FINAL REPORT
GEOTECHNICAL INVESTIGATION AND FOUNDATION ENGINEERING FOR STURGEON CREEK BRIDGE REPLACEMENT

Prepared for
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October 28, 2011

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1.0 SUMMARY

The existing Sturgeon Creek Bridge, located north of the intersection of Portage Avenue and Sturgeon Road, will be replaced with a new bridge structure. A geotechnical investigation was conducted on July 15 and 16, 2010 to evaluate the soil conditions for the new structure. The geotechnical investigation revealed a general soil profile consisting of clay fill, clay and silt till overlying carbonate bedrock. It is our understanding that the preferred foundation systems for the new bridge structure are precast concrete piles and rock-socketed caissons. Although these foundation systems may be used to support the bridge, foundation installation will be complicated by the limited depth of overburden, presence of boulders within the silt till and heavy groundwater seepage.

2.0 TERMS OF REFERENCE

The National Testing Laboratories Limited was retained to conduct a site investigation and provide foundation recommendations for the proposed structure to replace the existing Sturgeon Creek Bridge. The scope of work for this project was outlined in our proposal dated February 10, 2010. An investigation of the subgrade conditions for the associated roads was included in our scope of work and the geotechnical report for the road works was previously submitted on October 1, 2010. The soil conditions and foundation recommendations for the proposed bridge structure are outlined in this report.

3.0 GEOTECHNICAL INVESTIGATION

3.1 Testhole Drilling and Soil Sampling

The subsurface drilling and sampling program was conducted on July 15 and 16, 2010 with drilling services provided by Paddock Drilling Ltd. under the supervision of our geotechnical field personnel. A total of nine testholes (TH1 to TH9) were drilled using a truck-mounted drill rig at the locations shown on Testhole Location Plan provided in Appendix A. Testholes TH1, TH2, TH7, TH8 and TH9 were drilled to a depth of 3.1 m to evaluate the existing pavement structure. Testholes TH3, TH4, TH5 and TH6 were drilled to depths ranging from 10 to 13 m to evaluate the soil conditions for the bridge foundation. Testholes TH4 and TH5 were drilled through the concrete bridge deck and Testholes TH3 and TH6 were drilled in the approach slabs for the bridge. The testholes were advanced to auger refusal on suspected bedrock using 125 mm diameter solid stem augers. Testholes TH3 and TH5 were completed by coring to a depth of 3 m into the carbonate bedrock. Core samples were recovered from the carbonate bedrock and grab samples were obtained directly off the augers. Undisturbed samples of the clay soil were obtained in Shelby tubes. A Standard Penetration Test was conducted in Testhole TH3 to evaluate the relative density of the silt till. The testholes were logged based upon visual, physical, and textural properties observed in the field. Upon completion of drilling, the testholes were examined for evidence of sloughing and groundwater seepage. The samples were visually classified in the field and returned to our soils laboratory for additional examination and testing.

3.2 Laboratory Testing

Water content and torvane tests were conducted on selected soil samples and the test results are shown on the attached testholes logs. Two clay samples recovered in Shelby tubes were tested for unconfined compressive strength and consolidation properties. The test data for the clay samples are summarized in the following table.

Testhole no.	Sample Depth	Soil Type	Initial Void Ratio $e_\circ$	Compression Index C_c	Recompression Index C_r	Unconfined Compressive Strength
TH3	3.5 m	Clay	1.3	0.45	0.10	170 kPa
TH3	6.0 m	Clay	1.7	0.74	0.18	59 kPa

Rock cores recovered from Testholes TH3 and TH5 were tested for unconfined compressive strength and the test results are summarized in the following table.

Testhole no.	Sample Depth	Sample Elevation	Rock Type	Uniaxial Compressive Strength
TH3	10.8 m	223.3 m	carbonate	17.6 MPa
TH3	12.2 m	221.9 m	carbonate	56.8 MPa
TH5	9.9 m	224.3 m	carbonate	24.7 MPa
TH5	12.0 m	222.2 m	carbonate	4.6 MPa

The type of failure for the core sample obtained from Testhole TH5 at a depth of 11.8 m was atypical and therefore, the compressive strength for this sample is not considered to be representative of the carbonate bedrock.

4.0 SUBSURFACE CONDITIONS

4.1 Soil Profile

Soil conditions encountered during the site investigation are shown on the testhole logs provided in Appendix B. The soil stratigraphy for the proposed bridge structure, as interpreted from the soil logs for Testholes TH3, TH4, TH5 and TH6, consists of clay fill, clay and silt till overlying carbonate bedrock.

Clay Fill

Clay fill was encountered at or near the ground surface in the testholes. The clay fill extended to depths ranging from approximately 3.3 to 3.8 m below the existing bridge deck and road surface. The clay fill contained gravel, sand and organic material. The clay fill was black, stiff to very stiff, moist, and of high plasticity. Water contents of the clay fill ranged from 24 to 47%.

Clay

Clay was encountered beneath the clay fill in the testholes. The clay was brown to grey, soft to firm, moist, and of high plasticity. Water contents of the clay ranged from 33 to 64%.

Silt Till

Silt till was encountered beneath the clay at depths from 6.7 to 7.8 m below the existing bridge deck and road surface. The silt till was tan, loose to dense, moist, of low plasticity, and contained some sand and fine to coarse gravel. Although not encountered during our field drilling program, it is our understanding that boulders are present in the silt till in this area of Winnipeg. Water contents of the silt till ranged from 14 to 25%.

Carbonate Bedrock

Carbonate bedrock was suspected at the depth of auger refusal in Testholes TH4 and TH6 and confirmed by coring in Testholes TH3 and TH5. Core recovery for the two testholes was greater than 90%. The bedrock contact for the testholes ranged from elevation 223.4 to 224.7 m. A review of the soil logs provided on the construction drawings for the existing bridge show bedrock elevations between elevation 224.3 and 224.9 m.

Core samples recovered from Testholes TH3 and TH5 revealed the upper 2 to 2.5 m of the bedrock to be poor quality. Fracture zones were observed in Testhole TH3 to a depth of approximately 2.5 m below the surface of the bedrock. A horizontal fracture in filled with clay was observed in Testhole TH5 at a depth of 2.5 m below the surface of the bedrock. The carbonate bedrock was mottled, with colour varying from white to red. Examination of the core samples revealed the upper 2 to 2.5 m of the rock mass to be fair to poor quality with good to excellent rock quality below this depth. Photographs of the core samples are provided in Appendix C. The Rock Quality Designation (RQD) of the core samples recovered from the testholes is summarized in the following table.

Testhole TH3			**Testhole TH5**		
Elevation (m)	**RQD**	**Rock Quality**	**Elevation (m)**	**RQD**	**Rock Quality**
224.2 to 222.6	59%	fair	224.7 to 223.2	45%	poor
222.6 to 221.5	83%	good	223.2 to 221.6	93%	excellent

The unconfined compressive strength of rock cores was variable, with strengths in the range of 17.6 to 56.8 MPa. Carbonate bedrock can contain zones of poor quality rock and other discontinuities problematic to foundation construction. It is not possible to fully identify these features in a geotechnical investigation because their occurrence in the local carbonate bedrock is unpredictable.

4.2 Groundwater

Heavy groundwater seepage was observed from the silt till in the testholes. The groundwater level was between an elevation of 228.7 and 229.5 m upon completion of drilling. Information obtained from Manitoba Water Stewardship indicates the potentiometric surface within the carbonate bedrock aquifer have been as high as 232.5 m near the project site. A hydrograph from an observation well at the Grace Hospital and a contour map for the carbonate aquifer potentiometric surface are provided in Appendix D. Soil sloughing was typically observed within the silt till. It should be noted that only short-term seepage and sloughing conditions were observed in the testholes and groundwater levels will normally fluctuate during the year. Based upon the hydrograph for the observation well at the Grace Hospital, the seasonal variation in the carbonate aquifer potentiometric surface is approximately 2 m.

5.0 GEOTECHNICAL CONSIDERATIONS

Based upon our current understanding of the proposed bridge construction and the findings from our geotechnical investigation, the primary geotechnical concerns for this project are:

- Limited depth of overburden
- Boulders within the silt till
- Heavy groundwater seepage and sloughing conditions within the silt till
- High potentiometric surface in the carbonate bedrock aquifer
- Poor quality bedrock near the bedrock surface

Details on the potential impact of these issues on foundation construction are provided in the following sections.

6.0 FOUNDATION RECOMMENDATIONS AND COMMENTS

It is our understanding that the preferred foundation systems for the new bridge structure are precast concrete piles and rock-socketed caissons. The existing bridge is supported on precast concrete piles and there have been no reported concerns regarding foundation performance for the bridge. However, it was reported by the foundation contractor for the existing bridge that it was very difficult to maintain alignment of the precast concrete piles during foundation construction due to the limited depth of overburden and presence of boulders within the silt till. Foundation specifications and drawings should be submitted for our review before they are issued for tender.

It was reported that the elevation of the new bridge structure may be 3 to 4 m higher than the elevation of the existing structure. It should be noted that placement of fill materials to increase the elevation of the approach roads for the bridge will lead to down drag forces on the abutment foundations. An assessment of the down drag forces for the bridge foundation should be undertaken after details of the bridge elevation have been finalized.

6.1 Precast Concrete Piles

A foundation system suitable for the proposed bridge structure is a system of driven, prestressed, precast concrete piles. These units, when driven to practical refusal with a hammer capable of delivering a minimum rated energy of 40 KJ per blow, may be assigned the following allowable loads.

Nominal Pile Size	Allowable Load	Refusal Criteria
300 mm	450 kN	5 blows/25 mm
350 mm	625 kN	8 blows/25 mm
400 mm	800 kN	12 blows/25 mm

Pile spacing should not be less than 2.5 pile diameters, measured center to center. The top elevation for precast piles should be recorded immediately after driving and where pile heave is observed, piles should be redriven. The limited depth of overburden and the presence of boulders within the silt till will complicate installation of driven precast concrete piles on the project site. It was reported that difficult conditions were encountered during installation of the precast concrete piles for the existing Sturgeon Creek bridge. There is a high risk that precast concrete piles will be out of alignment after installation where there is limited depth of overburden. To enhance pile alignment, pre-boring is recommended. The prebored hole diameter should be similar to the nominal pile diameter to ensure the piles are in contact with the soil and therefore, maintain lateral pile capacity. All piles should be driven continuously to their required depth once driving is initiated. Precast concrete piles driven to practical refusal will develop the majority of their capacity from toe resistance, and therefore, no reduction in pile capacity is necessary for group action. The design capacity of a pile group is equal to the number of piles in the group times the allowable capacity per pile. A minimum void space of 150 mm should be provided beneath all structural elements to accommodate potential heave of the high plasticity clay fill and clay.

A summary of the bedrock elevations at the testhole locations is provided in the following table.

Testhole no.	Bedrock Elevation
TH3	224.2 m
TH4	223.4 m
TH5	224.7 m
TH6	224.0 m

Based upon the information obtained from our investigation and the soil logs provided on the construction drawings for the existing bridge, driven piles are expected to reach refusal on bedrock at an elevation between 223 and 225 m.

Negligible settlement beyond the elastic compression of the pile can be expected with an end-bearing pile system. To ensure that the piles achieve their design capacities, full time inspection by qualified geotechnical personnel is recommended during pile installation.

6.2 Rock-Socketed Caissons

Cast-in-place concrete caissons socketed into the underlying carbonate bedrock may be utilized to support the proposed bridge structure. Although the capacity of rock-socketed caissons may be derived from both toe resistance and shaft resistance, inspection is required to ensure the caisson base properly cleaned if toe resistance is to be used to estimate the socket capacity. Due to the high potentiometric surface within the carbonate bedrock, it is anticipated that heavy groundwater seepage will prevent visual inspection of the caisson base and therefore, the design of the rock-socketed caissons for this project should be based upon the shaft resistance over the length of the caisson embedded into the bedrock. The following table provides the recommended allowable shaft resistance values for design of rock-socketed caissons.

Bedrock Quality	Depth Below the Bedrock Surface	Allowable Shaft Resistance
Fair to poor bedrock (RQD <60%)	0 to approximately 2.5 m	150 kPa
Good to excellent bedrock (RQD >90%)	Below approximately 2.5 m	1000 kPa

The shaft resistance values provided in the table above are based upon an evaluation of the rock cores recovered from Testholes TH3 and TH5, and a minimum concrete compressive strength of 40 MPa for the rock-socketed caissons. It should be noted that the quality of bedrock can change significantly over short distances and actual bedrock quality at the caisson locations may differ from the bedrock quality observed in the core samples. If the actual bedrock quality is less than the quality assumed in the design of the caisson, the rock socket length must be increased to ensure the design capacity is achieved. It is recommended that the tender documents include a provision for an increase in the shaft lengths if poor quality bedrock is encountered.

Rock sockets should be embedded a minimum of two caisson diameters or 2 m below the fair to poor quality bedrock, whichever is greater. Rock-socketed caissons should not be spaced closer than 2.5 diameters centre to centre. Local foundation contractors should be contacted to verify the available shaft sizes prior to finalizing the shaft diameters for the caissons. A minimum void space of 150 mm should be provided beneath all structural elements to accommodate potential heave of the high plasticity clay soil.

Heavy groundwater seepage and sloughing from the silt till should be anticipated during installation of the rock-socketed caissons. Although not encountered during our investigation, boulders and cobbles may be encountered within the silt till during caisson construction. Highly fractured rock and clay seams may be encountered within the bedrock. The contractor should be adequately equipped to deal with these conditions during construction. To reduce the risks associated with construction of rock-socketed caissons on the project site, the installation of a test caisson is recommended.

Temporary casings should be used to prevent soil sloughing into the caisson and pumps should be available to dewater the caissons prior to concrete placement. If excessive groundwater flows are encountered, concrete for the caissons should be placed using tremie procedures. Good quality tremie concrete is obtained through a continuous placement at a constant placement rate. Any prolonged interruption in concrete placement imposes high risks for defective concrete. Although the caisson capacity is not derived from end-bearing, the caisson base should be cleaned to remove sediment prior to concrete placement. Concrete should be placed immediately after the caisson has been inspected and approved to minimize the risk of sloughing within the caisson.

Full time inspection by qualified geotechnical personnel is required to evaluate the bedrock quality and to make recommendations regarding any requirement for socket deepening. A camera should be used to confirm the quality of the bonding surface for each caisson prior to concrete placement.

7.0 LATERAL LOAD ANALYSIS

Analyses were undertaken for both fixed and pinned pile head conditions for precast concrete piles. The lateral resistance provided by a pile cap was neglected. The output of the lateral analysis is provided in Appendix D. Incorporating the pile cap into the analysis may reduce the associated lateral deflections by approximately 50%. It is recommended that the lateral load capacity of precast concrete piles be confirmed when the foundation design has been completed.

8.0 FOUNDATION CONCRETE

The clay soils in Winnipeg and surrounding areas contain sulphates that will cause deterioration of concrete. The class of exposure for concrete in contact with clay soil in the Winnipeg and surrounding areas is considered to be severe (S-2 in CSA A23.1-09 Table 3). The requirements for concrete exposed to severe sulphate attack are provided in the following table:

Parameter	Design Requirement
Class of exposure	S-2
Compressive strength	32 MPa at 56 days
Air content	4 to 7%
Water-to-cementing materials ratio	0.45 max.
Cement	Type HS or HSb

It should be noted that the strength requirements for structural purposes may exceed the strength requirements for sulphate resistance.

9.0 SLOPE STABILITY EVALUATION

At the time of writing this report, details on the side slopes for the creek bank were not available. It is our understanding that the new bridge structure may be 3 to 4 m higher than the existing structure. It should be noted that placement of fill materials to increase the elevation of the approach roads for the bridge may impact the stability of the creek bank. A stability analysis of the creek bank should be undertaken after details of the bridge elevation and side slopes for the creek bank have been finalized.

