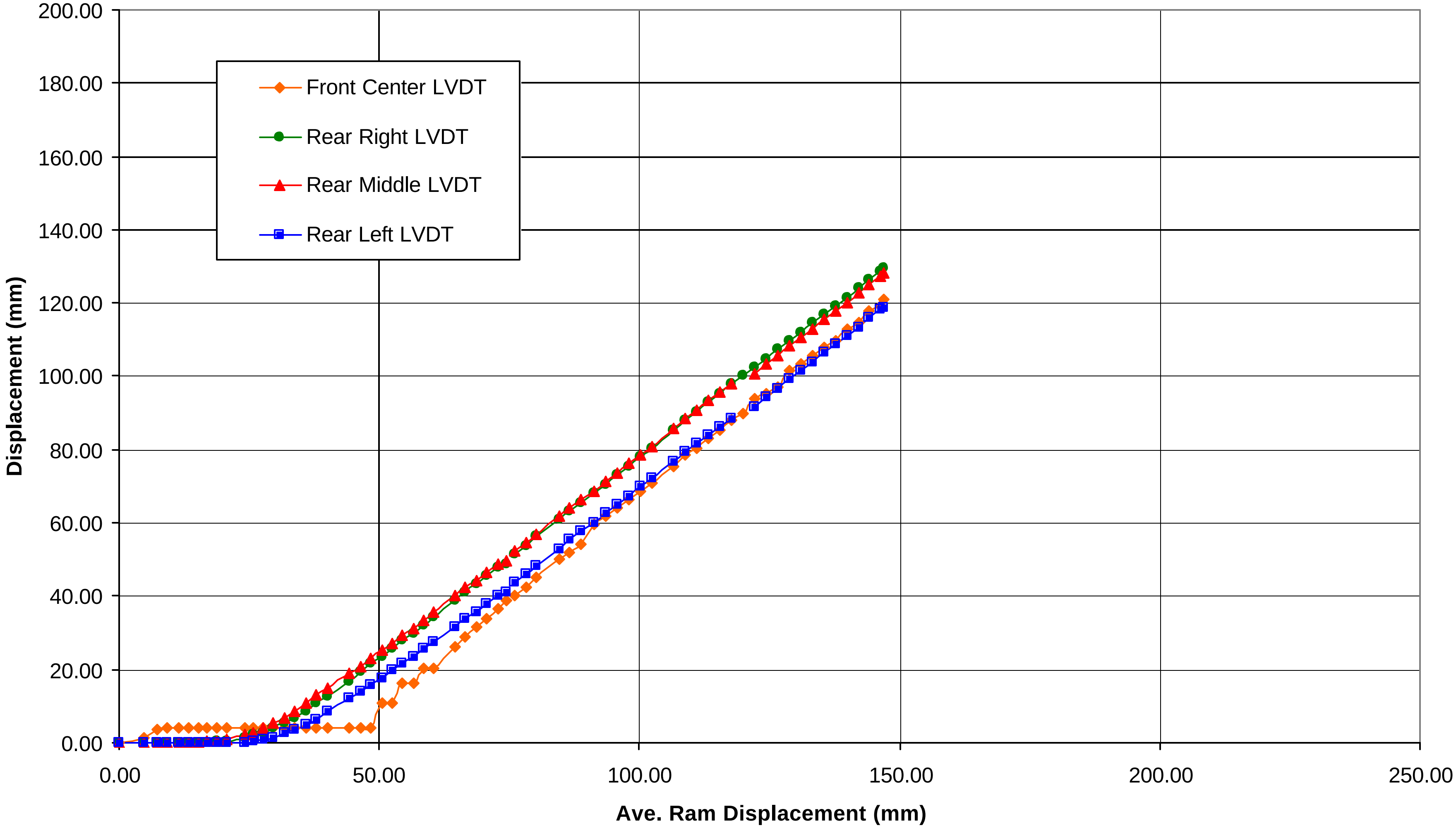
May23-3-LSGT-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

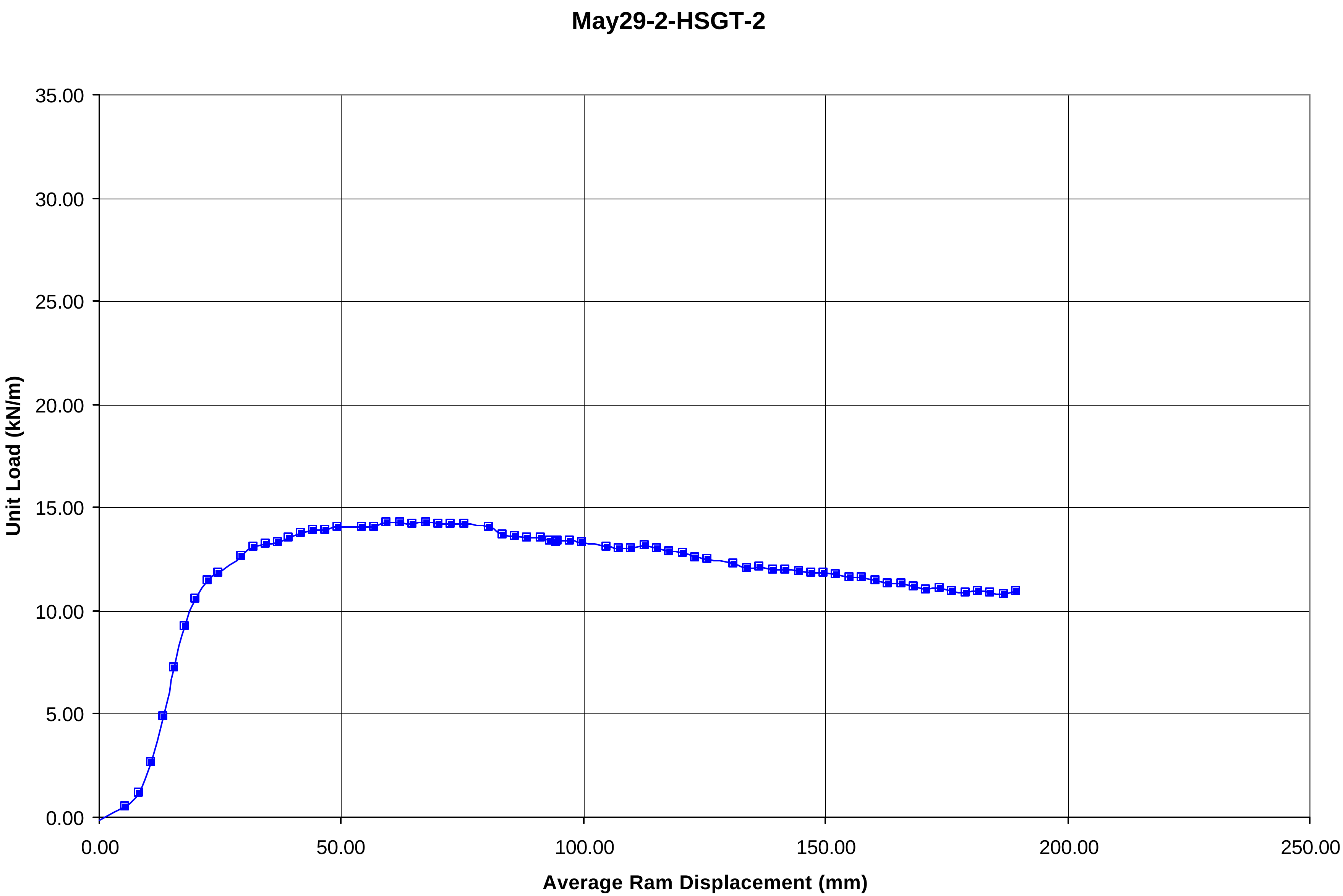


May23-2-LSGT-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
Average Ram Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00