10.0 CLOSURE

Professional judgments and recommendations are presented in this report. They are based partly on an evaluation of the technical information gathered during our site investigation and partly on our general experience with subsurface conditions in the area. We do not guarantee the performance of the project in any respect other than that our engineering work and judgment rendered meet the standards and care of our profession. It should be noted that the testholes may not represent potentially unfavorable subsurface conditions between testholes. If during construction soil conditions are encountered that vary from those discussed in this report, we should be notified immediately in order that we may evaluate effects, if any, on the foundation performance. The recommendations presented in this report are applicable only to this specific site. These data should not be used for other purposes.

We appreciate the opportunity to assist you in this project. Please call the undersigned should you have any questions regarding this report.

Prepared by

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Reviewed by

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Senior Geotechnical Engineer

APEGM
Certificate of Authorization
The National Testing Laboratories Limited
No. 690 Date: October 28, 2011

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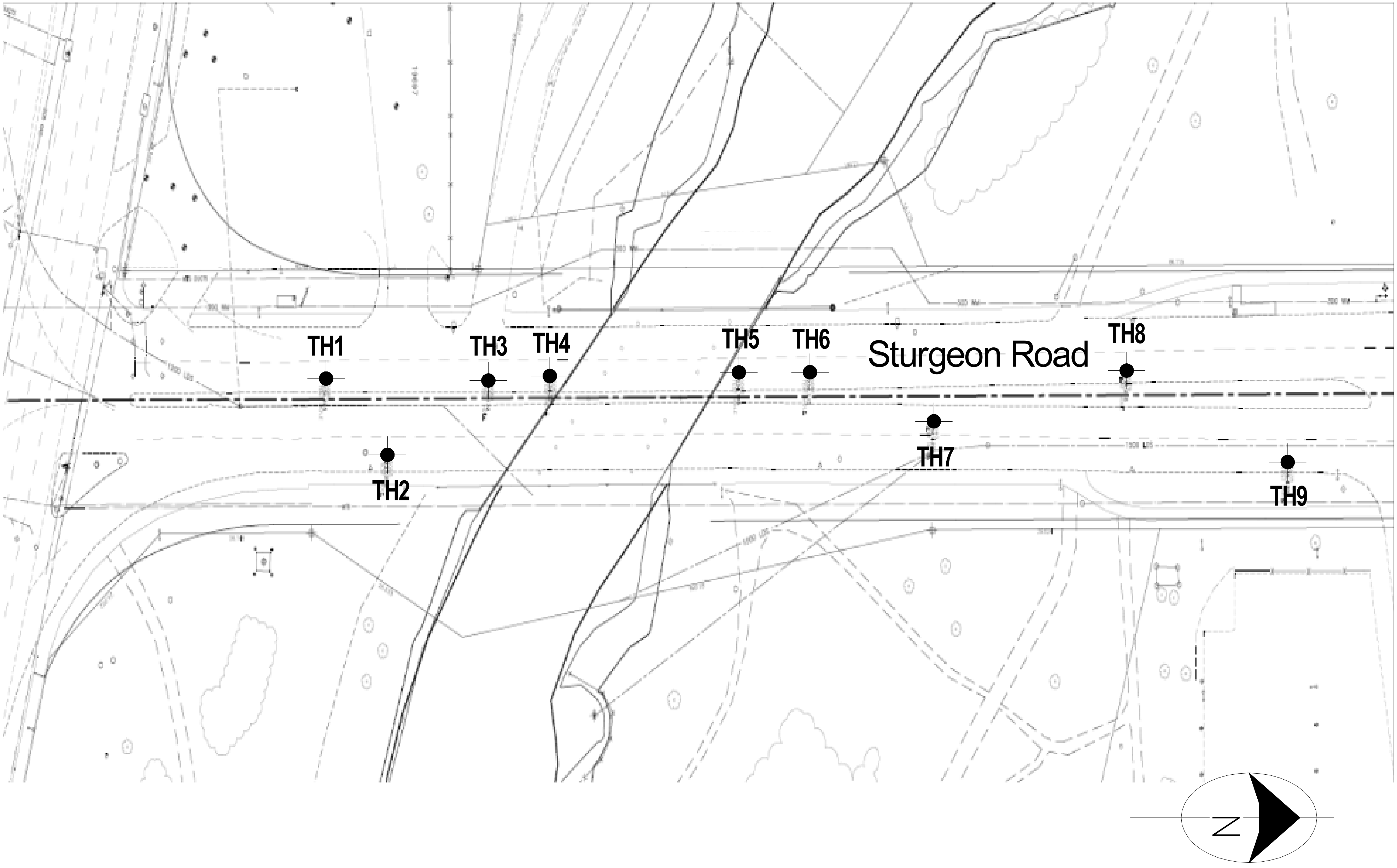
APPENDIX A

TESTHOLE LOCATION PLAN



THE NATIONAL TESTING LABORATORIES LIMITED
Established in 1923

Project No. STA-1029
Date:October 4, 2010
Drawn by: D.M
Reviewed by: DF
Figure: 1
Scale: NTS

Testhole Location Plan
Replacement of Sturgeon Creek Bridge
Winnipeg, Manitoba

APPENDIX B

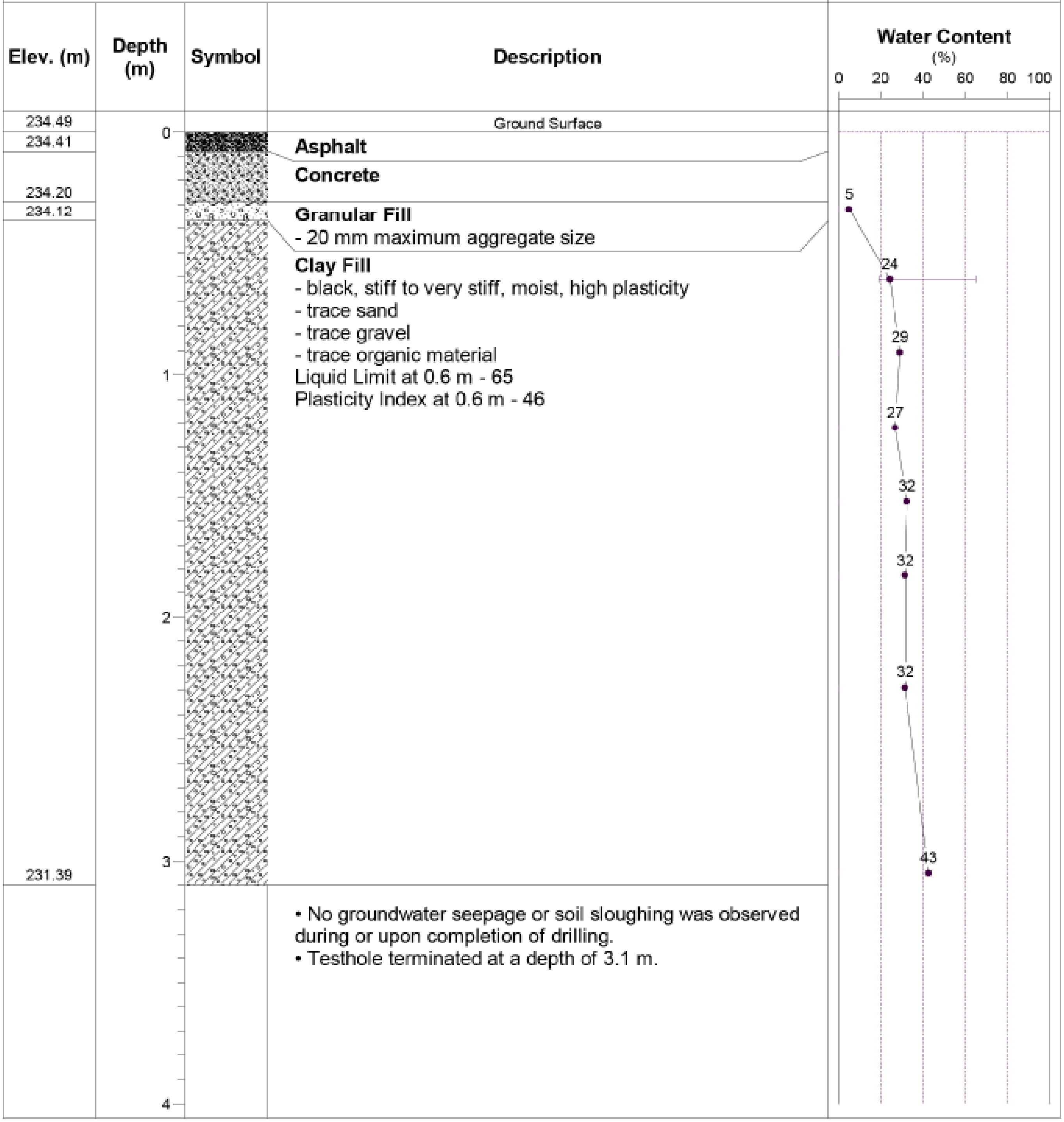
TESTHOLE LOGS

TESTHOLE TH1

Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 16, 2010
Depth of Testhole: 3.1 m
Testhole Elevation: 234.49 m
Logged By: Larry Presado

Subsurface Profile | **Laboratory Testing**



Ground Surface

Asphalt

Concrete

Granular Fill
- 20 mm maximum aggregate size

Clay Fill
- black, stiff to very stiff, moist, high plasticity
- trace sand
- trace gravel
- trace organic material

Liquid Limit at 0.6 m - 65
Plasticity Index at 0.6 m - 46

- No groundwater seepage or soil sloughing was observed during or upon completion of drilling.
- Testhole terminated at a depth of 3.1 m.

TESTHOLE TH2

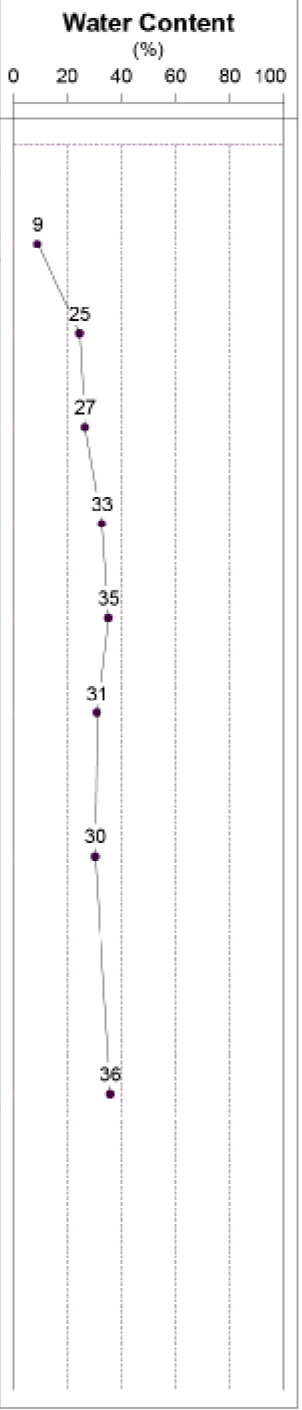
Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 16, 2010
Depth of Testhole: 3.1 m
Testhole Elevation: 233.99 m
Logged By: Larry Presado

Subsurface Profile

Elev. (m)	Depth (m)	Symbol	Description
233.99			Ground Surface
233.91	0		**Asphalt**
233.71			**Concrete**
233.62			**Granular Fill** - 20 mm maximum aggregate size
230.89	1 2 3		**Clay Fill** - black, stiff, moist, high plasticity - trace fine to coarse sand - trace organic material
	4		• No groundwater seepage or soil sloughing observed during or upon completion of drilling. • Testhole terminated at a depth of 3.1 m.

Laboratory Testing



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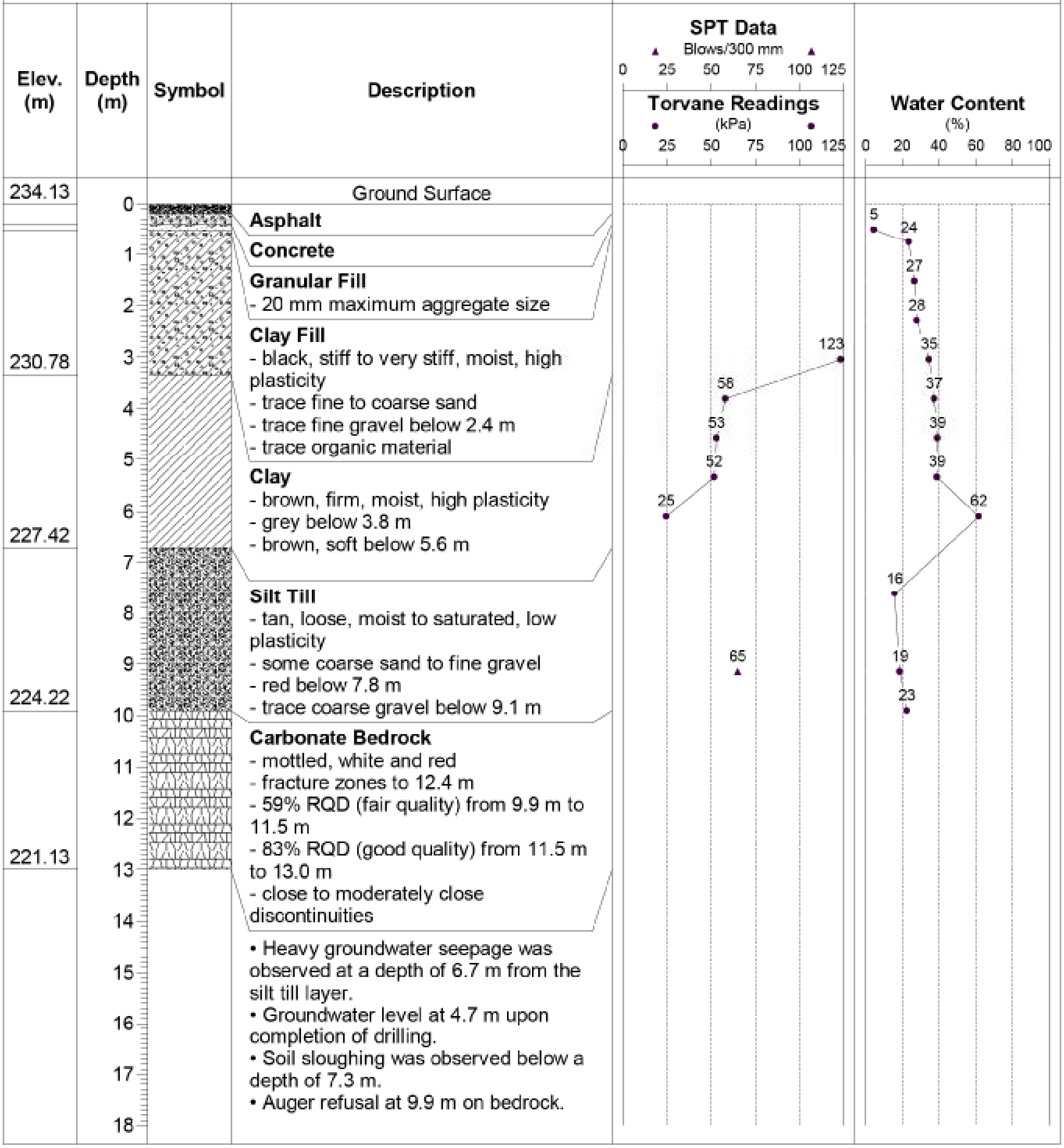
TESTHOLE TH3

Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger and 50 mm Core Bit

Date Drilled: July 15, 2010
Depth of Testhole: 13.0 m
Testhole Elevation: 234.13 m
Logged By: Larry Presado

Subsurface Profile | **Field / Laboratory Testing**



Ground Surface

Asphalt

Concrete

Granular Fill
- 20 mm maximum aggregate size

Clay Fill
- black, stiff to very stiff, moist, high plasticity
- trace fine to coarse sand
- trace fine gravel below 2.4 m
- trace organic material

Clay
- brown, firm, moist, high plasticity
- grey below 3.8 m
- brown, soft below 5.6 m