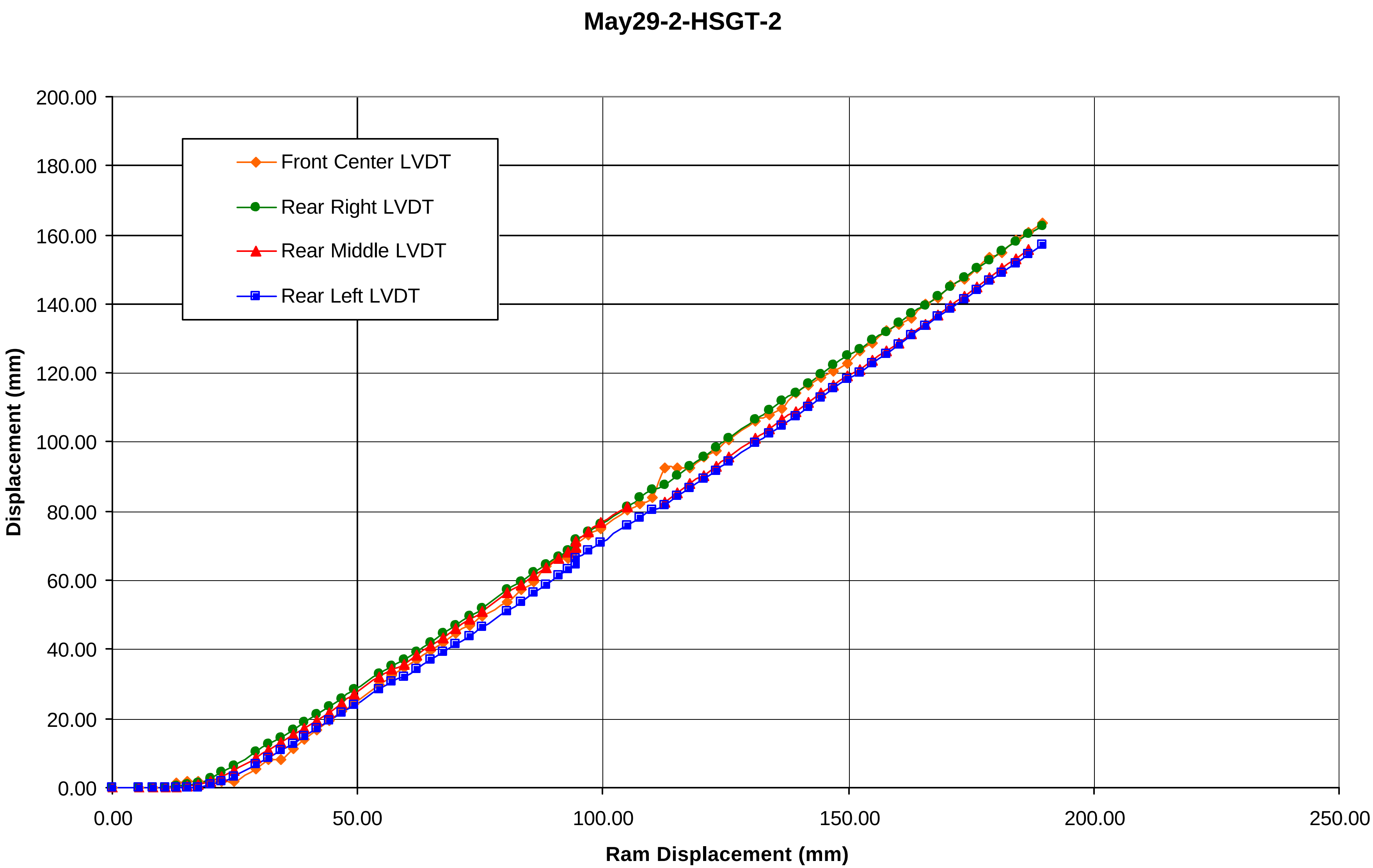

May24-2-HSGT-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

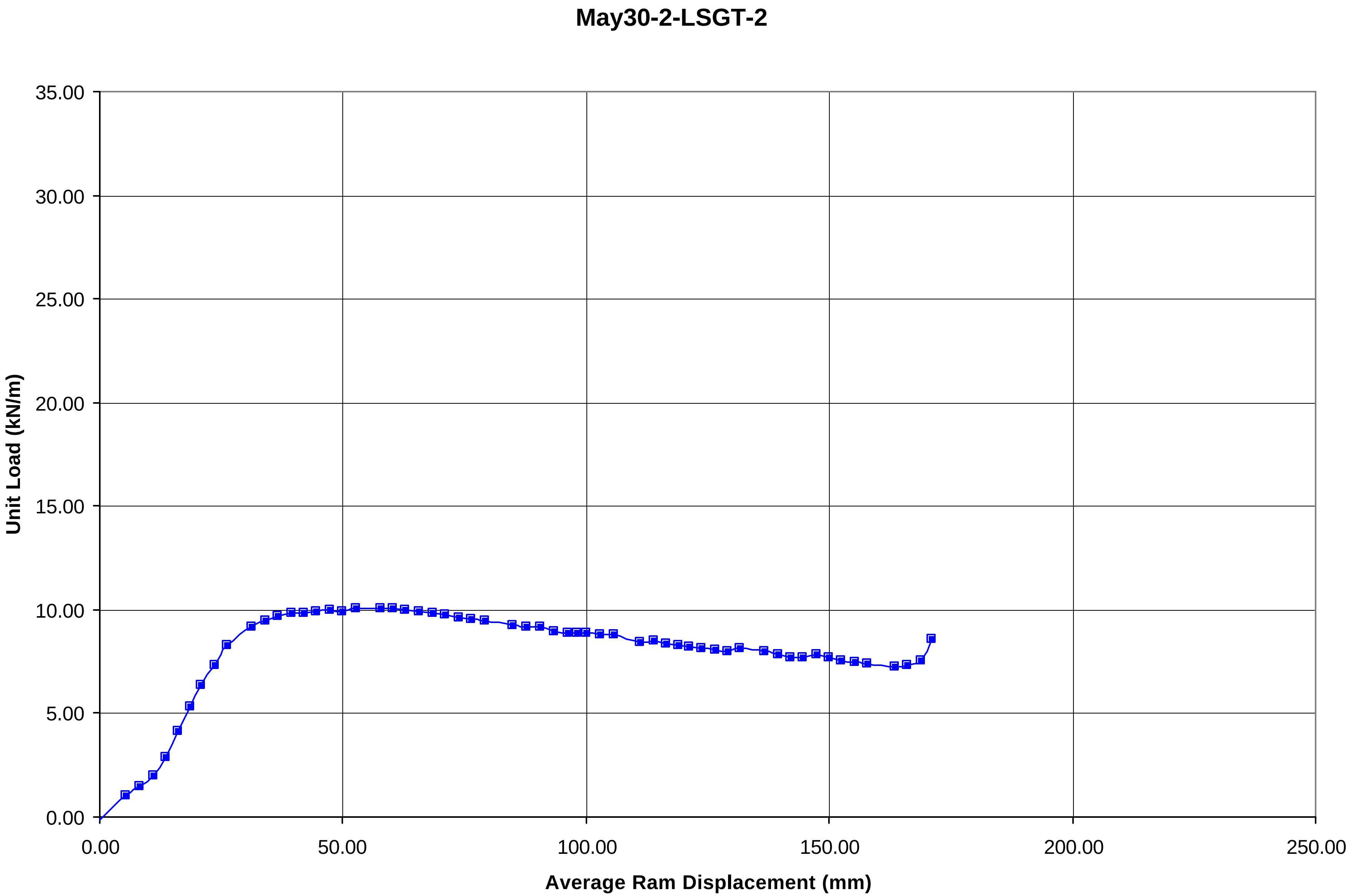
May24-2-HSGT-4



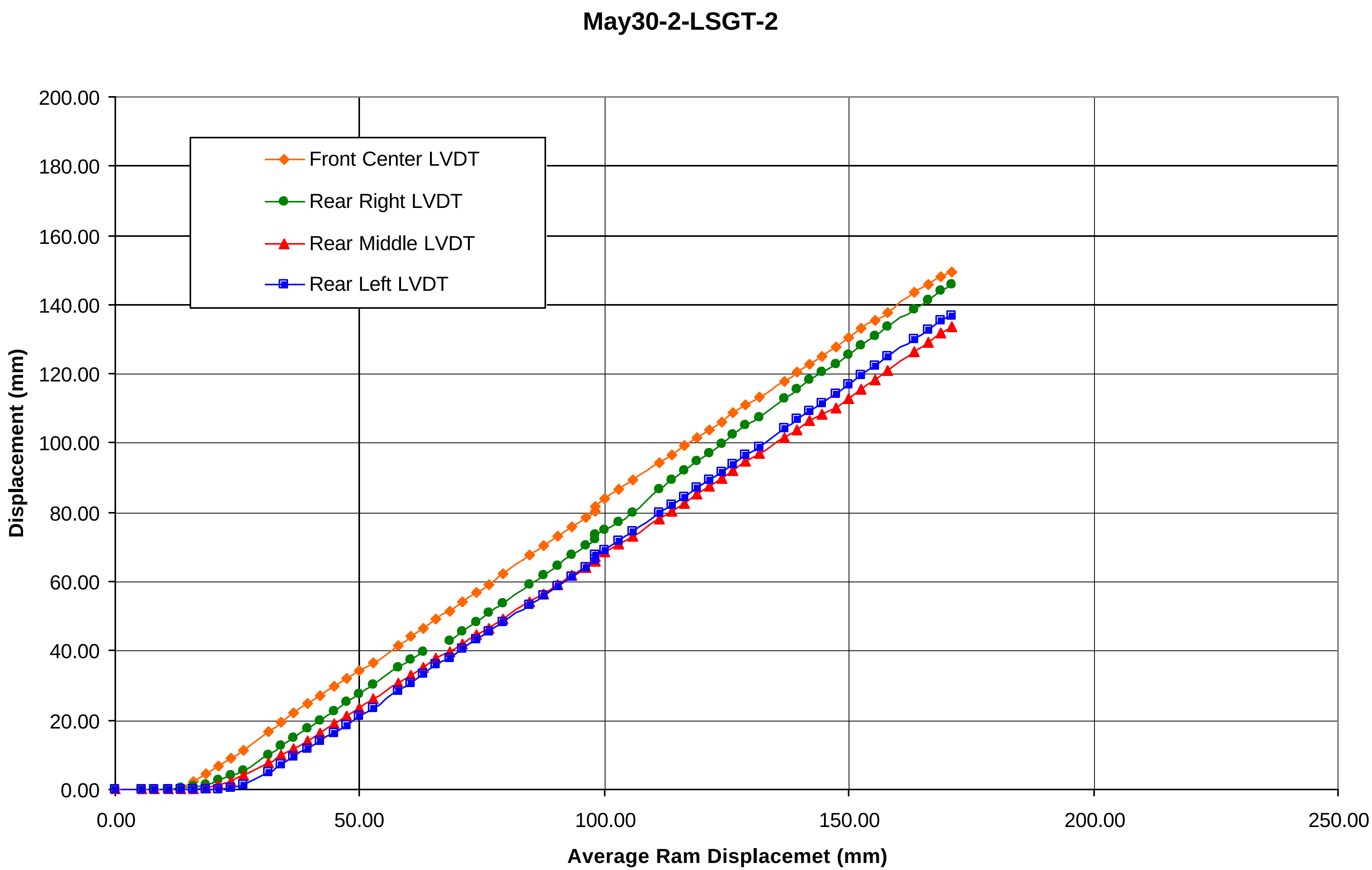


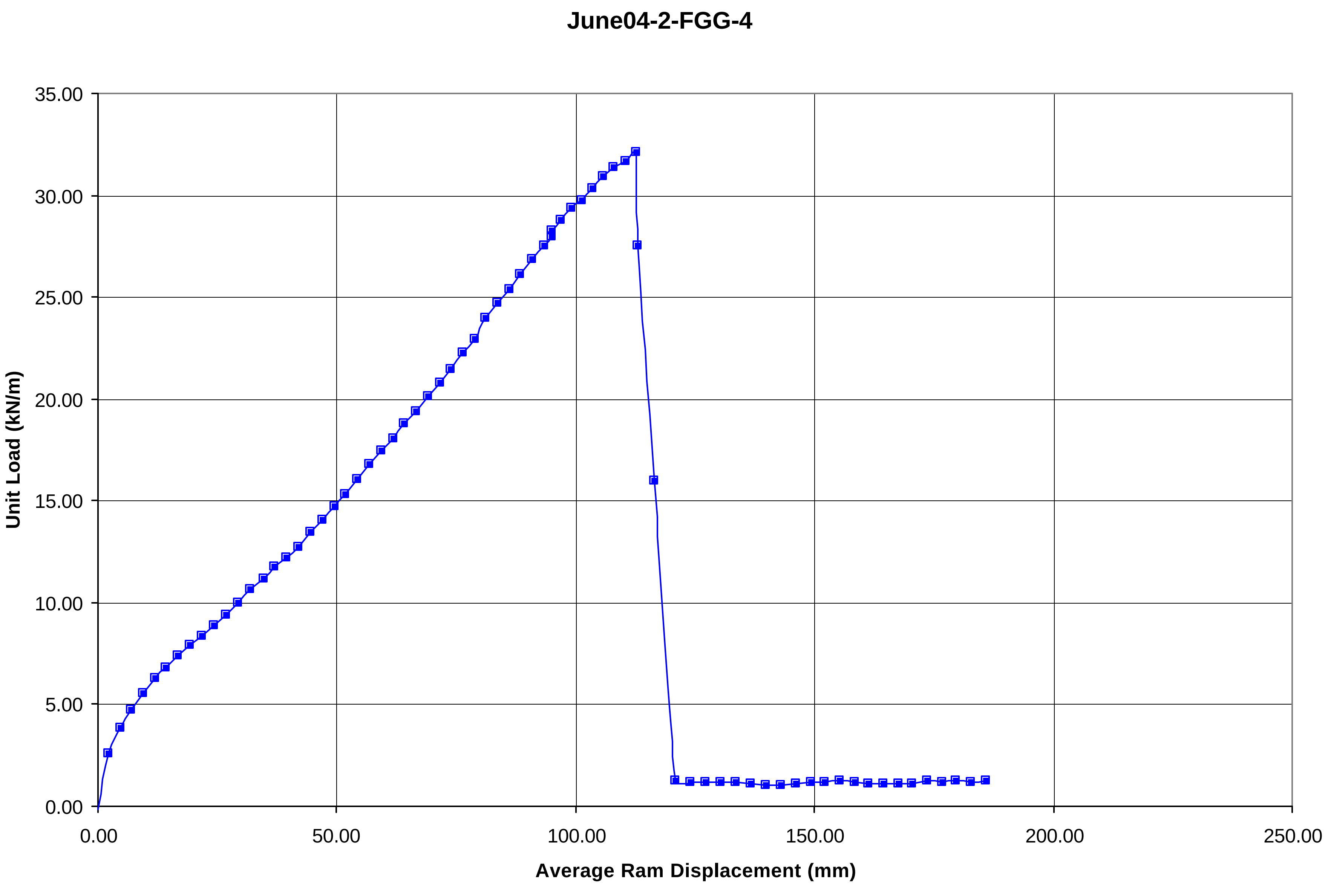
May29-2-HSGT-2
Unit Load (kN/m)
Average Ram Displacement (mm)
0.00
5.00
10.00
15.00
20.00
25.00
30.00
35.00
0.00
50.00
100.00
150.00
200.00
250.00