Silt Till
- tan, loose, moist to saturated, low plasticity
- some coarse sand to fine gravel
- red below 7.8 m
- trace coarse gravel below 9.1 m

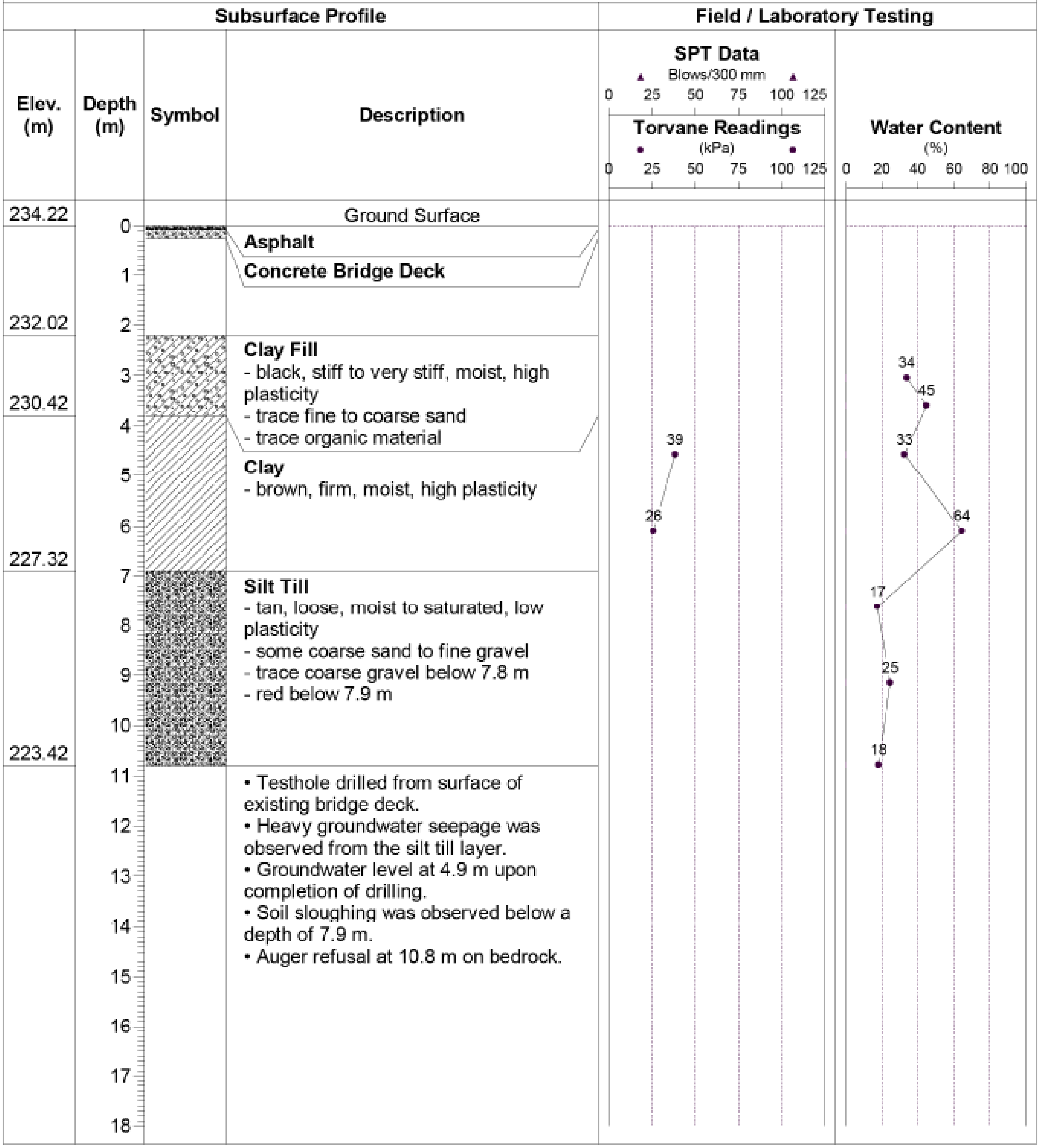
Carbonate Bedrock
- mottled, white and red
- fracture zones to 12.4 m
- 59% RQD (fair quality) from 9.9 m to 11.5 m
- 83% RQD (good quality) from 11.5 m to 13.0 m
- close to moderately close discontinuities

- Heavy groundwater seepage was observed at a depth of 6.7 m from the silt till layer.
- Groundwater level at 4.7 m upon completion of drilling.
- Soil sloughing was observed below a depth of 7.3 m.
- Auger refusal at 9.9 m on bedrock.

TESTHOLE TH4

Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 15, 2010
Depth of Testhole: 10.8 m
Testhole Elevation: 234.22 m
Logged By: Larry Presado



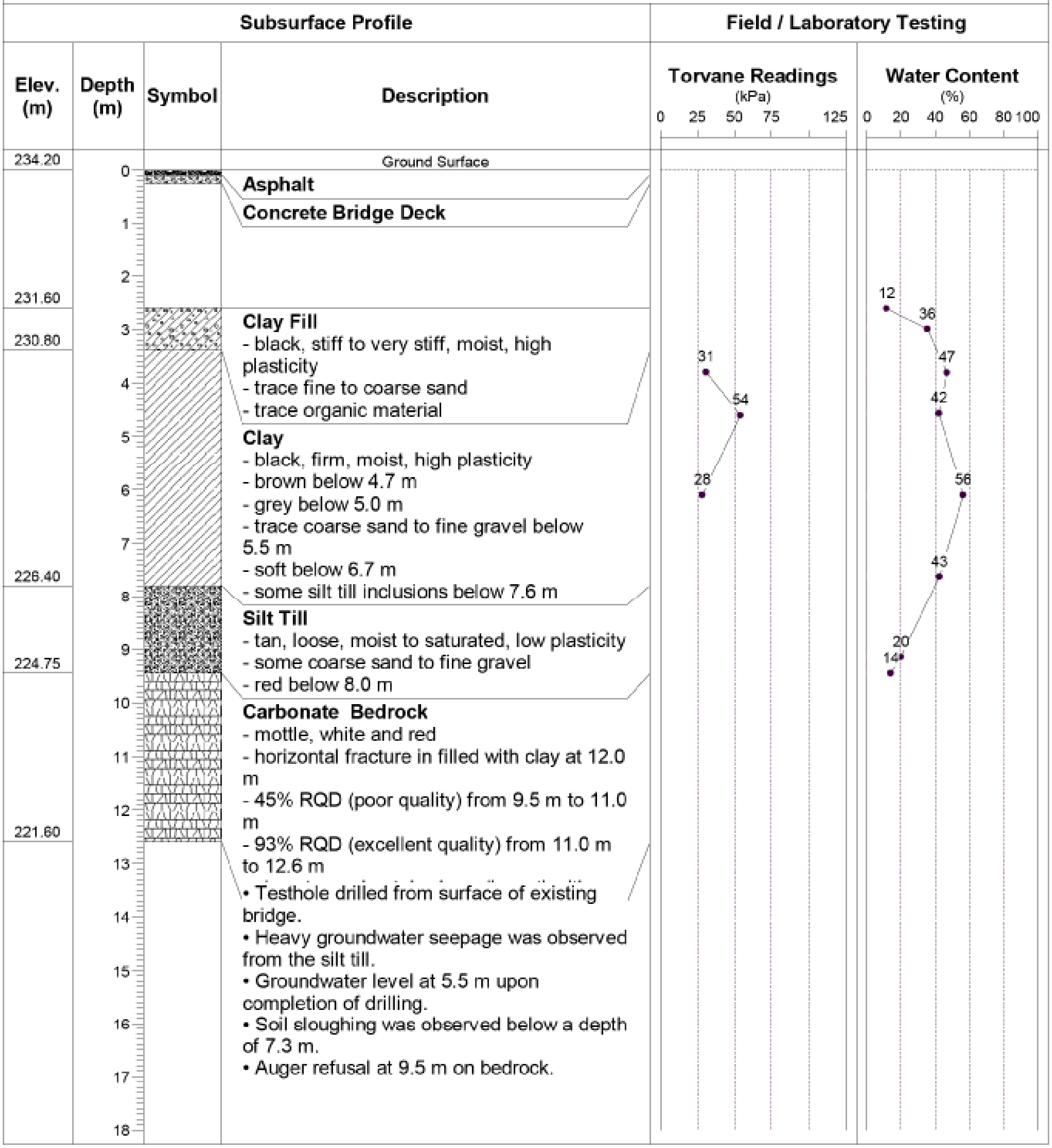
Elev. (m)	Description
234.22	Ground Surface
	Asphalt
	Concrete Bridge Deck
232.02	**Clay Fill** - black, stiff to very stiff, moist, high plasticity - trace fine to coarse sand - trace organic material
230.42	**Clay** - brown, firm, moist, high plasticity
227.32	**Silt Till** - tan, loose, moist to saturated, low plasticity - some coarse sand to fine gravel - trace coarse gravel below 7.8 m - red below 7.9 m
223.42	

- Testhole drilled from surface of existing bridge deck.
- Heavy groundwater seepage was observed from the silt till layer.
- Groundwater level at 4.9 m upon completion of drilling.
- Soil sloughing was observed below a depth of 7.9 m.
- Auger refusal at 10.8 m on bedrock.

TESTHOLE TH5

Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger and 25 mm Core Bit

Date Drilled: July 15, 2010
Depth of Testhole: 12.6 m
Testhole Elevation: 234.20 m
Logged By: Larry Presado



Asphalt

Concrete Bridge Deck

Clay Fill
- black, stiff to very stiff, moist, high plasticity
- trace fine to coarse sand
- trace organic material

Clay
- black, firm, moist, high plasticity
- brown below 4.7 m
- grey below 5.0 m
- trace coarse sand to fine gravel below 5.5 m
- soft below 6.7 m
- some silt till inclusions below 7.6 m

Silt Till
- tan, loose, moist to saturated, low plasticity
- some coarse sand to fine gravel
- red below 8.0 m

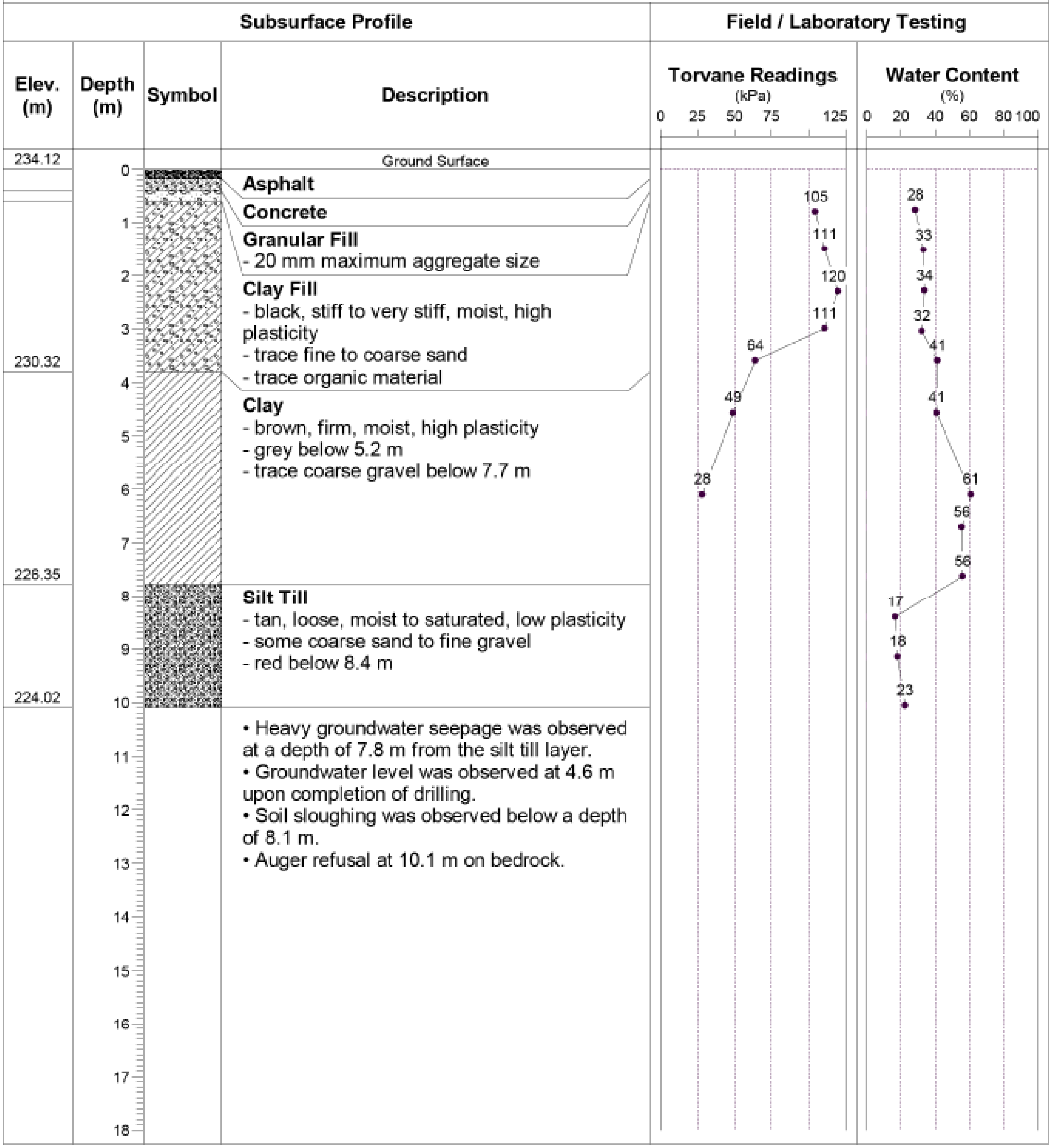
Carbonate Bedrock
- mottle, white and red
- horizontal fracture in filled with clay at 12.0 m
- 45% RQD (poor quality) from 9.5 m to 11.0 m
- 93% RQD (excellent quality) from 11.0 m to 12.6 m

• Testhole drilled from surface of existing bridge.
• Heavy groundwater seepage was observed from the silt till.
• Groundwater level at 5.5 m upon completion of drilling.
• Soil sloughing was observed below a depth of 7.3 m.
• Auger refusal at 9.5 m on bedrock.

TESTHOLE TH6

Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 15, 2010
Depth of Testhole: 10.1 m
Testhole Elevation: 234.12
Logged By: Larry Presado



Ground Surface

Asphalt

Concrete

Granular Fill
- 20 mm maximum aggregate size

Clay Fill
- black, stiff to very stiff, moist, high plasticity
- trace fine to coarse sand
- trace organic material

Clay
- brown, firm, moist, high plasticity
- grey below 5.2 m
- trace coarse gravel below 7.7 m

Silt Till
- tan, loose, moist to saturated, low plasticity
- some coarse sand to fine gravel
- red below 8.4 m

- Heavy groundwater seepage was observed at a depth of 7.8 m from the silt till layer.
- Groundwater level was observed at 4.6 m upon completion of drilling.
- Soil sloughing was observed below a depth of 8.1 m.
- Auger refusal at 10.1 m on bedrock.