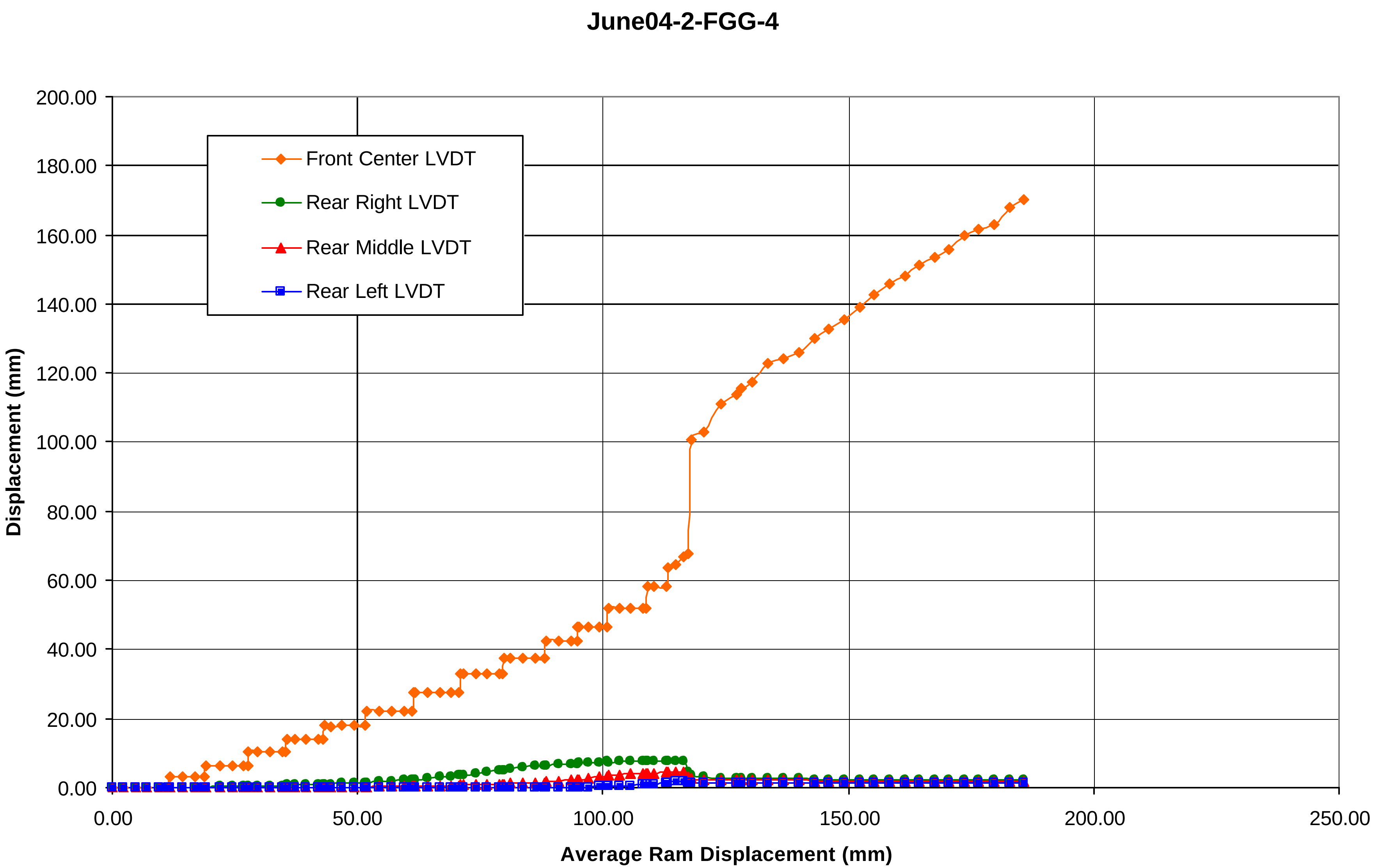

May29-2-HSGT-2
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Ram Displacement (mm)



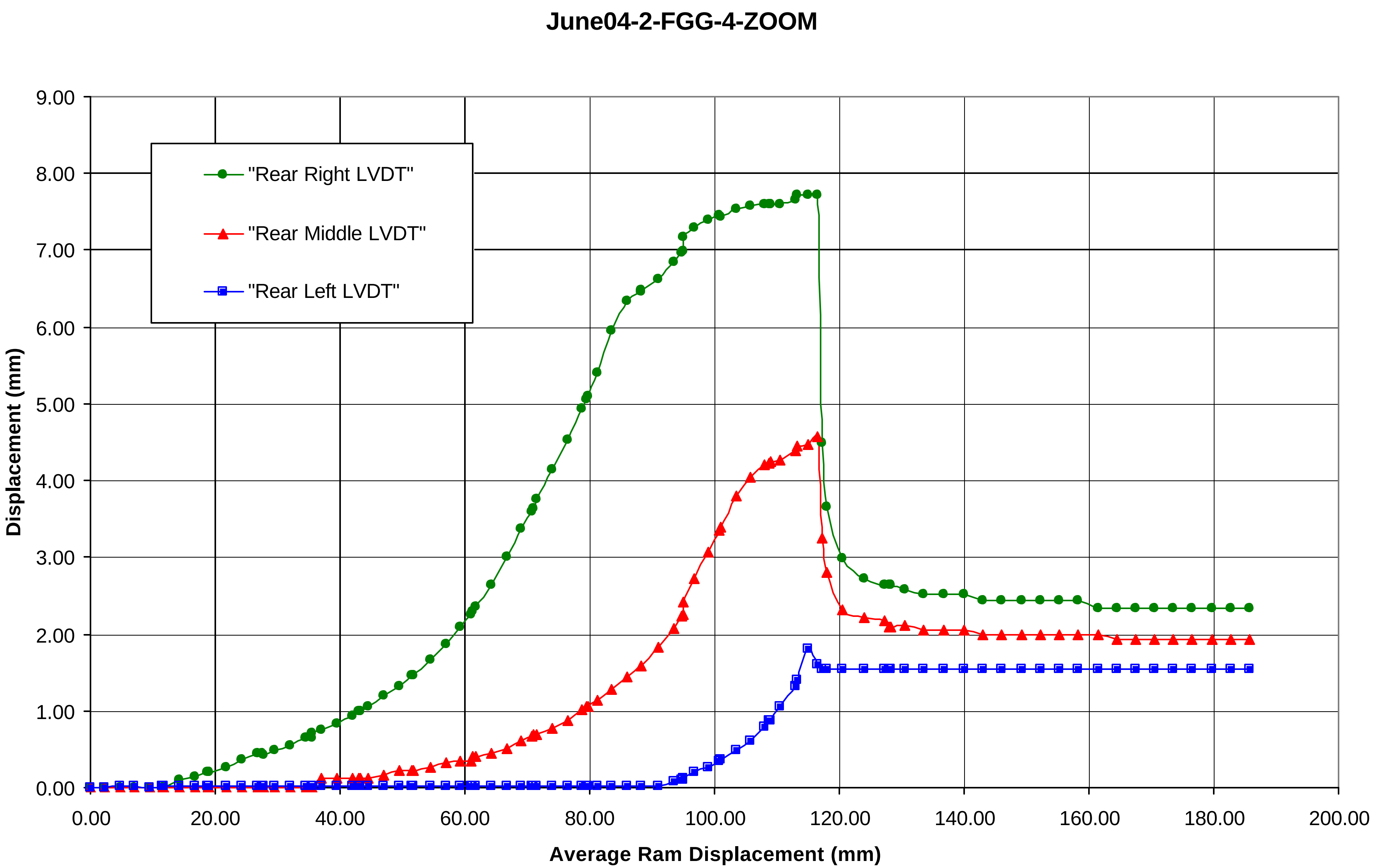
May30-2-LSGT-2
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