TESTHOLE TH7

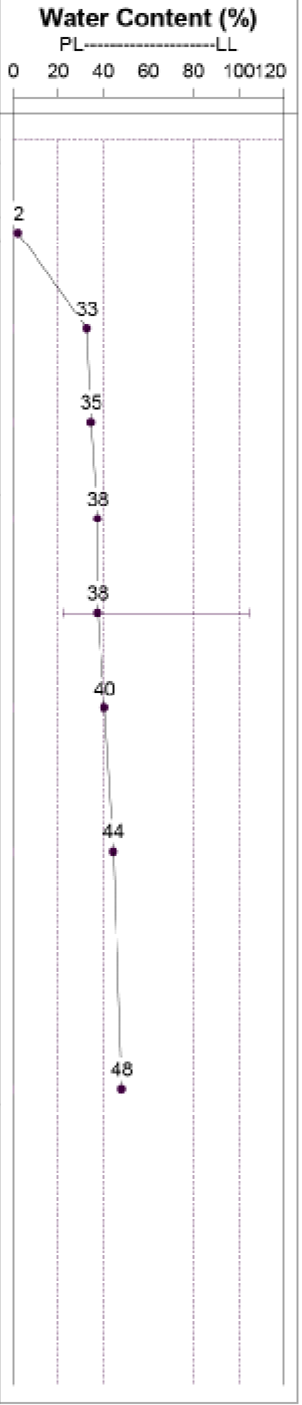
Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 16, 2010
Depth of Testhole: 3.1 m
Testhole Elevation: 234.43 m
Logged By: Larry Presado

Subsurface Profile

Elev. (m)	Depth (m)	Symbol	Description
234.43			Ground Surface
234.36	0		**Asphalt**
234.18			**Concrete**
234.11			**Granular Fill** - 20 mm maximum aggregate size
	1		**Clay Fill** - black, stiff, moist, high plasticity - trace fine to coarse sand - trace organic material
233.29	2 3		**Clay** - brown, firm, moist, high plasticity - trace silt Liquid Limit at 1.5 m - 105 Plasticity Index at 1.5 m - 83
231.33	4		• No groundwater seepage or soil sloughing observed during or upon completion of drilling. • Testhole terminated at a depth of 3.1 m.

Laboratory Testing



TESTHOLE TH8

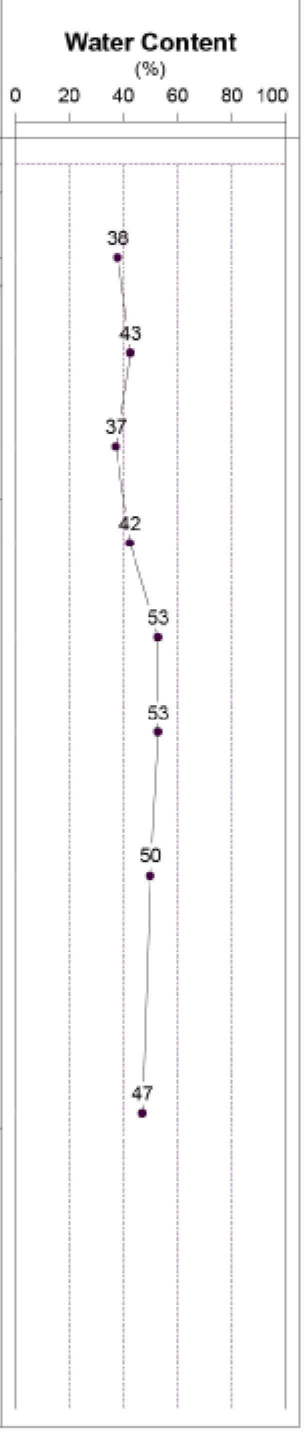
Project Name:
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 16, 2010
Depth of Testhole: 3.1 m
Testhole Elevation: 234.92 m
Logged By: Larry Presado

Subsurface Profile

Elev. (m)	Depth (m)	Symbol	Description
234.92			Ground Surface
234.83	0		**Asphalt**
234.62			**Concrete**
234.53			**Granular Fill** - 20 mm maximum aggregate size
233.84	1		**Clay Fill** - black, stiff, moist, high plasticity - trace fine gravel - trace organic material
231.82	2 3		**Clay** - brown, stiff, moist, high plasticity - trace silt below 1.2 m
	4		• No groundwater seepage or soil sloughing observed during or upon completion of drilling. • Testhole terminated at a depth of 3.1 m.

Laboratory Testing



TESTHOLE TH9

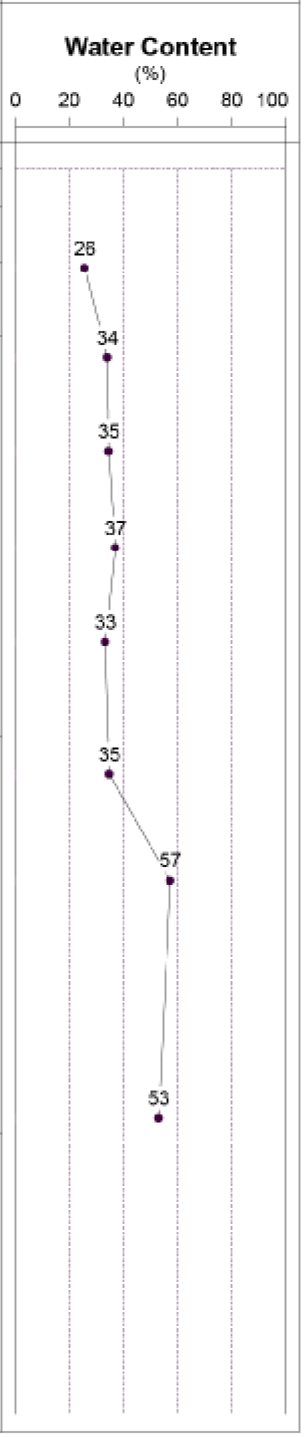
Project Name: Replacement of Sturgeon Creek Bridge
Client: Stantec Consulting Ltd.
Drilling Contractor: Paddock Drilling Ltd.
Drilling Method: 125 mm Auger

Date Drilled: July 16, 2010
Depth of Testhole: 3.1 m
Testhole Elevation: 235.44 m
Logged By: Larry Presado

Subsurface Profile

Elev. (m)	Depth (m)	Symbol	Description
235.44	0		Ground Surface
235.32		▓	**Asphalt**
235.14		▒	**Concrete**
234.90		░	**Granular Fill** - 20 mm maximum aggregate size
233.61	1	▩	**Clay Fill** - black, stiff, moist, high plasticity - trace sand from 1.4 m to 1.8 m - trace organic material
232.34	2 3	▨	**Clay** - brown, firm, moist, high plasticity - trace silt
	4		• No groundwater seepage or soil sloughing observed during or upon completion of drilling. • Testhole terminated at a depth of 3.1 m.