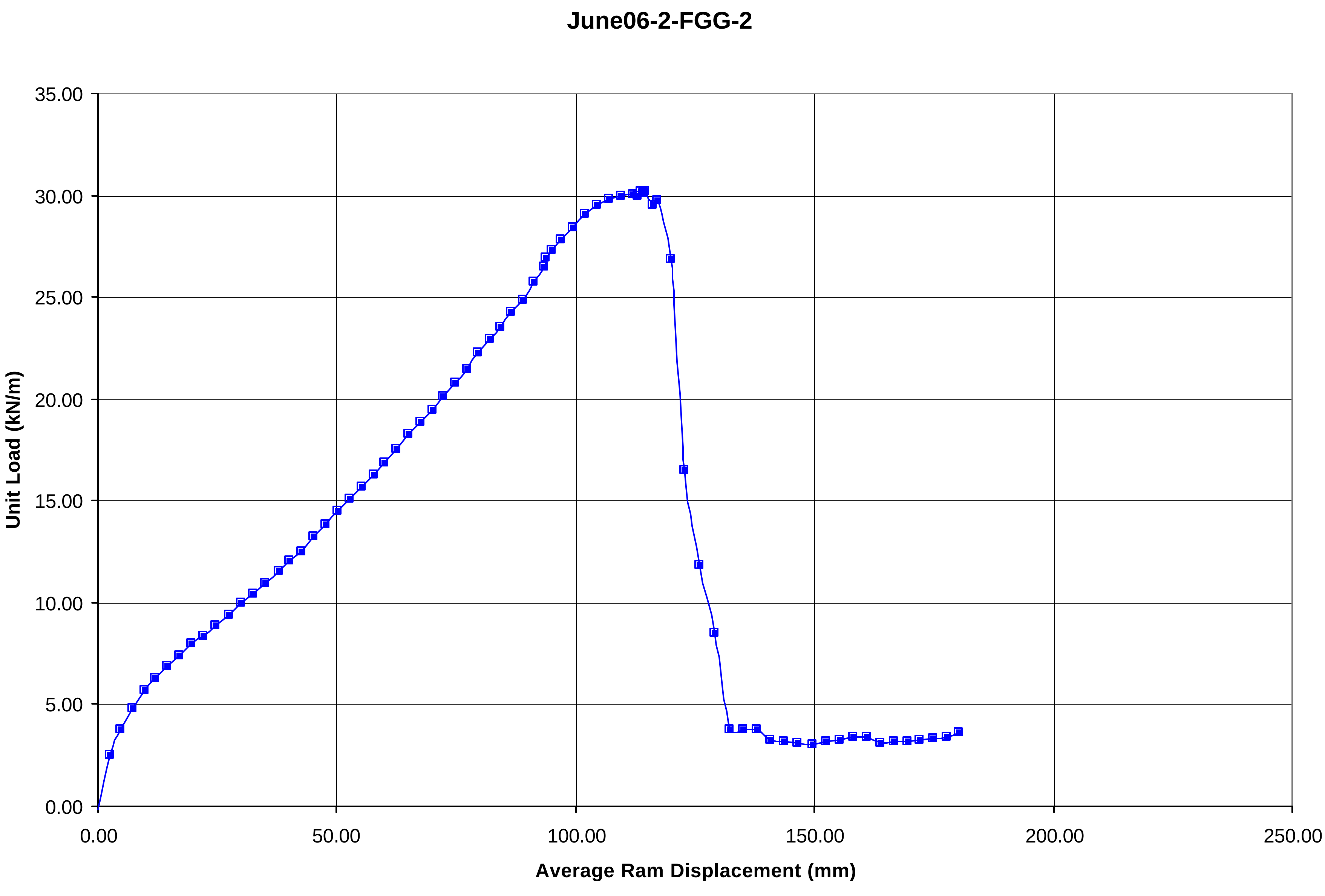

May30-2-LSGT-2
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacemet (mm)


June04-2-FGG-4
Unit Load (kN/m)
Average Ram Displacement (mm)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00

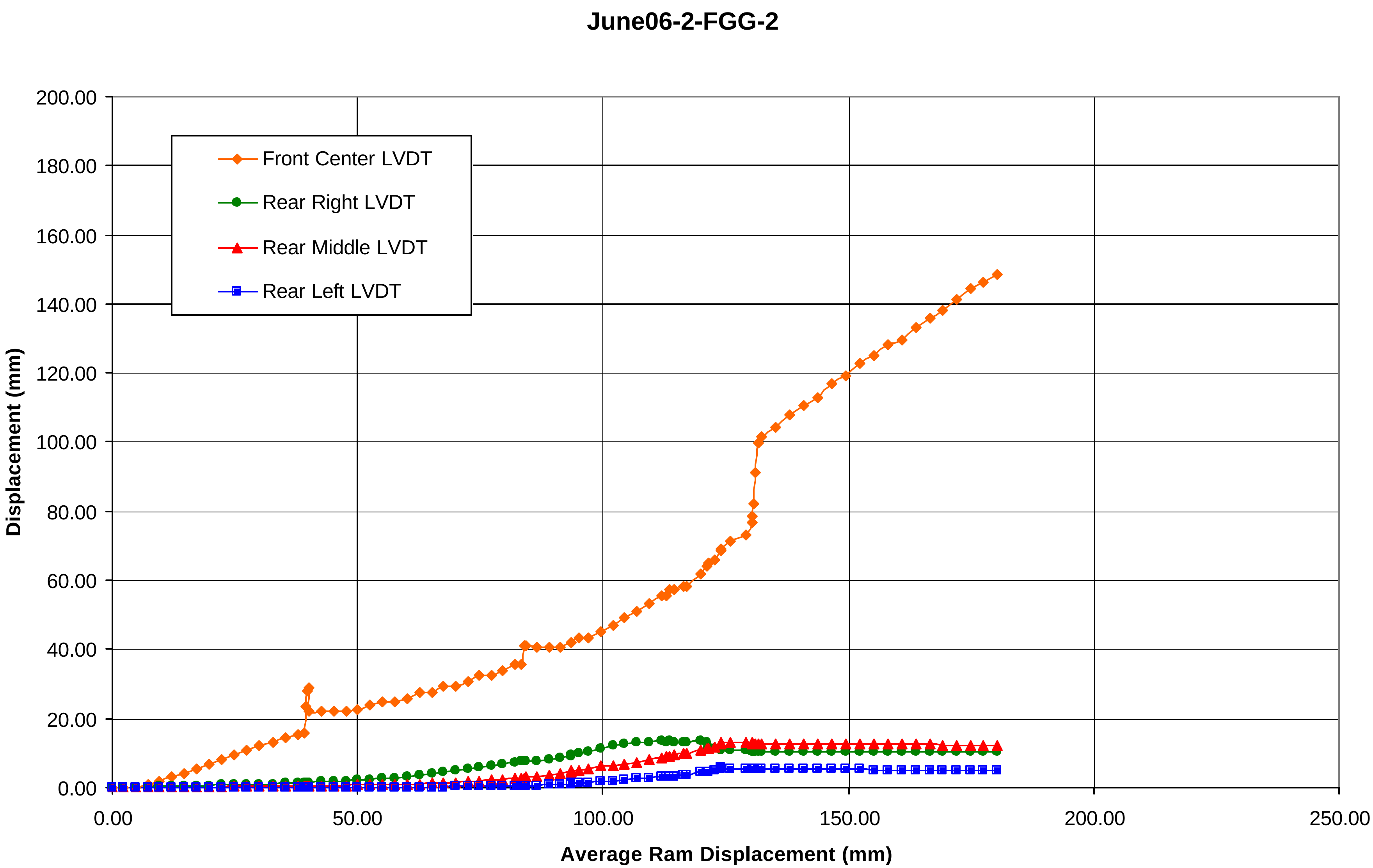


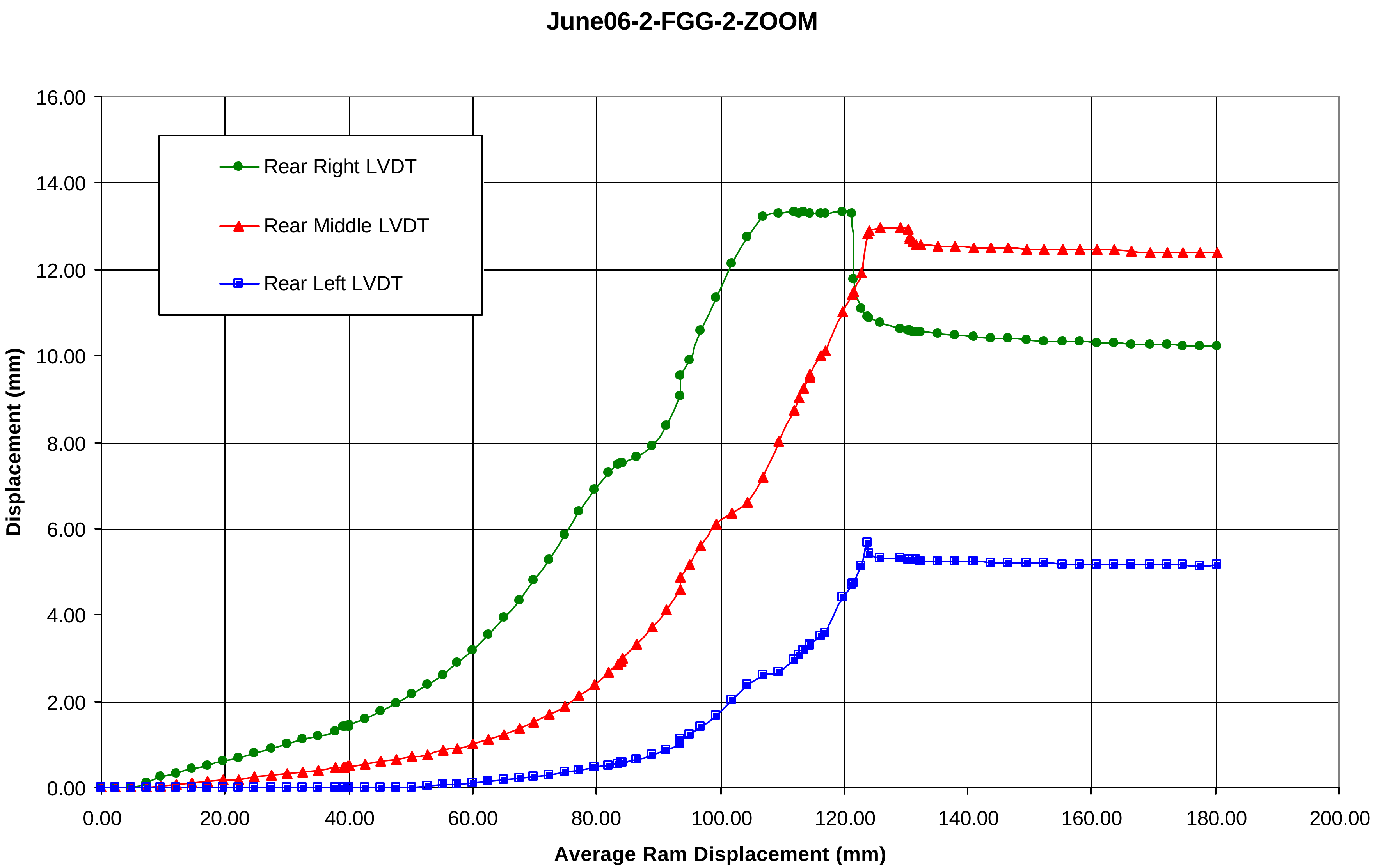
June04-2-FGG-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)


June04-2-FGG-4-ZOOM
"Rear Right LVDT"
"Rear Middle LVDT"
"Rear Left LVDT"
Displacement (mm)
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
Average Ram Displacement (mm)

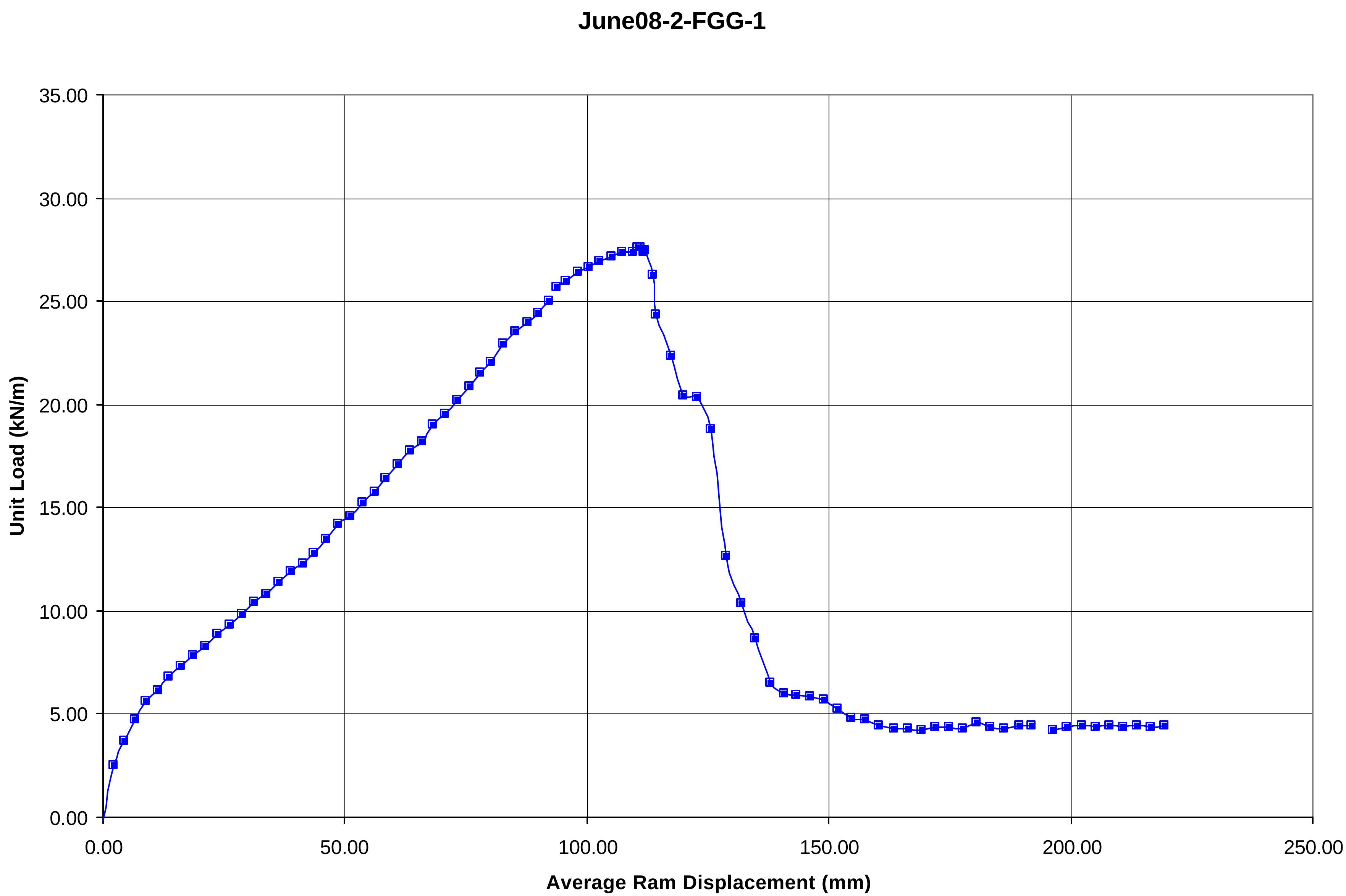


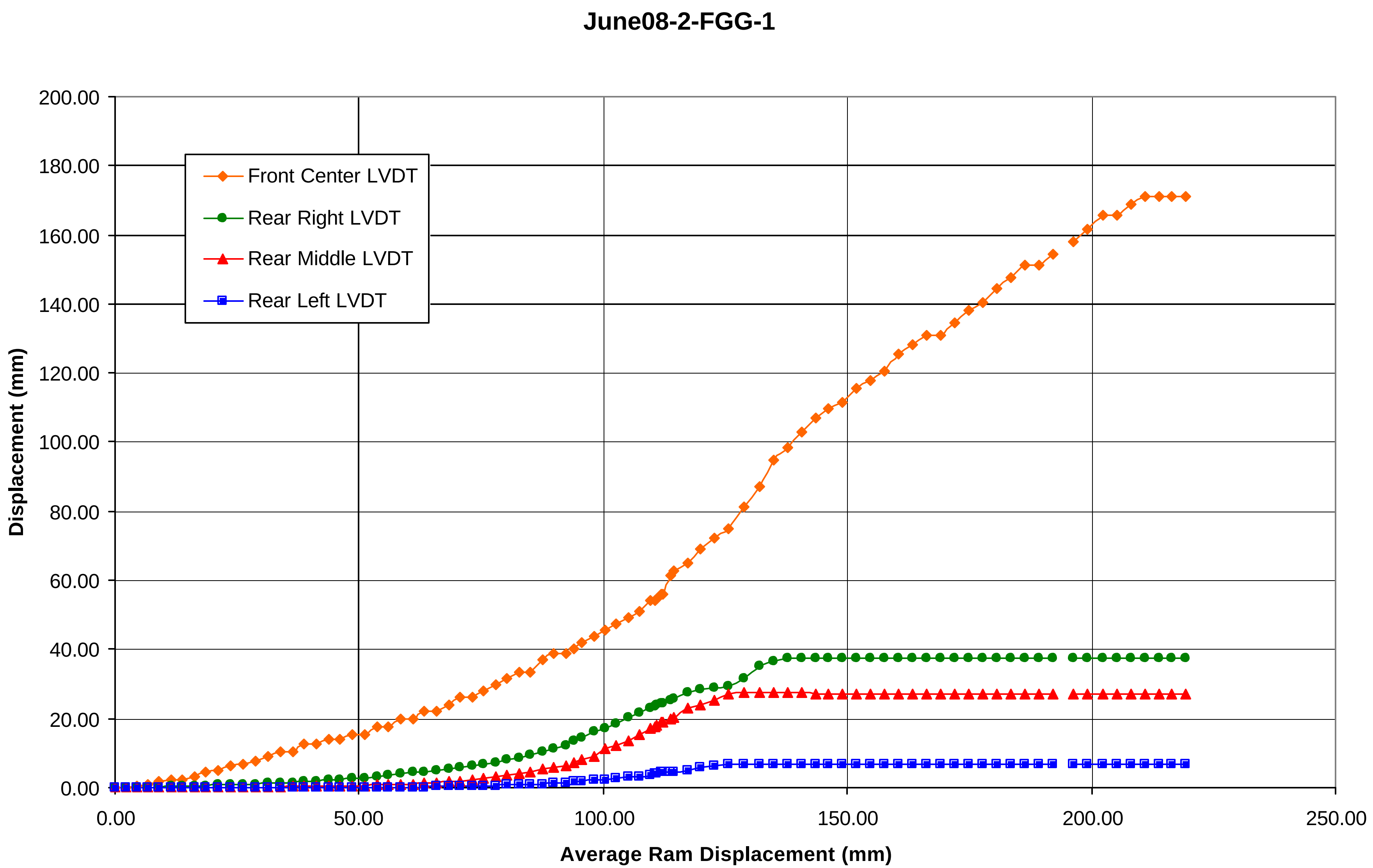
June06-2-FGG-2
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)


June06-2-FGG-2
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
Average Ram Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00

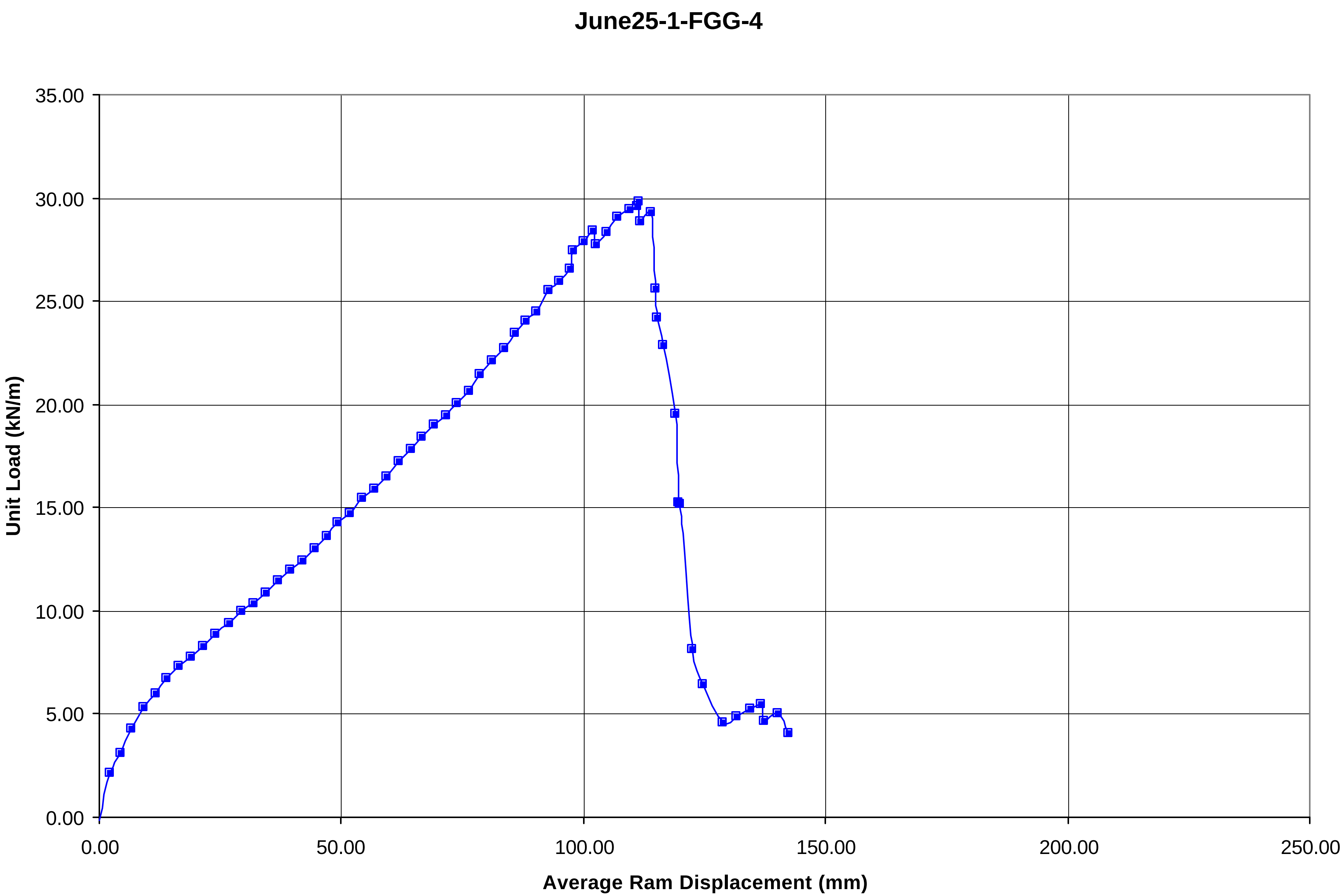


June06-2-FGG-2-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
Average Ram Displacement (mm)