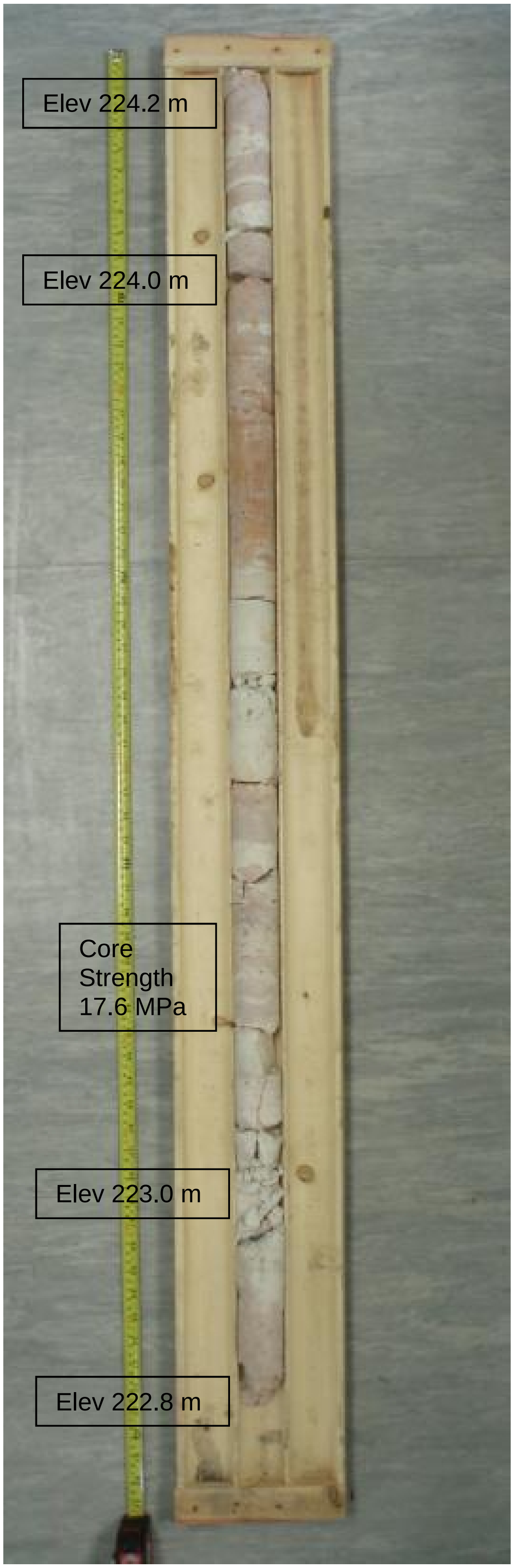
Laboratory Testing



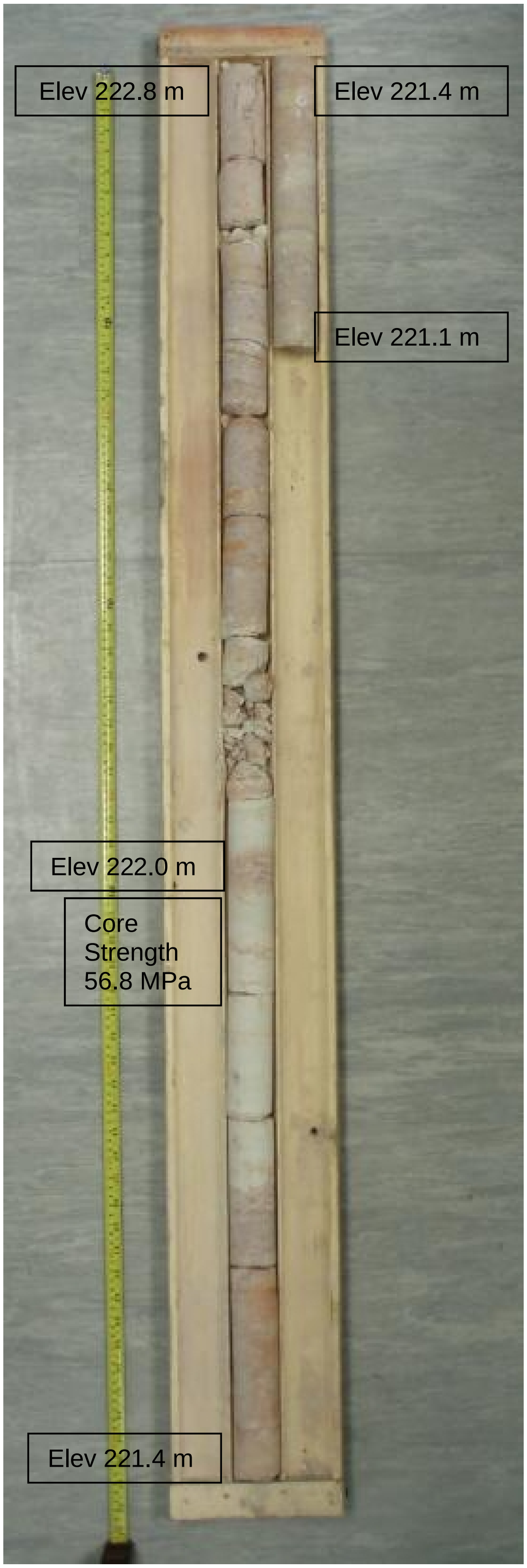
APPENDIX C

CARBONATE BEDROCK CORE PHOTOGRAPHS

TESTHOLE TH3

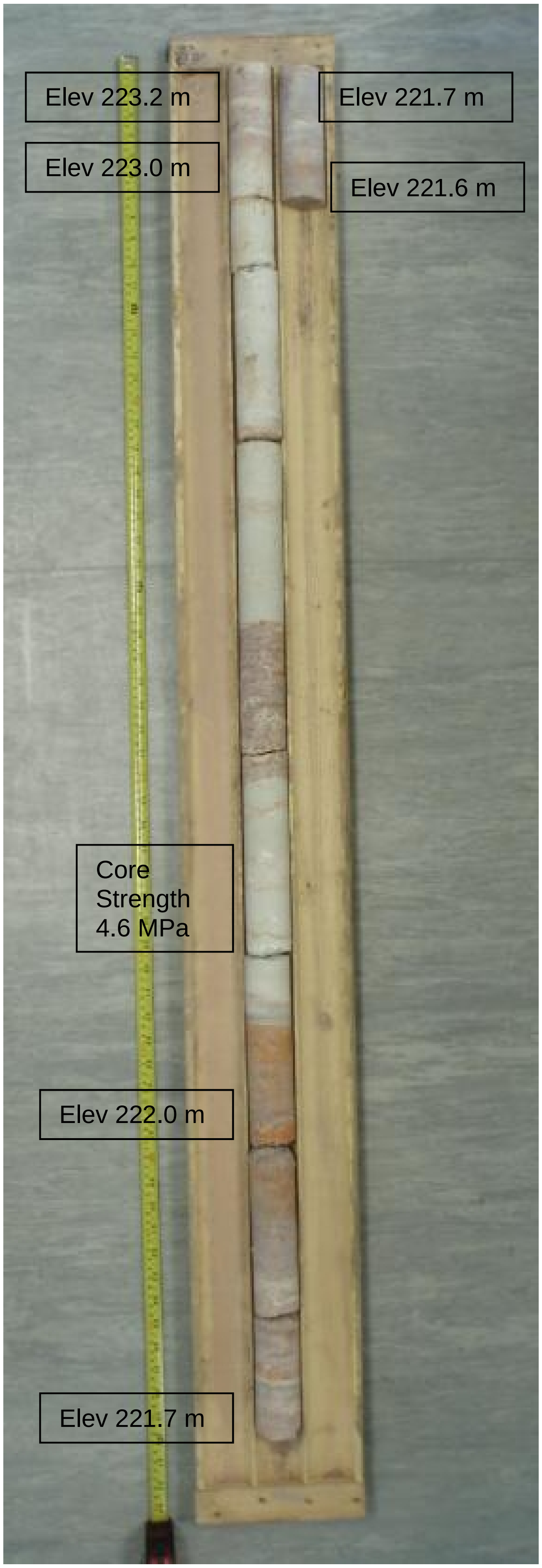


TESTHOLE TH3



TESTHOLE TH5

TESTHOLE TH5

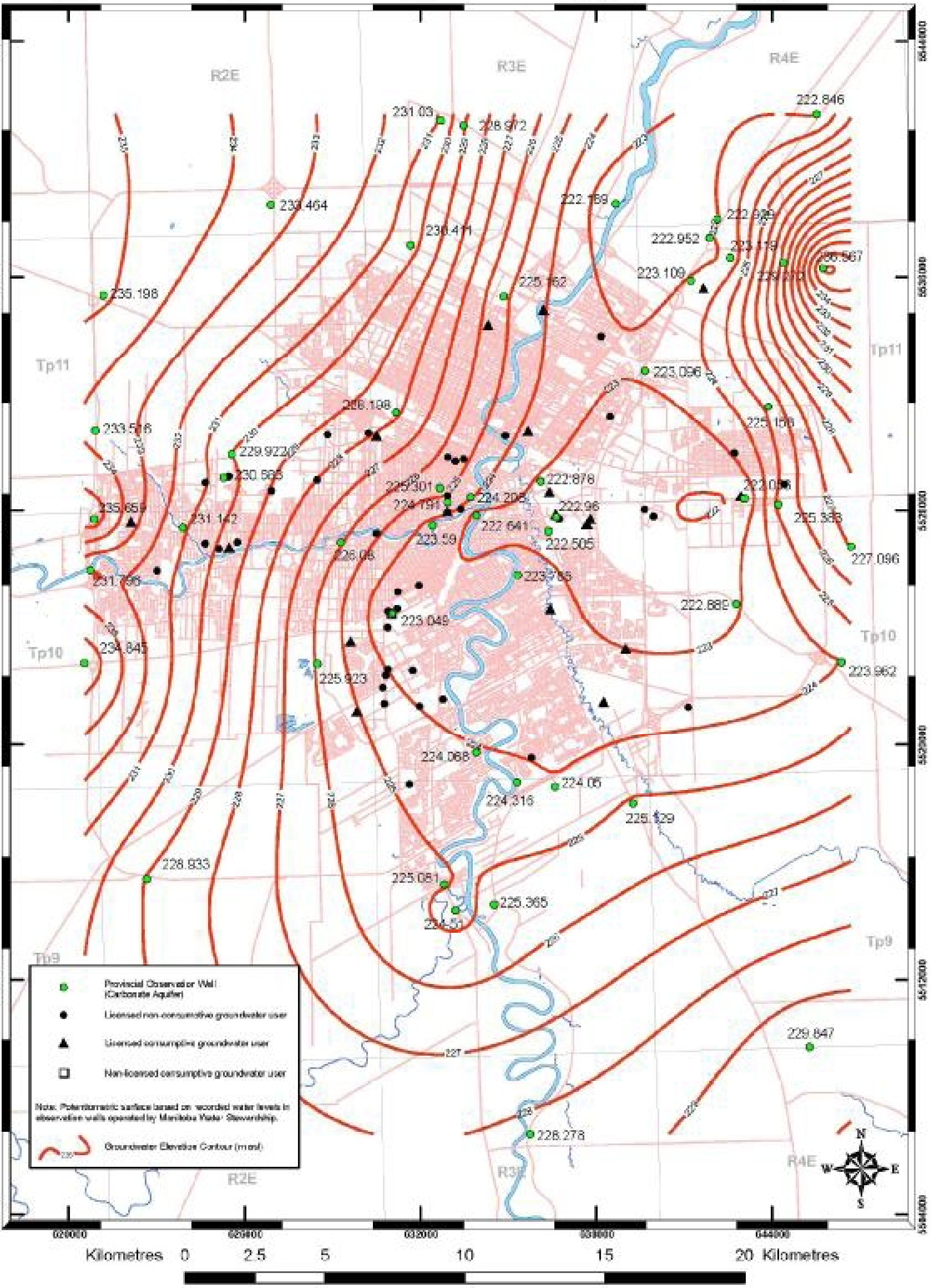


APPENDIX D

CARBONATE AQUIFER POTENTIOMETRIC SURFACE

Carbonate Aquifer Potentiometric Surface
Winnipeg Area

December 15, 2006



UTM NAD83, Zone 14

Manitoba
Water Stewardship

Groundwater Management Section
June 2007

Manitoba Water Stewardship
Water Branch
WELL INFORMATION REPORT

Manitoba

2010 Dec 06

Well PID: 75545

LOCATION: RIVER LOT 0012 IN PARISH OF St. James
UTMX:623676 UTMY:5526853 XY Accuracy:1 EXACT [<5M] [GPS]
Owner: GRACE HOSPITAL/WRB
Driller: Friesen Drillers Ltd.
Well Name: G05MJ076 GRACE RETURN #1
Date Completed: 1992 May 14
Well Use: OBSERVATION
Well Status: ACTIVE Aquifer: LIMESTONE OR DOLOMITE

REMARKS:
WINNIPEG REGION - CURRENTLY USED AS A WRB MONITORING STATION FOR WATER LEVELS (1992-). SW CORNER OF PARKING LOT AT STURGEON RD, RECHARGE WELL #1 USED AS OBS WELL

WELL LOG (Imperial units)

From	To(ft.)	Log
0.0	25	CLAY
25.0	32	TILL
32.0	87.9	SHALE WITH LIMESTONE LAYERS
87.9	349.8	LIMESTONE

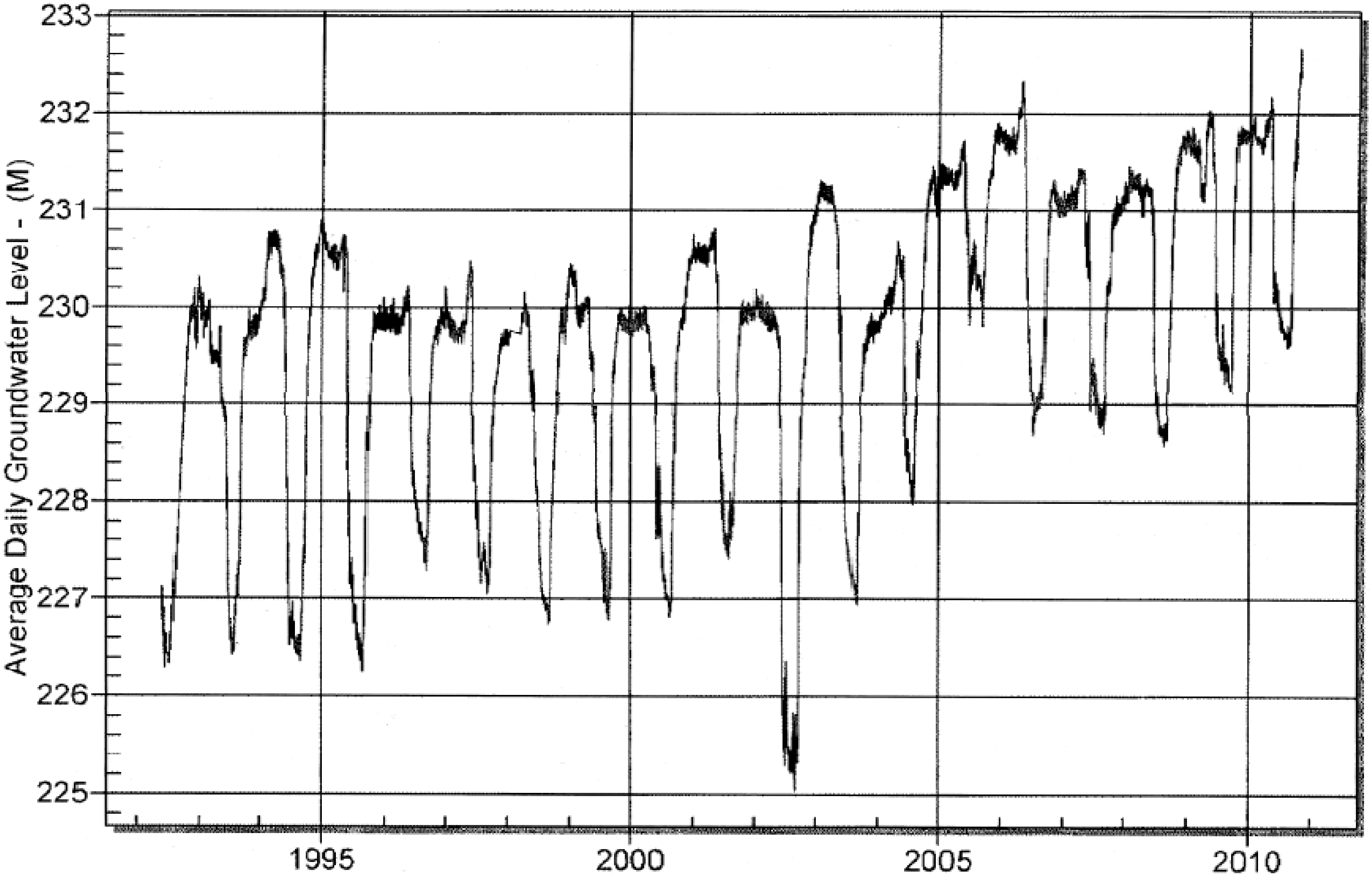
WELL CONSTRUCTION

From	To(ft)	Const.Method	Inside Dia.(in)	Outside Dia.(in)	Slot Size(in)	Type	Material
0.0	88.9	casing	10.0			WELDED	BLACK IRON
88.9	349.8	open hole	9.0				
4.0	88.9	casing grout					CEMENT

Top of Casing: 1.0 ft. above ground

G05MJ076 GRACE HOSPITAL 012 ST JAMES

CASING HEIGHT IS



Prepared by Manitoba Water Stewardship 06 Dec 2010

APPENDIX E

LATERAL STABILITY ANALYSIS OUTPUT

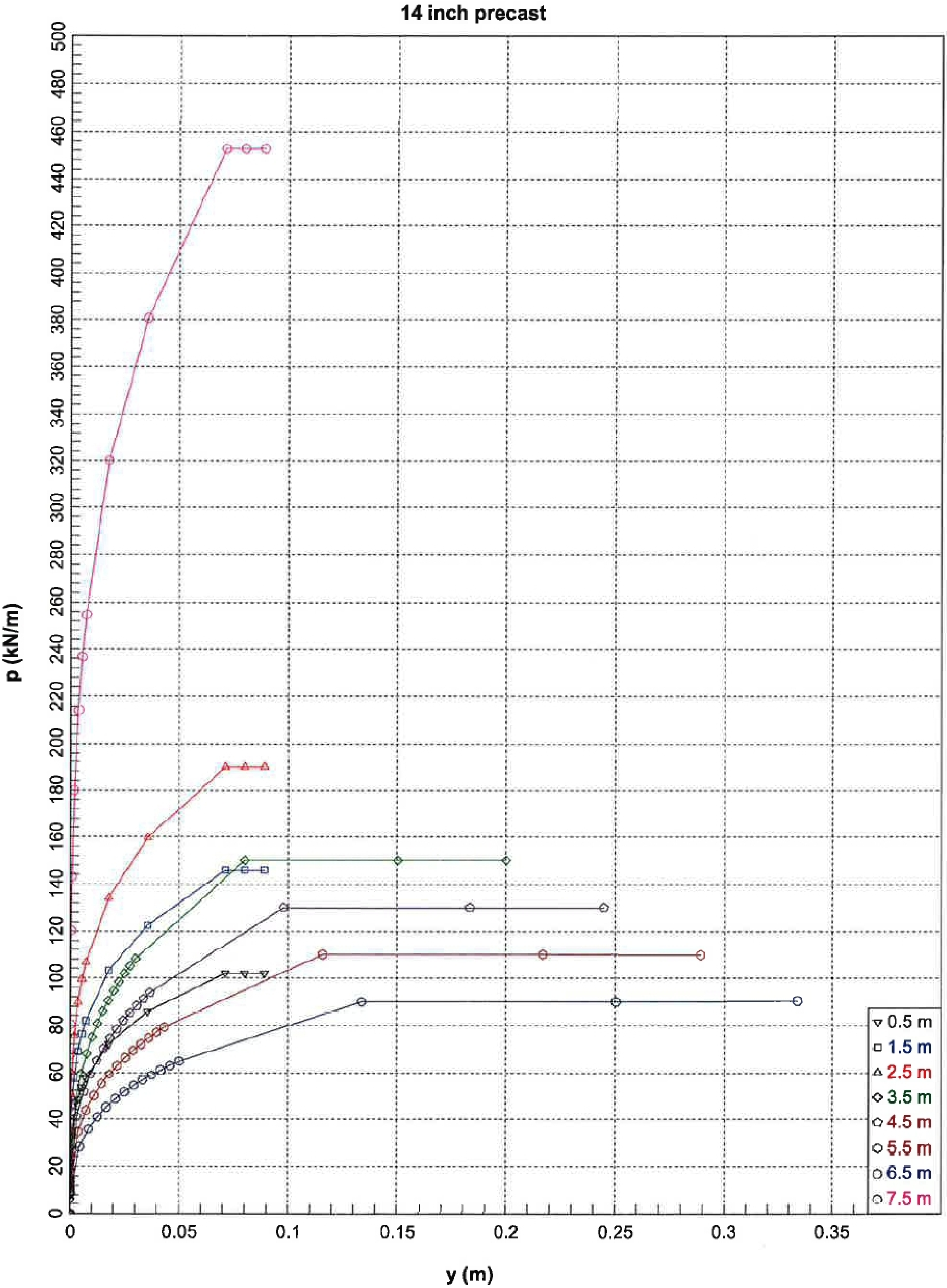
14 inch precast
p (kN/m)
y (m)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
▽ 0.5 m
□ 1.5 m
△ 2.5 m
◇ 3.5 m
⬠ 4.5 m
○ 5.5 m
○ 6.5 m
○ 7.5 m

FIGURE 1

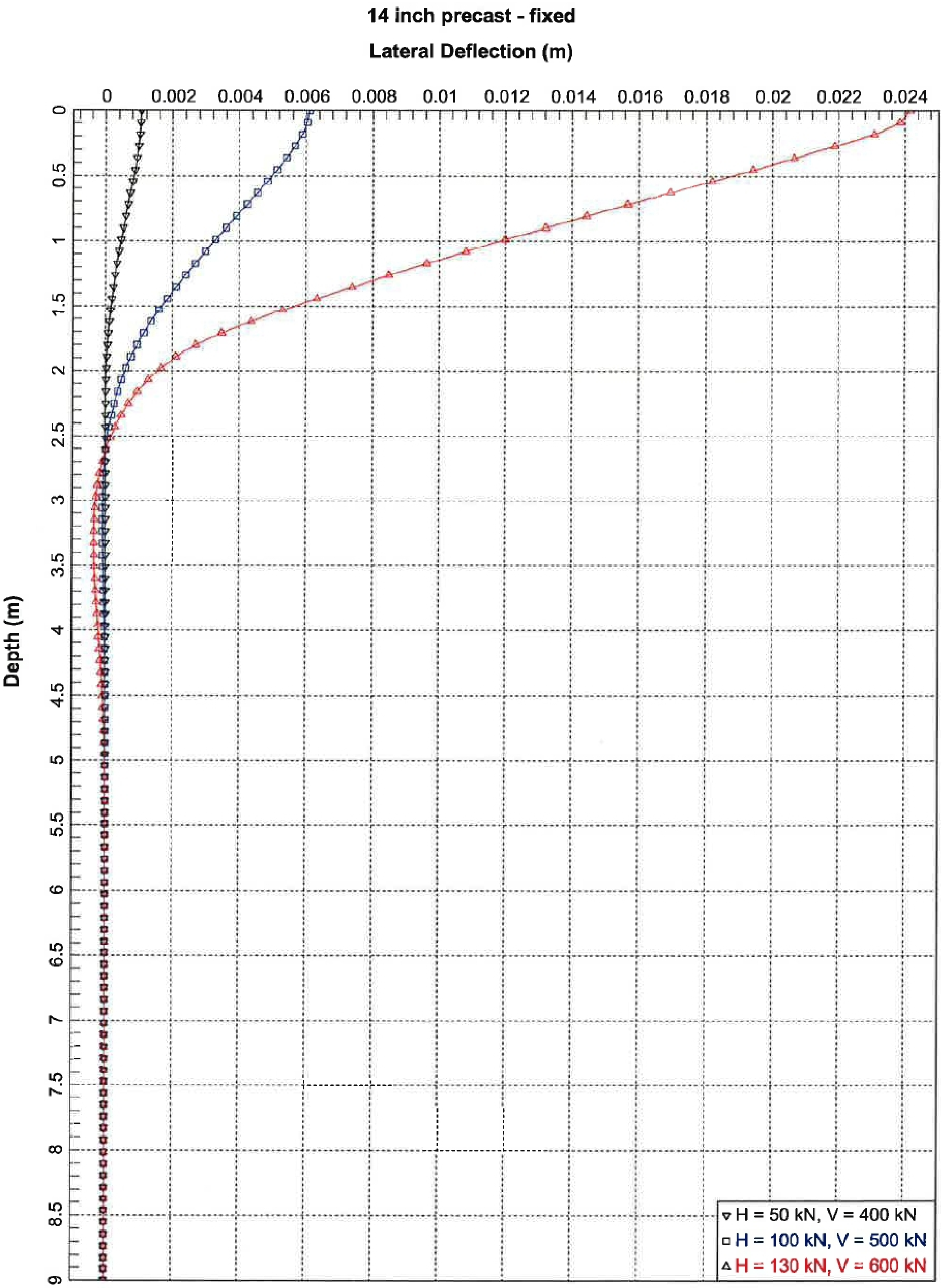

14 inch precast - fixed
Lateral Deflection (m)
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
0.022
0.024
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 130 kN, V = 600 kN

FIGURE 2

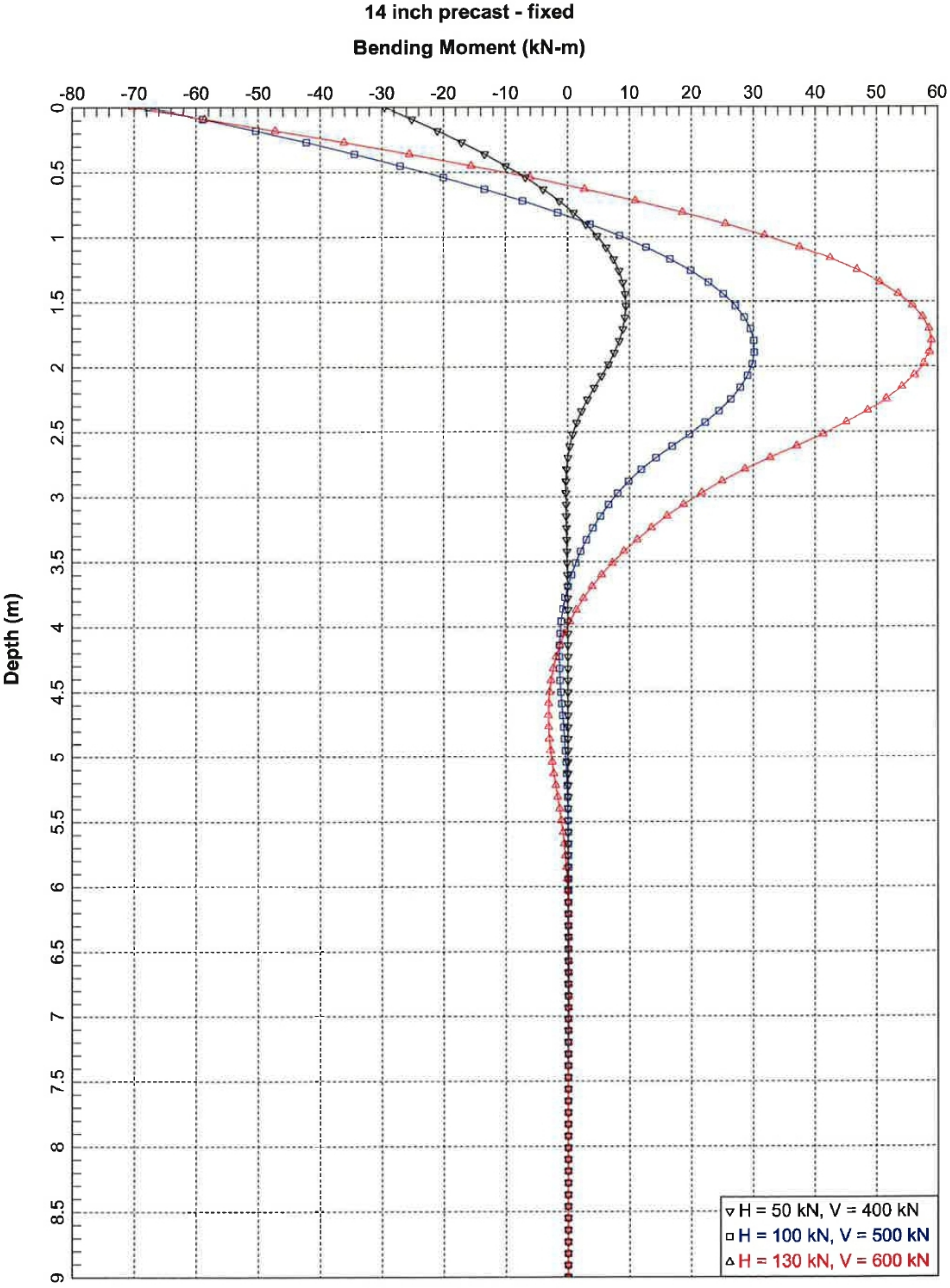

14 inch precast - fixed
Bending Moment (kN-m)
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 130 kN, V = 600 kN

FIGURE 3

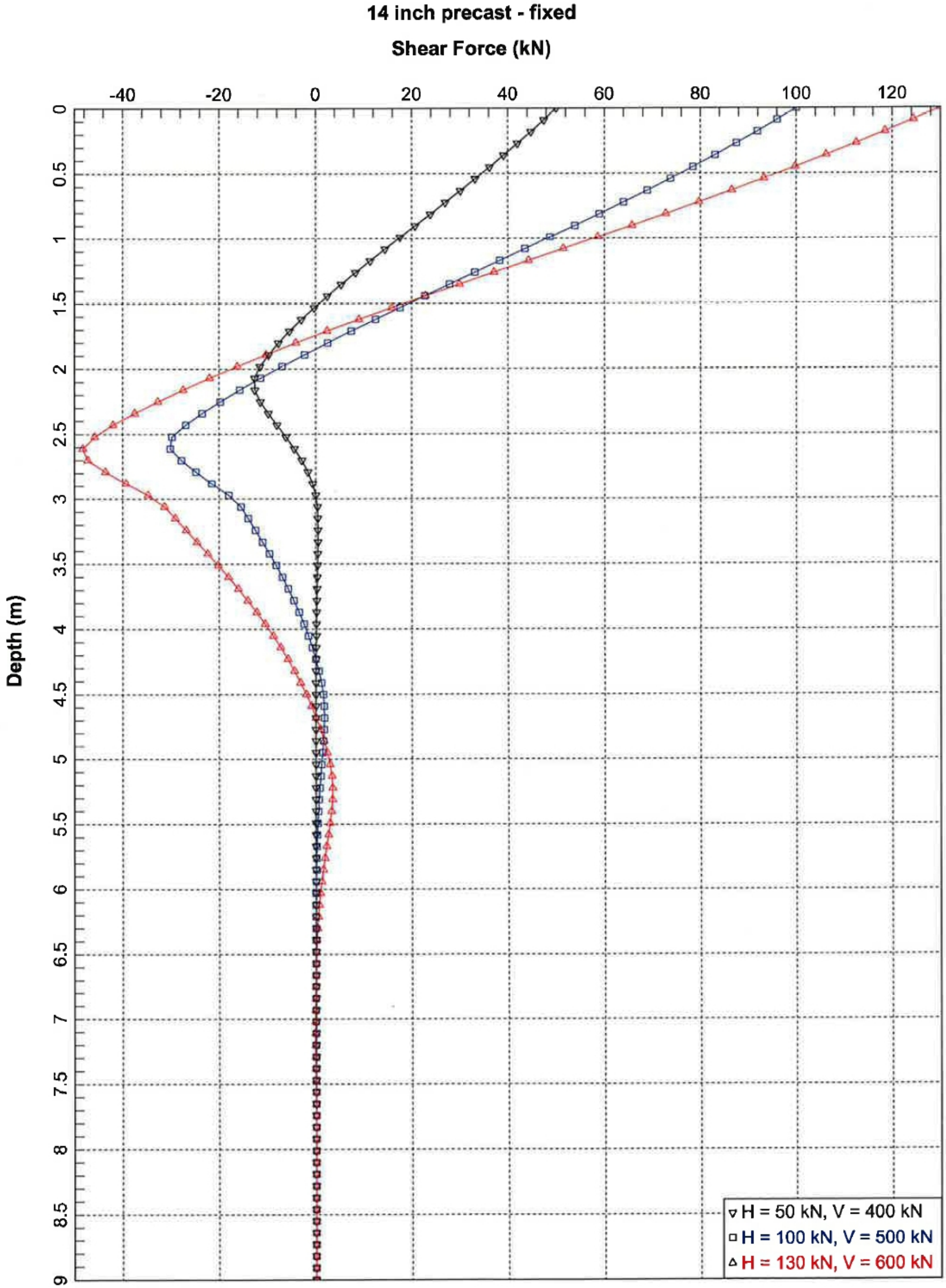

14 inch precast - fixed
Shear Force (kN)
-40
-20
0
20
40
60
80
100
120
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 130 kN, V = 600 kN

FIGURE 4

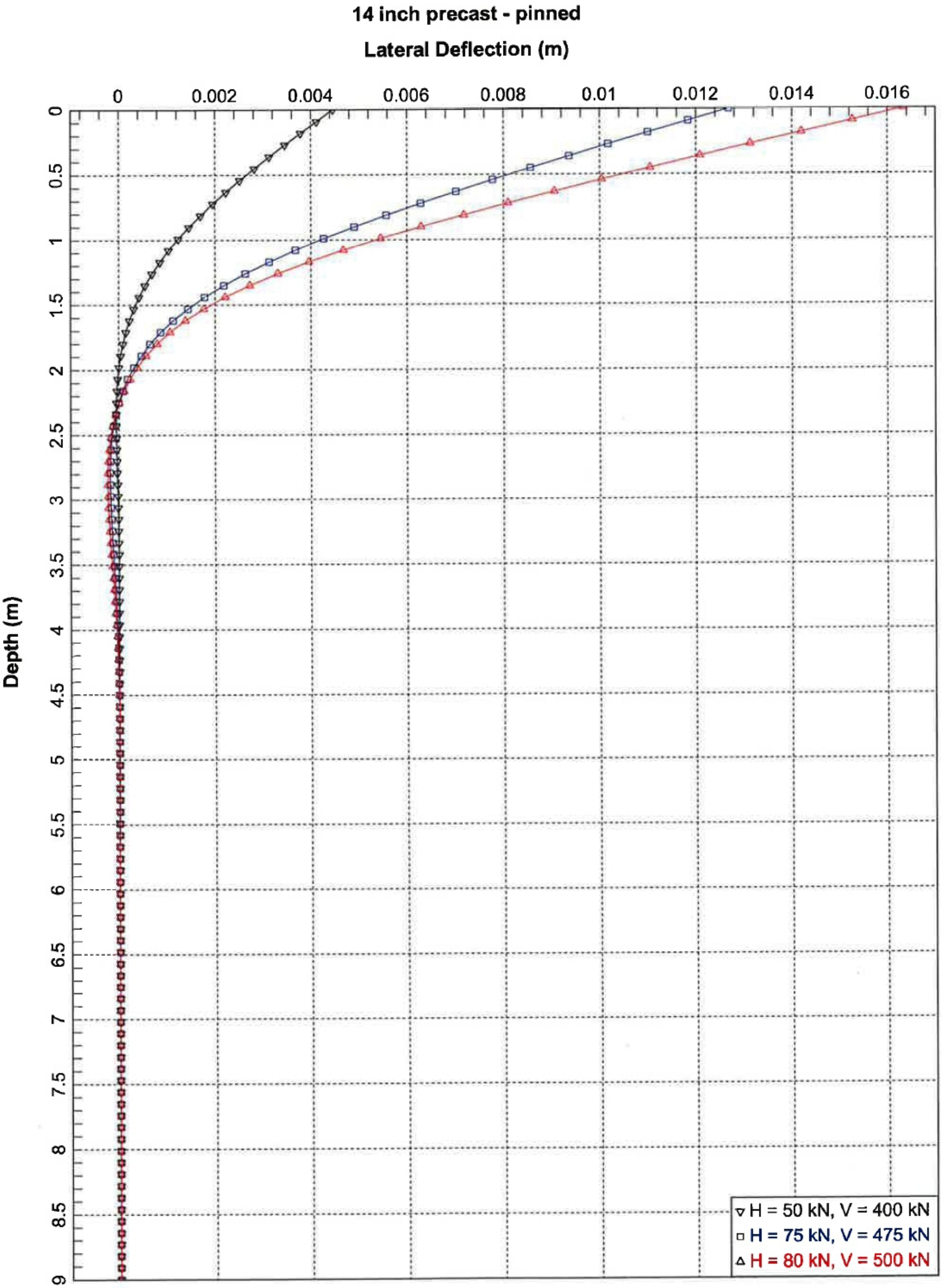

14 inch precast - pinned
Lateral Deflection (m)
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 75 kN, V = 475 kN
△ H = 80 kN, V = 500 kN

FIGURE 5

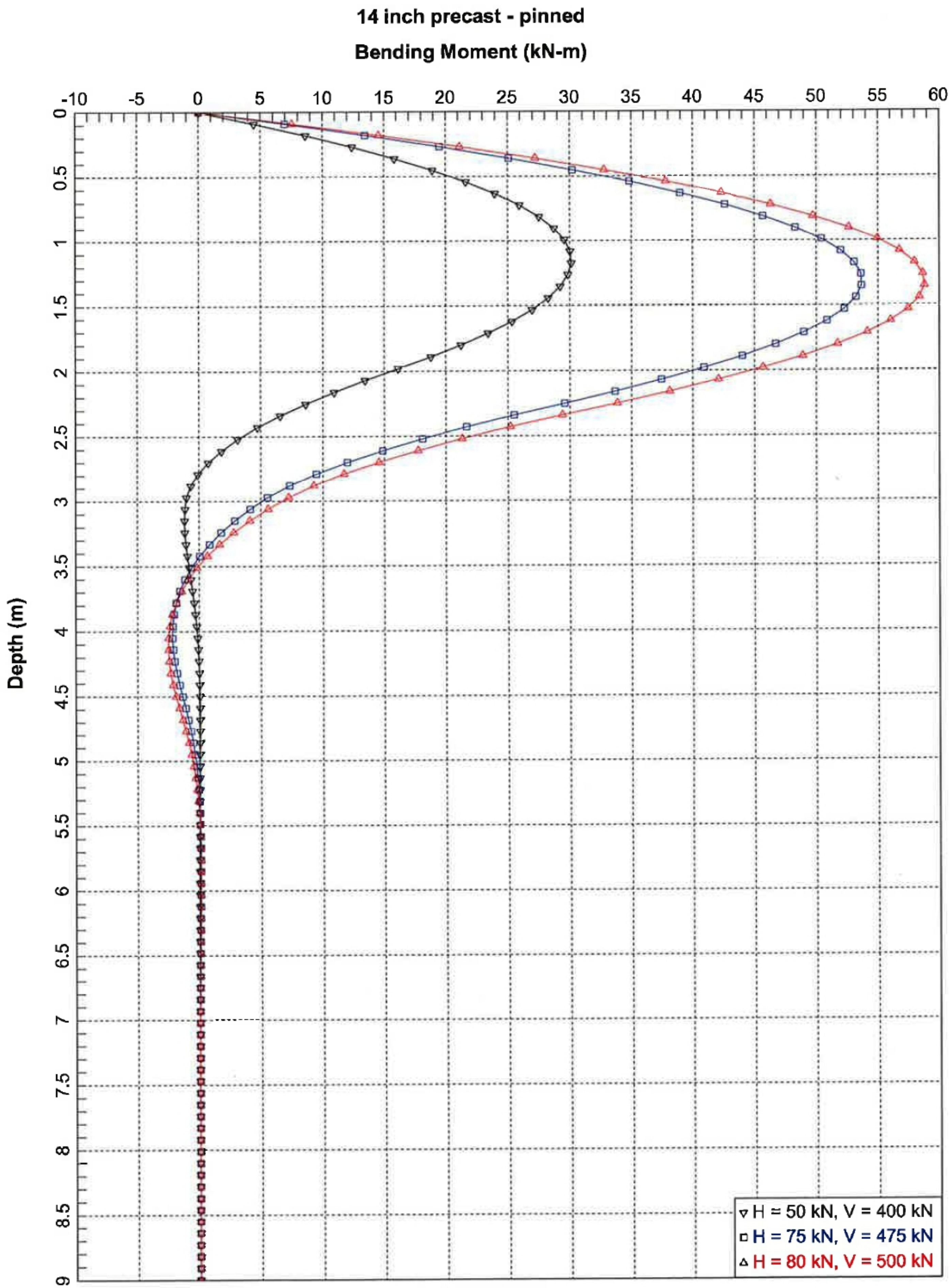

14 inch precast - pinned
Bending Moment (kN-m)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
55
60
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 75 kN, V = 475 kN
△ H = 80 kN, V = 500 kN