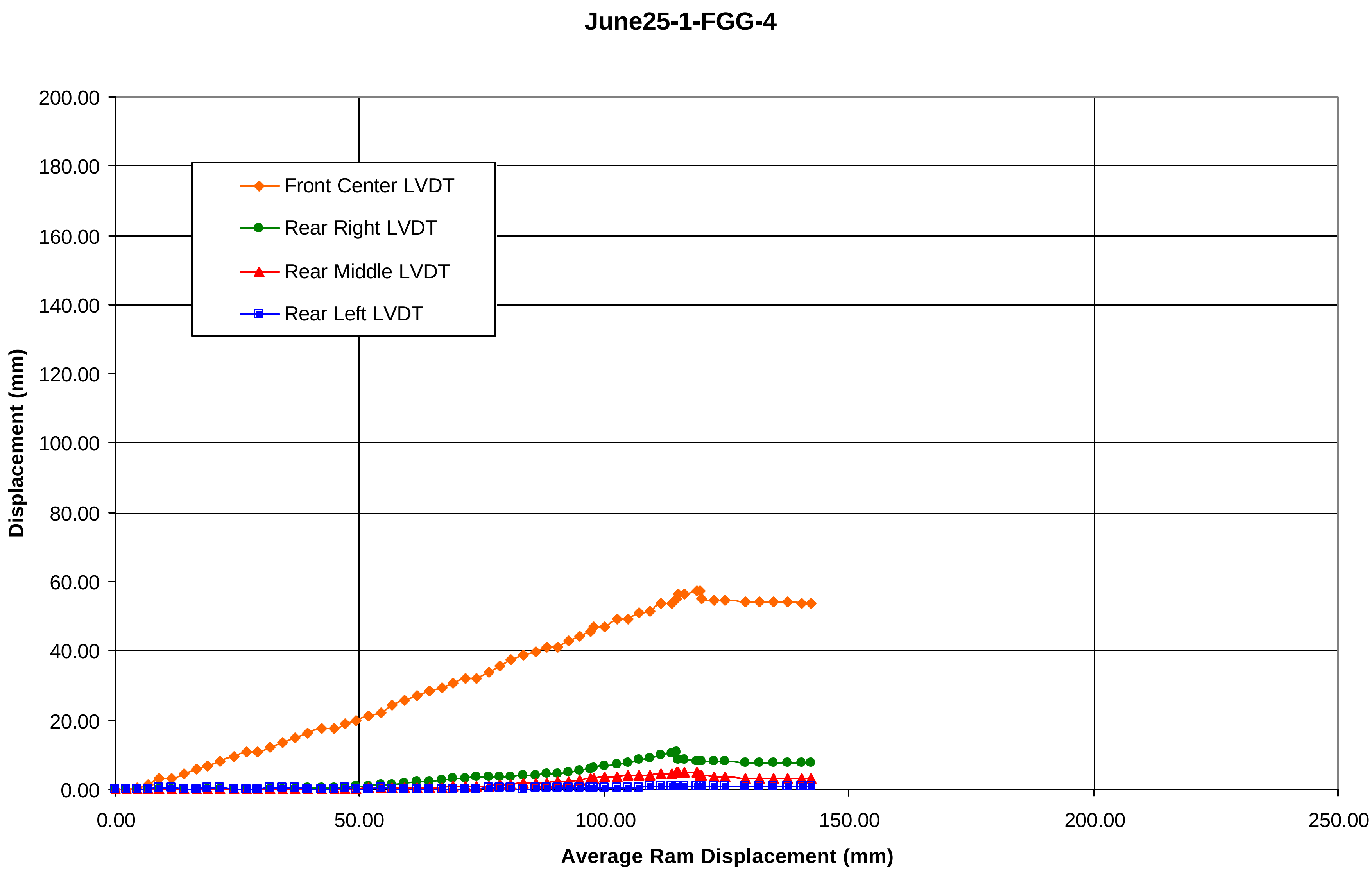

June08-2-FGG-1
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



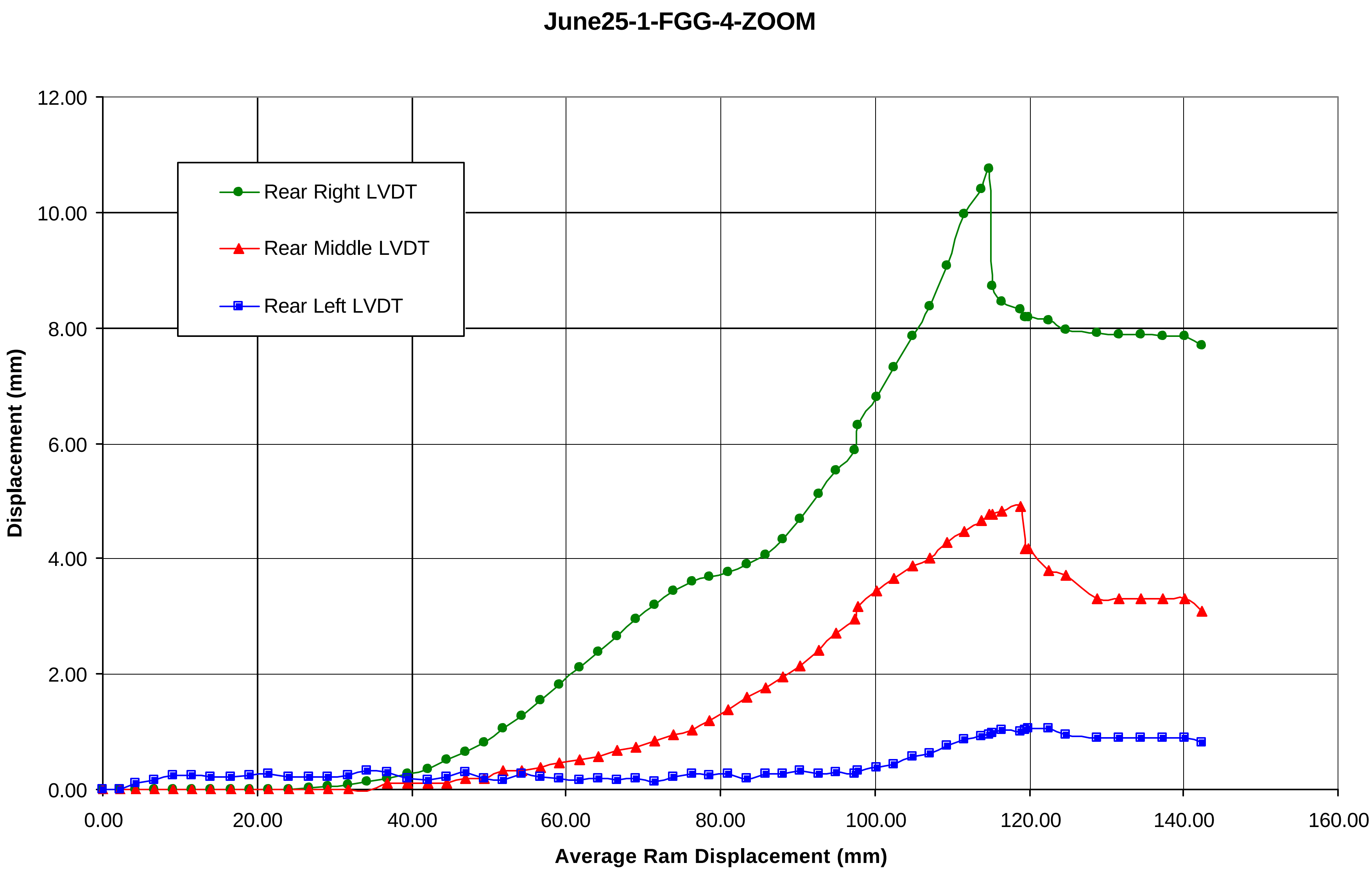
June08-2-FGG-1
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
Average Ram Displacement (mm)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
50.00
150.00
200.00
250.00