FIGURE 6

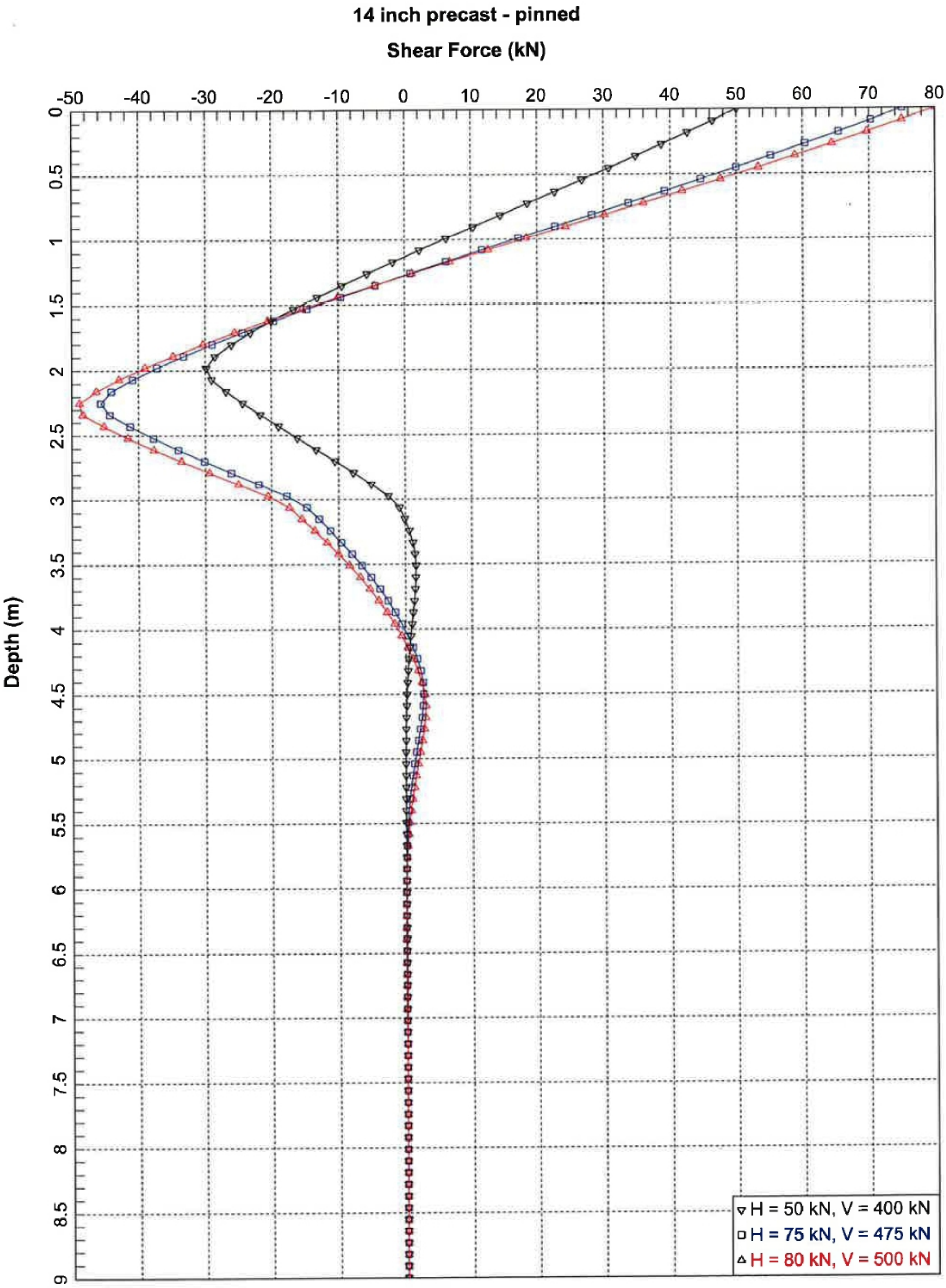

14 inch precast - pinned
Shear Force (kN)
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 75 kN, V = 475 kN
△ H = 80 kN, V = 500 kN

FIGURE 7

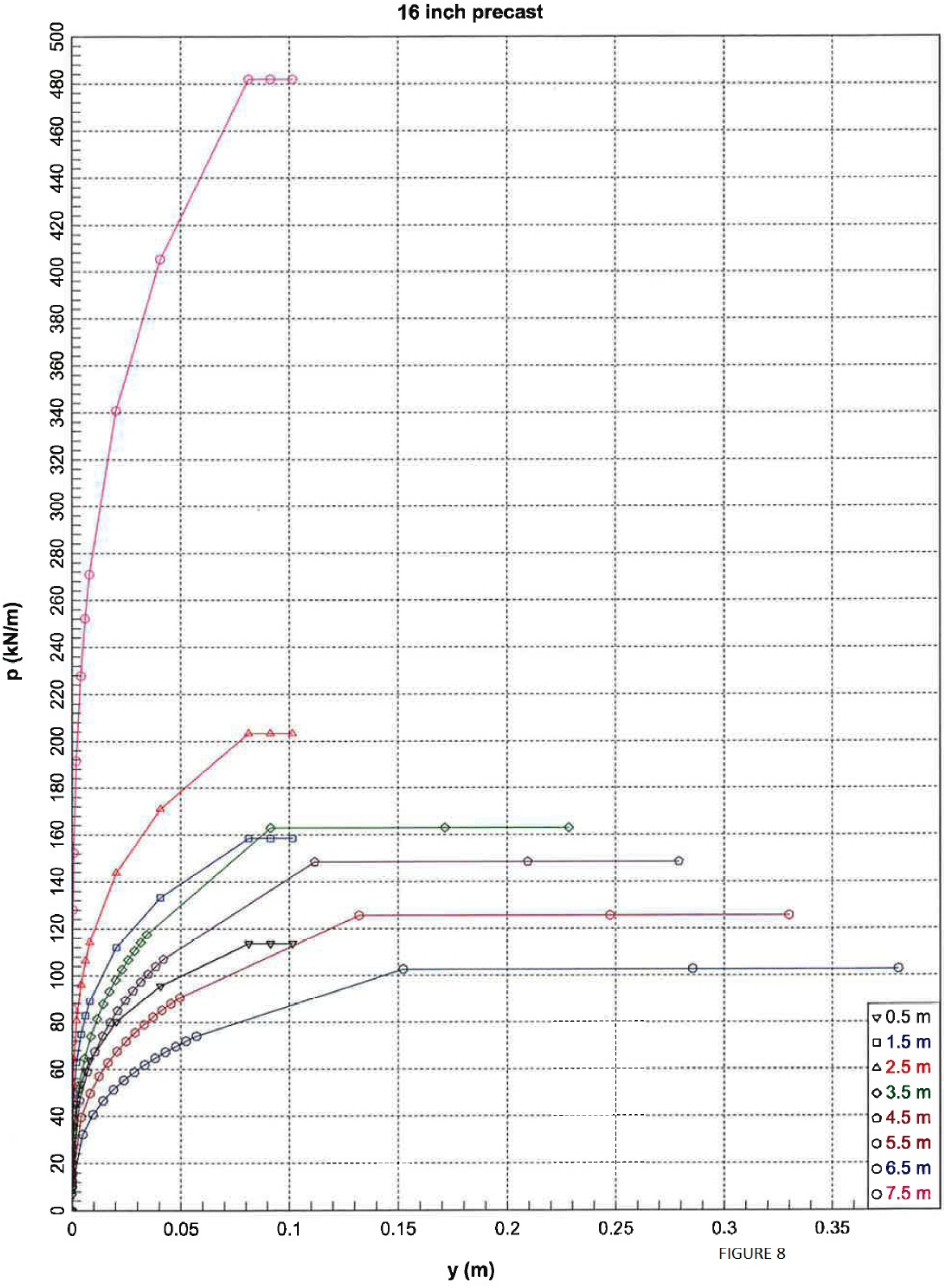

16 inch precast
p (kN/m)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
y (m)
▽ 0.5 m
□ 1.5 m
△ 2.5 m
◇ 3.5 m
○ 4.5 m
○ 5.5 m
○ 6.5 m
○ 7.5 m

FIGURE 8

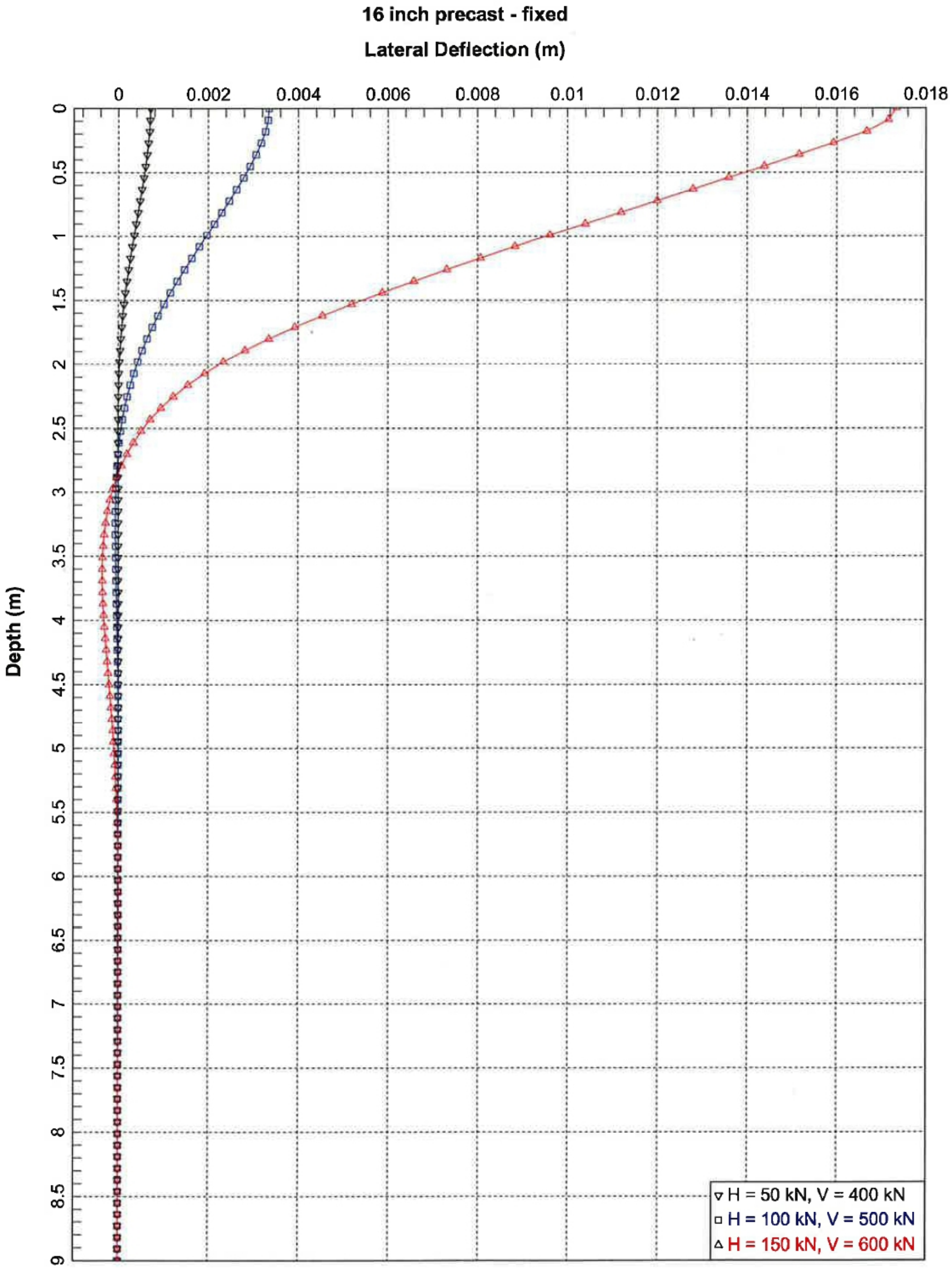

16 inch precast - fixed
Lateral Deflection (m)
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 9

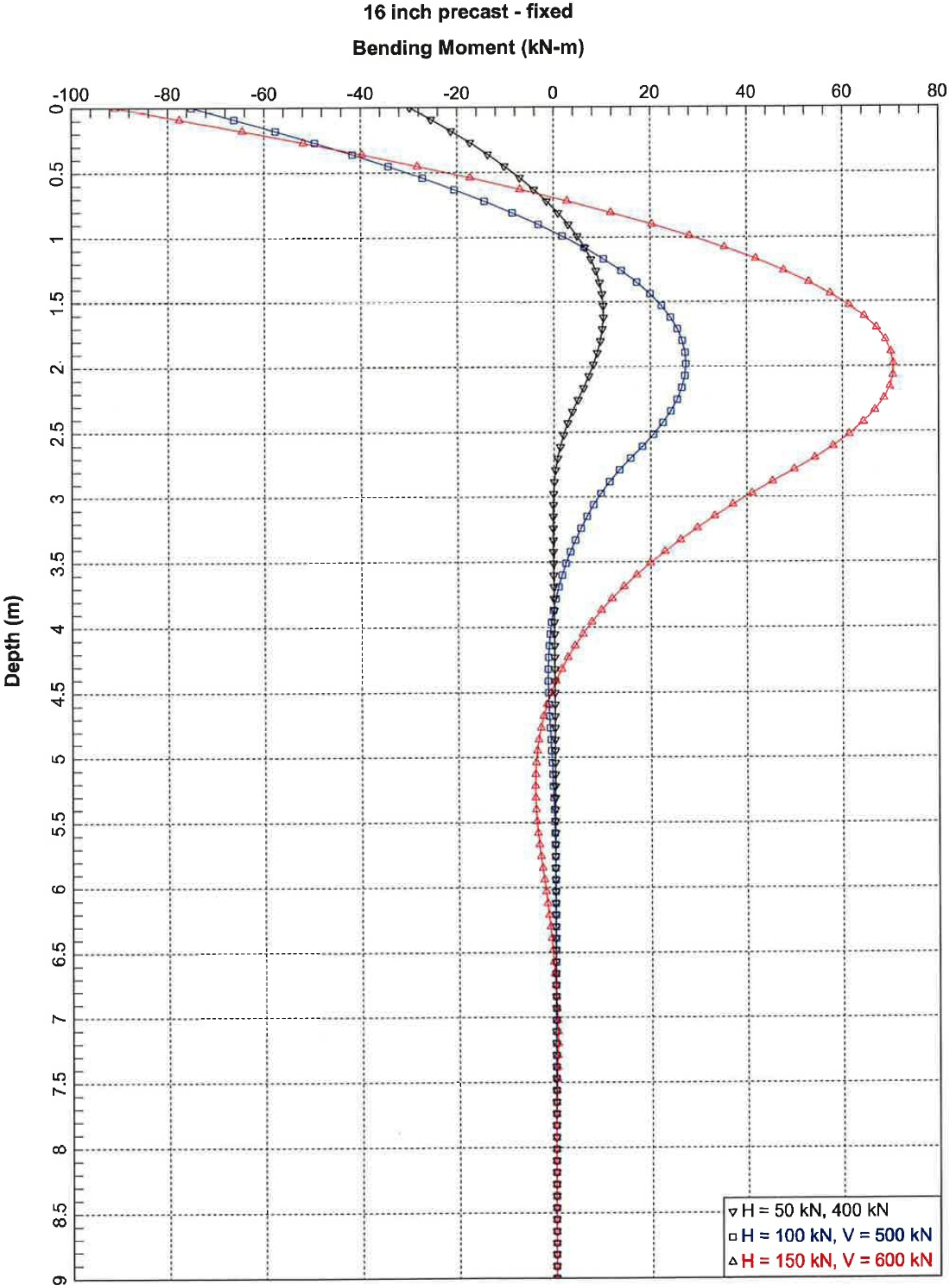

16 inch precast - fixed
Bending Moment (kN-m)
-100
-80
-60
-40
-20
0
20
40
60
80
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▿ H = 50 kN, 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 10