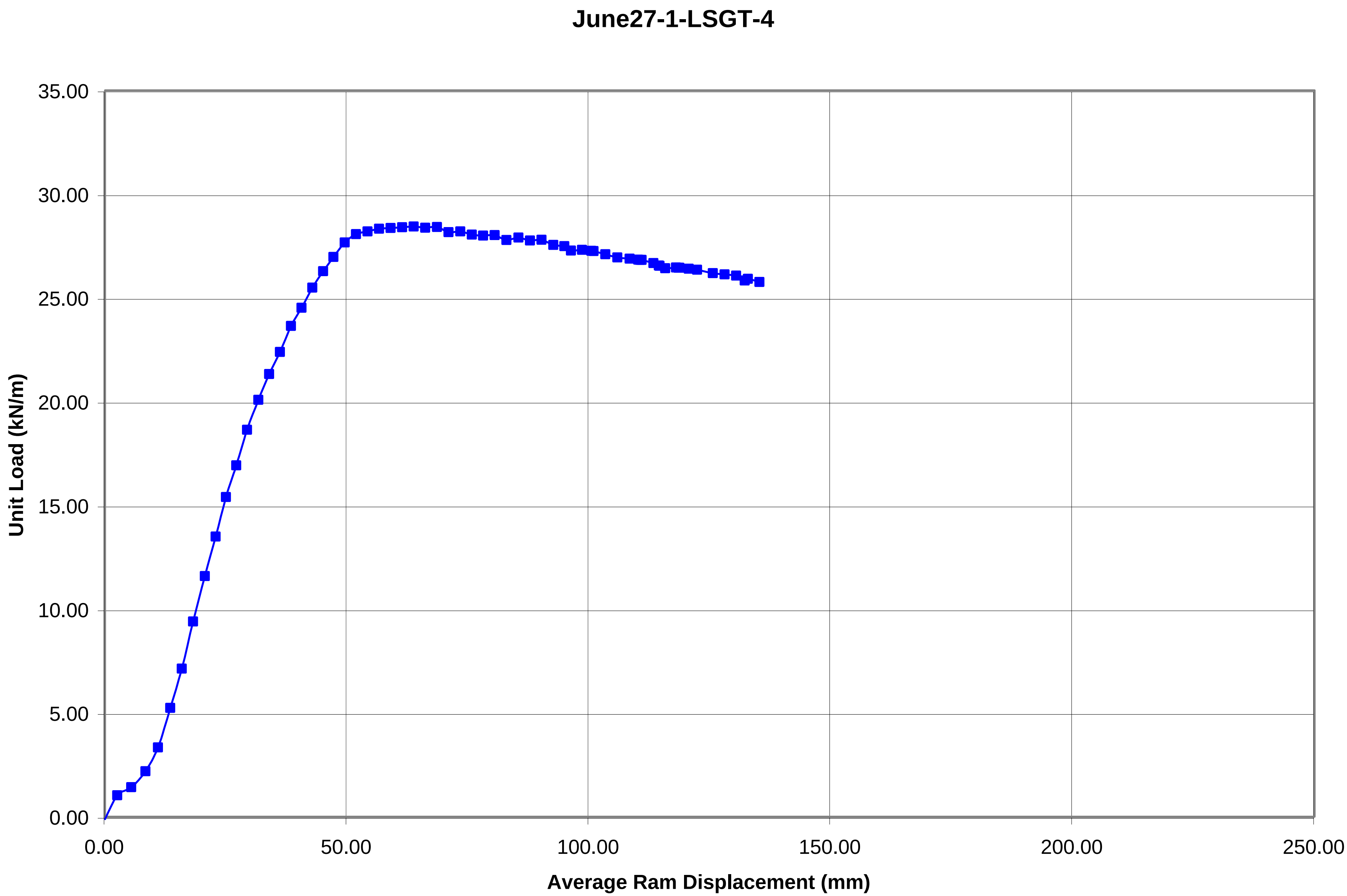
June25-1-FGG-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)

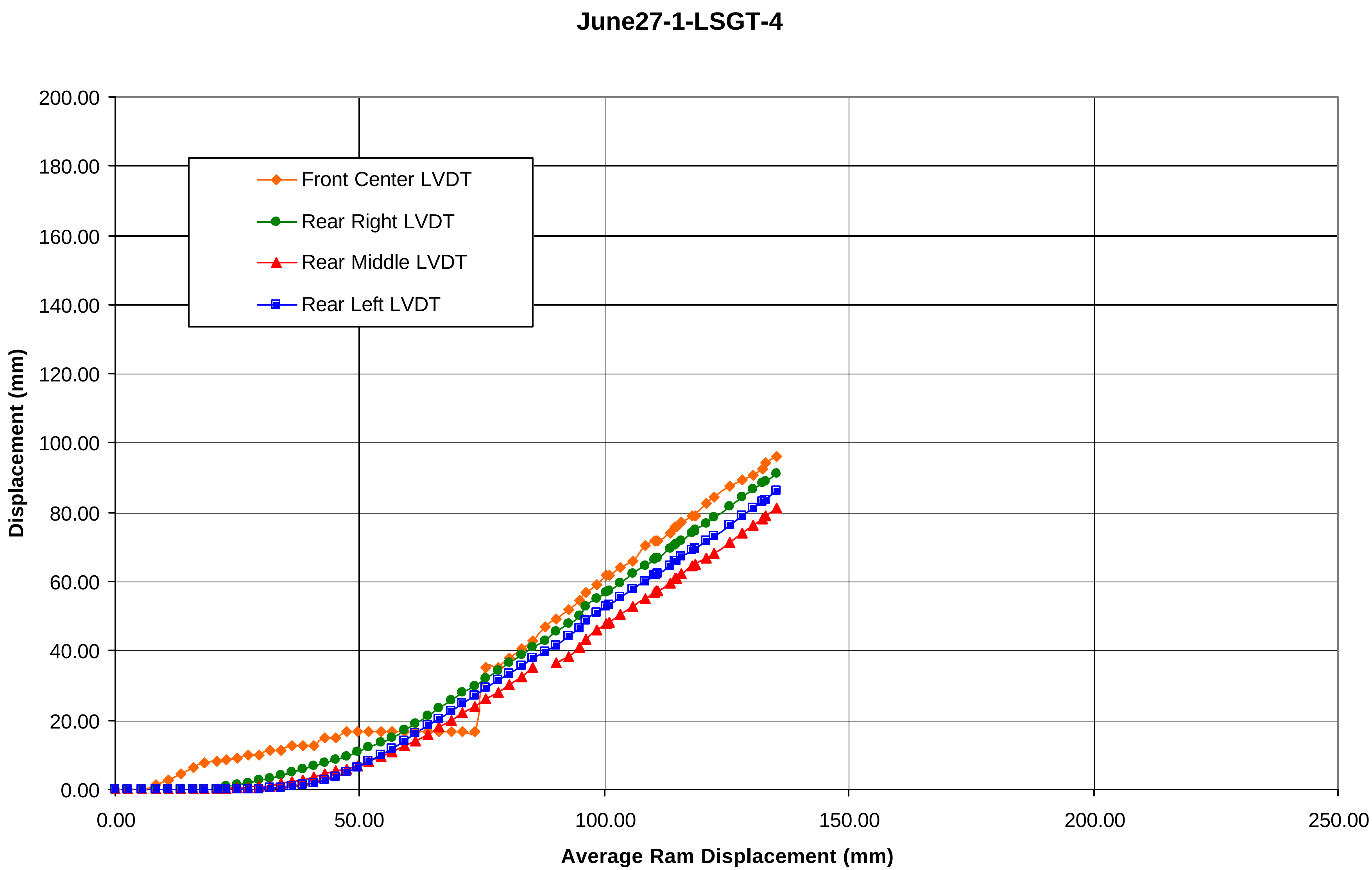


June25-1-FGG-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



June25-1-FGG-4-ZOOM
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
Average Ram Displacement (mm)


June27-1-LSGT-4
Unit Load (kN/m)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
Average Ram Displacement (mm)



June27-1-LSGT-4
Front Center LVDT
Rear Right LVDT
Rear Middle LVDT
Rear Left LVDT
Displacement (mm)
Average Ram Displacement (mm)
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00

CHAPTER 8: ANALYSIS AND INTERPRETATION

8.1. Parametric Combinations

For easy reference, the complete test matrix is repeated in Table 8.1.

Table 8.1. Pullout Test Matrix

Project Test Number	Test Name	Soil Type (Class II, Type 2)	Reinforcement Type	Confining Pressure (psi)
1	Feb28-1-FGG-4	A-2-4	Flexible Geogrid	4
2	Mar06-1-FGG-8	A-2-4	Flexible Geogrid	8
3	Mar07-1-FGG-4	A-2-4	Flexible Geogrid	4
4	Mar21-1-FGG-2	A-2-4	Flexible Geogrid	2
5	Apr12-1-HSGT-4	A-2-4	High Strength Geotextile	4
6	Apr16-1-HSGT-8	A-2-4	High Strength Geotextile	8
7	Apr20-1-LSGT-4	A-2-4	Low Strength Geotextile	4
8	Apr27-1-RGG-4	A-2-4	Rigid Geogrid	4
9	May06-1-RGG-2	A-2-4	Rigid Geogrid	2
10	May10-1-RGG-1	A-2-4	Rigid Geogrid	1
11	May15-2-FGG-4	A-4	Flexible Geogrid	4
12	May17-2-FGG-2	A-4	Flexible Geogrid	2
13	May18-2-RGG-4	A-4	Rigid Geogrid	4
14	May22-2-RGG-2	A-4	Rigid Geogrid	2
15	May23-2-LSGT-4	A-4	Low Strength Geotextile	4
16	May24-2-HSGT-4	A-4	High Strength Geotextile	4
17	May29-2-HSGT-2	A-4	High Strength Geotextile	2
18	May30-2-LSGT-2	A-4	Low Strength Geotextile	2
19	Empty box 1	none	none	none
20	Jun04-2-FGG-4	A-4	Flexible Geogrid	4
21	Jun06-2-FGG-2	A-4	Flexible Geogrid	2
22	Jun8-2-FGG-1	A-4	Flexible Geogrid	1
23	Jun25-1-FGG-4	A-4	Flexible Geogrid	4
24	Jun27-1-LSGT-4	A-4	Low Strength Geotextile	4
25	Empty box 2	none	none	none

The analysis and interpretation of these tests can be performed in a variety of ways. Since this testing program was essentially a parametric study, one clear way is to examine the results in that manner. The three parameters that were varied included the soil type, the reinforcement type, and the confining pressure. Tests can be grouped to assess the effects of these three parameters in the following ways:

1. *Same* soil type, *same* confining pressure, *different* reinforcement types.
2. *Different* soil type, *same* confining pressure, *same* reinforcement type.
3. *Same* soil type, *different* confining pressures, *same* reinforcement type

The tests are grouped according to these three combinations in Tables 8.2a, 8.2b, and 8.2c. Observations on test results within each of these categories are presented in the following sections.

Table 8.2a. Parametric Combination #1 and Associated Project Test Numbers

Case No.	Parametric Combination	Project Test Numbers	Comments
1-I	1 (Same soil type, same confining pressure, different reinforcement types)	4 & 9	Soil No. 1, 2 psi confining pressure, flexible and rigid geogrids
1-II	1 (Same soil type, same confining pressure, different reinforcement types)	3, 5, 7, 8, 23, & 24	Soil No. 1, 4 psi confining pressure, flexible and rigid geogrids, high and low strength geotextiles
1-III	1 (Same soil type, same confining pressure, different reinforcement types)	12, 14, 17, 18, & 21	Soil No. 2, 2 psi confining pressure, flexible and rigid geogrids, high and low strength geotextiles
1-IV	1 (Same soil type, same confining pressure, different reinforcement types)	11, 13, 15, 16, & 20	Soil No. 2, 4 psi confining pressure, flexible and rigid geogrids, high and low strength geotextiles

Table 8.2b. Parametric Combination #2 and Associated Project Test Numbers

Case No.	Parametric Combination	Project Test Numbers	Comments
2-I	2 (Different soil type, same confining pressure, same reinforcement type)	4 & 12	Soil Nos. 1 & 2, 2 psi confining pressure, flexible geogrid
2-II	2 (Different soil type, same confining pressure, same reinforcement type)	9 & 14	Soil Nos. 1 & 2, 2 psi confining pressure, rigid geogrid
2-III	2 (Different soil type, same confining pressure, same reinforcement type)	3, 11, 20, & 23	Soil Nos. 1 & 2, 4 psi confining pressure, flexible geogrid
2-IV	2 (Different soil type, same confining pressure, same reinforcement type)	5 & 16	Soil Nos. 1 & 2, 4 psi confining pressure, high strength geotextile
2-V	2 (Different soil type, same confining pressure, same reinforcement type)	7, 15, & 24	Soil Nos. 1 & 2, 4 psi confining pressure, low strength geotextile
2-VI	2 (Different soil type, same confining pressure, same reinforcement type)	8 & 13	Soil Nos. 1 & 2, 4 psi confining pressure, rigid geogrid

Table 8.2c. Parametric Combination #3 and Associated Project Test Numbers

Case No.	Parametric Combination	Project Test Numbers	Comments
3-I	3 (Same soil type, different confining pressure, same reinforcement type)	3, 4, & 23	Soil No. 1, flexible geogrid, 2 & 4 psi confining pressure
3-II	3 (Same soil type, different confining pressure, same reinforcement type)	5 & 6	Soil No. 1, high strength geotextile, 4 & 8 psi confining pressure
3-III	3 (Same soil type, different confining pressure, same reinforcement type)	8, 9, & 10	Soil No. 1, rigid geogrid, 1, 2 & 4 psi confining pressure
3-IV	3 (Same soil type, different confining pressure, same reinforcement type)	11, 12, 20, 21, & 22	Soil No. 2, flexible geogrid, 1, 2, & 4 psi confining pressure
3-V	3 (Same soil type, different confining pressure, same reinforcement type)	13 & 14	Soil No. 2, rigid geogrid, 2 & 4 psi confining pressure
3-VI	3 (Same soil type, different confining pressure, same reinforcement type)	15 & 18	Soil No. 2, low strength geotextile, 2 & 4 psi confining pressure
3-VII	3 (Same soil type, different confining pressure, same reinforcement type)	16 & 17	Soil No. 2, high strength geotextile, 2 & 4 psi confining pressure

8.2. Parametric Combination #1

In Parametric Combination #1, the soil type and confining pressure was held constant and different geosynthetic reinforcement types were used. This combination gives a "side by side" comparison of the load versus displacement behavior of the reinforcing materials when embedded in the selected Piedmont residual soils. In this research program, there were four individual cases where the effects of Parametric Combination #1 could be examined. These are given in Table 8.2a.

The results of the four cases of Parametric Combination #1 are summarized in Table 8.3. Since load versus deformation information is most important, the measured values of peak load (per unit width of material) and the corresponding hydraulic ram displacement are provided for comparison.

In interpreting the data presented in Table 8.3, several items are worth noting. First, in general, the flexible geogrid exhibited better performance than the rigid geogrid. In virtually all cases, the flexible geogrid carried a higher peak load at a lower displacement. Second, under lower confining pressure (2 psi), the geogrids and the geotextiles tended to behave very similarly. The geogrids exhibited higher peak strengths than the geotextiles, but these peaks occurred at much larger displacements than those for the geotextiles. This is an important observation when displacement is an important performance consideration. Finally, under higher confining pressures (4 psi), the geotextiles exhibited peak loads approximately equal to the geogrids while undergoing displacements less than half as large. Again, this is an important observation with respect to minimizing displacements.

Table 8.3. Summary of Parametric Combination #1 Results

Case No.	Variable (geosynthetic type)	Peak Load (kN/m)	Ram Displacement @ Peak Load (mm)	Comments
1-I	Flexible geogrid	32	95	Soil No. 1, 2 psi confining pressure
	Rigid geogrid	21	130	
1-II	Flexible geogrid	28	112	Soil No. 1, 4 psi confining pressure
	Rigid geogrid	22	128	
	High strength geotextile	17	48	
	Low strength geotextile	28	50	
1-III	Flexible geogrid	21	110	Soil No. 2, 2 psi confining pressure
	Rigid geogrid	22	120	
	High strength geotextile	14.5	60	
	Low strength geotextile	10	60	
1-IV	Flexible geogrid	32	115	Soil No. 2, 4 psi confining pressure
	Rigid geogrid	22	125	
	High strength geotextile	27.5	50	
	Low strength geotextile	29	50	

8.3. Parametric Combination #2

In Parametric Combination #2, the confining pressure and geosynthetic reinforcement type was held constant and different soil types were used. This combination gives insight into how much of an effect the soil type, particularly the “lower quality” soils selected, has on the load versus displacement behavior of the reinforcing materials. In this research program, there were six individual cases where the effects of Parametric Combination #2 could be examined. These are given in Table 8.2b. Again, this information is not available from other sources.

The results of the six cases of Parametric Combination #2 are summarized in Table 8.4. Since load versus deformation information is most important, the measured values of peak load (per unit width of material) and the corresponding hydraulic ram displacement are provided for comparison.

In interpreting the data presented in Table 8.4, several items are worth noting. First, the magnitudes of the displacements at peak load do not appear to be sensitive to the soil type. However, as seen in Parametric Combination #1 data, the displacements at peak load for the geotextiles tend to be less than half of the displacements at peak load for the geogrids. Second, the peak loads carried by the geogrid materials are not strongly affected by varying the soil type. Conversely, the peak loads carried by the geotextiles are very strongly affected by the soil type. For both the low and high strength geotextiles, the peak load with Soil No. 2 was nearly double the peak load with Soil No. 1. This is a significant observation.

Table 8.4. Summary of Parametric Combination #2 Results

Case No.	Variable (soil type)	Peak Load (kN/m)	Ram Displacement @ Peak Load (mm)	Comments
2-I	Soil No. 1	32	96	2 psi conf. pressure, flexible geogrid
	Soil No. 2	22	110	
2-II	Soil No. 1	22	130	2 psi conf. pressure, rigid geogrid
	Soil No. 2	22.5	130	
2-III	Soil No. 1	29	110	4 psi conf. pressure, flexible geogrid
	Soil No. 2	32	112	
2-IV	Soil No. 1	17	50	4 psi conf. pressure, high strength geotextile
	Soil No. 2	27	50	
2-V	Soil No. 1	15.5	60	4 psi conf. pressure, low strength geotextile
	Soil No. 2	29	55	
2-VI	Soil No. 1	22.5	130	4 psi conf. Pressure, rigid geogrid
	Soil No. 2	22	130	

8.4. Parametric Combination #3

In Parametric Combination #3, the soil type and geosynthetic reinforcement type were held constant and different confining pressures were used. This combination gives insight into how a particular combination of soil and geosynthetic reinforcing material will interact at different heights along a retaining wall. Typically, the soil- geosynthetic interaction behavior near the top of the wall (where confining pressures are lower) is different than the interaction behavior near the bottom of the wall (where confining pressures are higher). In this research program, there were seven individual cases where the effects of Parametric Combination #3 could be examined. These are given in Table 8.2c.

The results of the seven cases of Parametric Combination #3 are summarized in Table 8.5. Since load versus deformation information is most important, the measured values of peak load (per unit width of material) and the corresponding hydraulic ram displacement are provided for comparison.

In interpreting the data presented in Table 8.5, several items are worth noting. For the geogrids, the peak load tends to increase slightly with increasing confining pressure, but appears to be relatively insensitive to the range of confining pressures used in this study. However, the same is not true for the geotextiles. Both the low and high strength geotextiles exhibited substantial increases in peak load under increased confining pressure. The peak loads nearly tripled when the confining pressure was doubled. This is a significant observation. Consistent with the other parametric combinations, the geotextiles typically experienced less than half the displacement of the geogrids at peak load. Again, this is important to know when displacements are an issue.

Table 8.5. Summary of Parametric Combination #3 Results

Case No.	Variable (confining pressure)	Peak Load (kN/m)	Ram Displacement @ Peak Load (mm)	Comments
3-I	2 psi	32	95	Soil No. 1, Flexible geogrid
	4 psi	29	112	
3-II	4 psi	17	50	Soil No. 1, High strength geotextile
	8 psi	50	90	
3-III	1 psi	19	130	Soil No. 1, Rigid geogrid
	2 psi	21	130	
	4 psi	23	130	
3-IV	1 psi	27	115	Soil No. 2, Flexible geogrid
	2 psi	30	116	
	4 psi	33	115	
3-V	2 psi	22	130	Soil No. 2, Rigid geogrid
	4 psi	22	130	
3-VI	2 psi	10	58	Soil No. 2, Low strength geotextile
	4 psi	29	55	
3-VI	2 psi	14.5	60	Soil No. 2, High strength geotextile
	4 psi	28	50	

CHAPTER 9: CONCLUSIONS AND RECOMMENDATIONS

9.1. Conclusions

The following conclusions are based on the activities performed during this research program. It is important to note these conclusions are based on research performed with four specific geosynthetic reinforcing materials embedded in two types of Piedmont residual soil. While certainly representative of overall behavior, the extension to general conclusions for all geogrid products or reinforced soil types may not be appropriate.

- A large database of soil- geosynthetic reinforcement interaction behavior has been developed. This data is particularly relevant to the work conducted by the NCDOT as it uses local, Piedmont residuum.
- Parametric studies have been performed to examine the importance of soil type, reinforcement type, and confining pressure on the load transfer mechanisms of geosynthetic-reinforced soils.
- The results of this study indicate that geotextile reinforcing materials may be a better choice than geogrid materials, particularly if minimizing displacement is an important performance consideration.
- Based on measurements of load and displacement, the use of "lower quality" soils appears feasible.

9.2. Recommendations

Based on the results of this research program, the following recommendations are made:

- For temporary earth retaining structures, it is feasible to use “lower quality” backfill in the reinforced zone. Material that satisfies the Class II – Type 2 classification in Section 1016-3 of the NCDOT Standard Specifications for Roads and Structures may be used provided the material is placed and compacted properly. As with virtually all projects employing earth as an engineering material, proper placement is absolutely critical if desired performance is to be achieved.
- It appears that the geotextile products may be the better choice of geosynthetic reinforcing material, provided the confining pressure is sufficiently large.
- The next appropriate step is to extend this laboratory-based study to the field. This may be achieved by constructing and monitoring prototype-scale temporary retaining walls either on the UNC Charlotte campus or at a more desirable location for the NCDOT personnel. These walls should be built using the same types of soil as used in this research program (Class II – Type 2) and be reinforced with, as a minimum, a representative variety of geotextiles. Performance monitoring should focus primarily on deformations (both horizontal and vertical) and should be made throughout the construction process and for at least 18 months afterward. At that point, the walls should be loaded to failure (destructive testing) to glean as much design and performance information as possible.

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