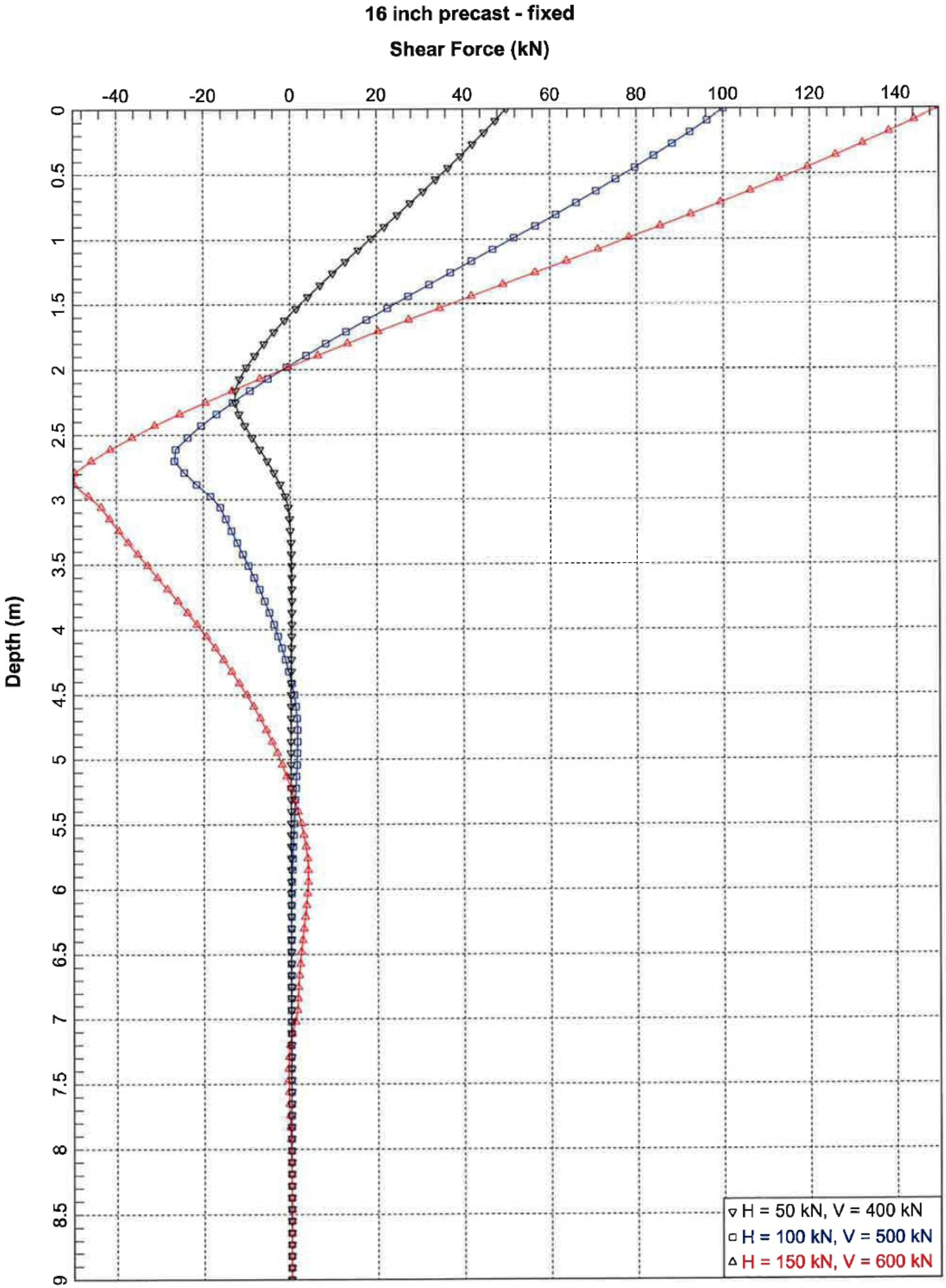
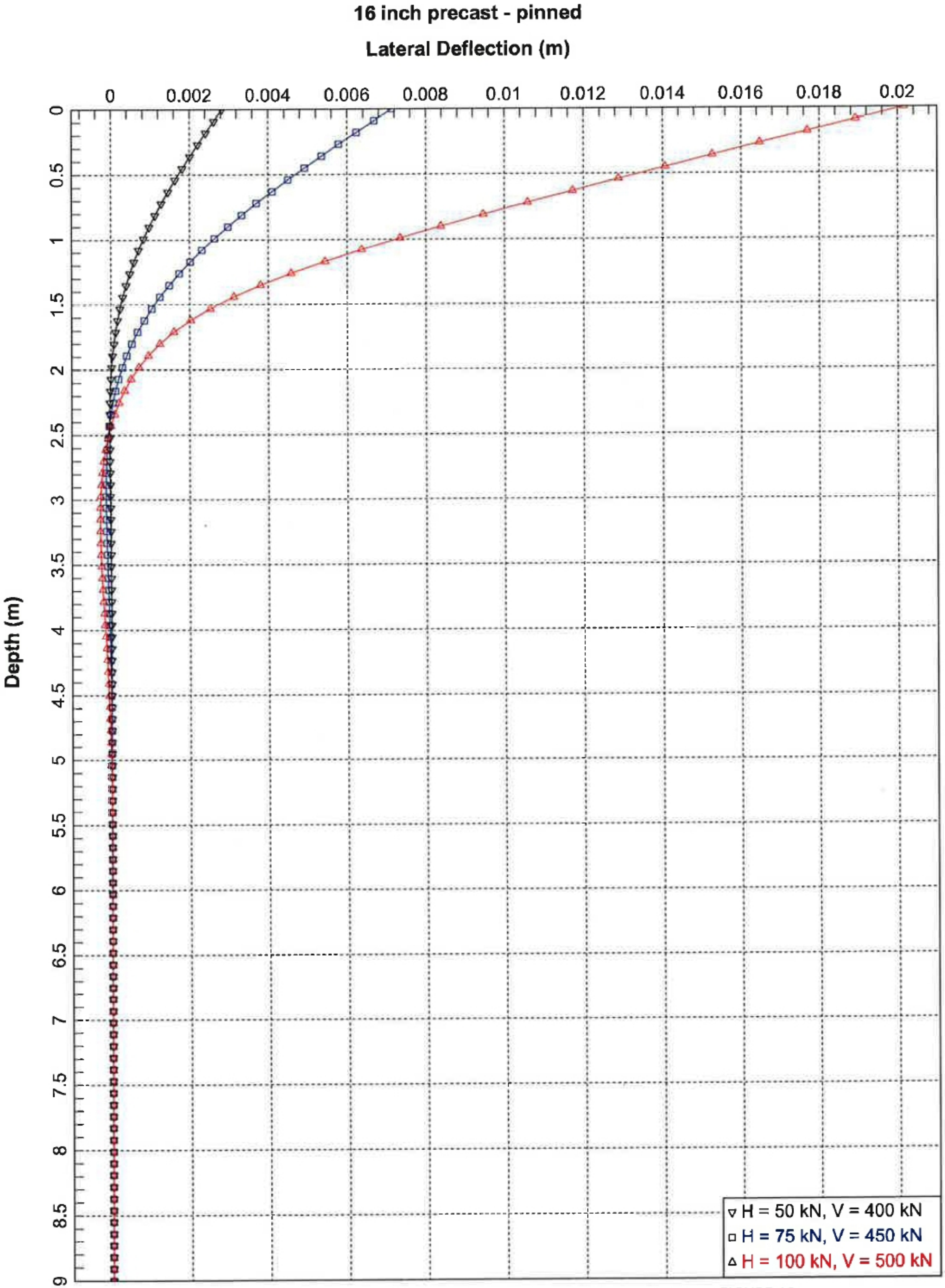

16 inch precast - fixed
Shear Force (kN)
-40
-20
0
20
40
60
80
100
120
140
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 11



16 inch precast - pinned

FIGURE 12

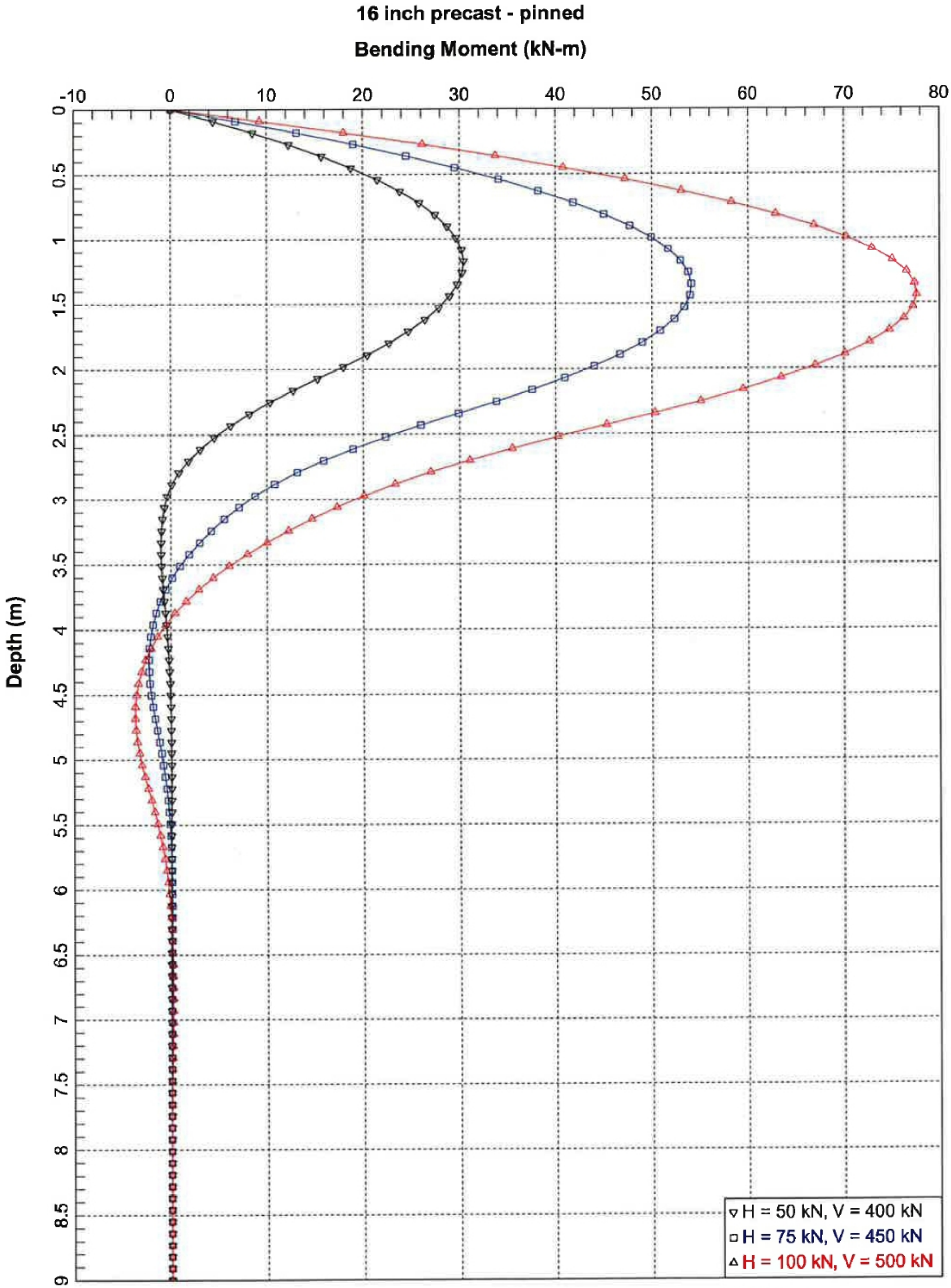

16 inch precast - pinned
Bending Moment (kN-m)
-10
0
10
20
30
40
50
60
70
80
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 75 kN, V = 450 kN
△ H = 100 kN, V = 500 kN

FIGURE 13

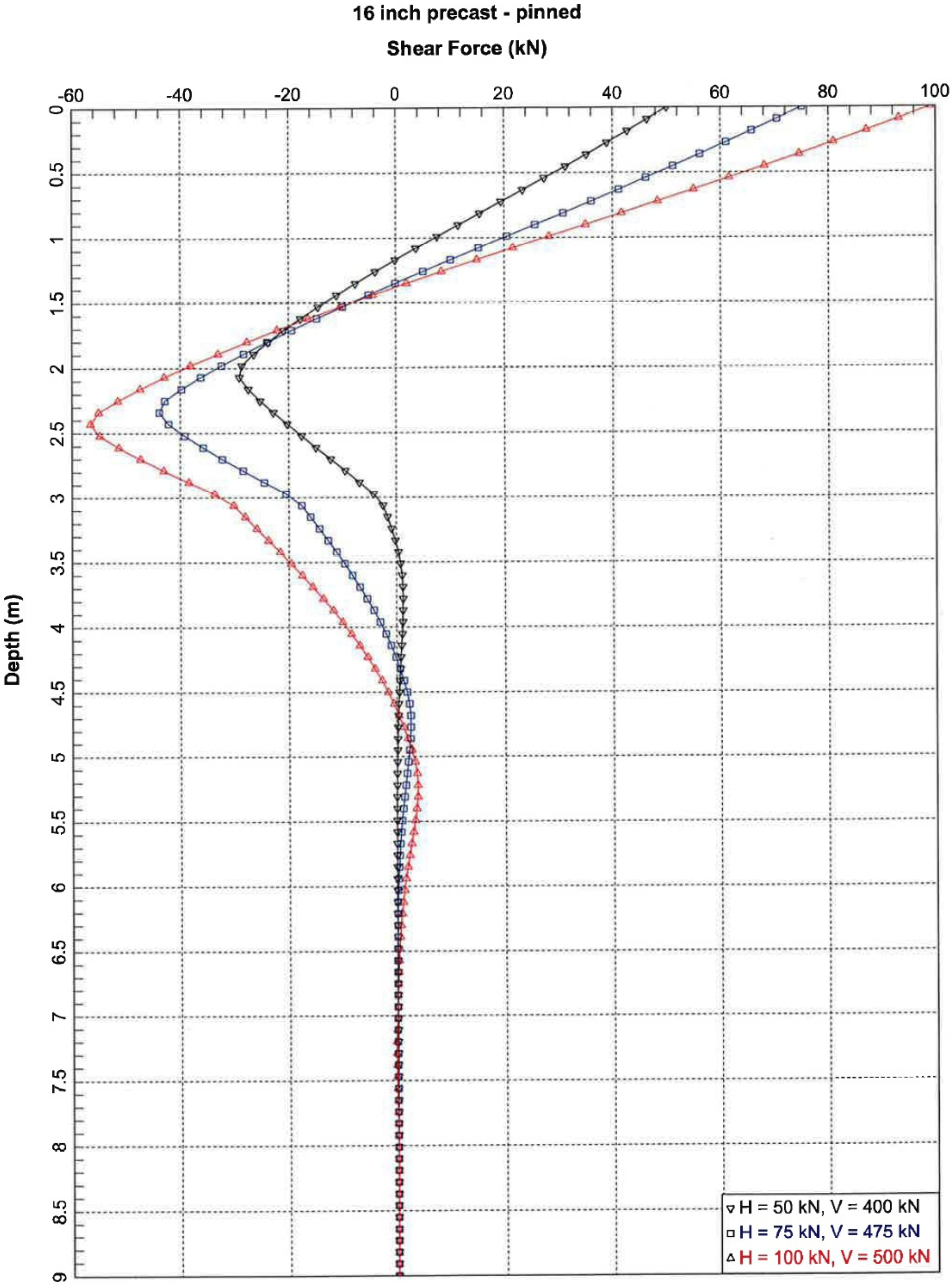

16 inch precast - pinned
Shear Force (kN)
-60
-40
-20
0
20
40
60
80
100
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▿ H = 50 kN, V = 400 kN
□ H = 75 kN, V = 475 kN
▵ H = 100 kN, V = 500 kN

FIGURE 14

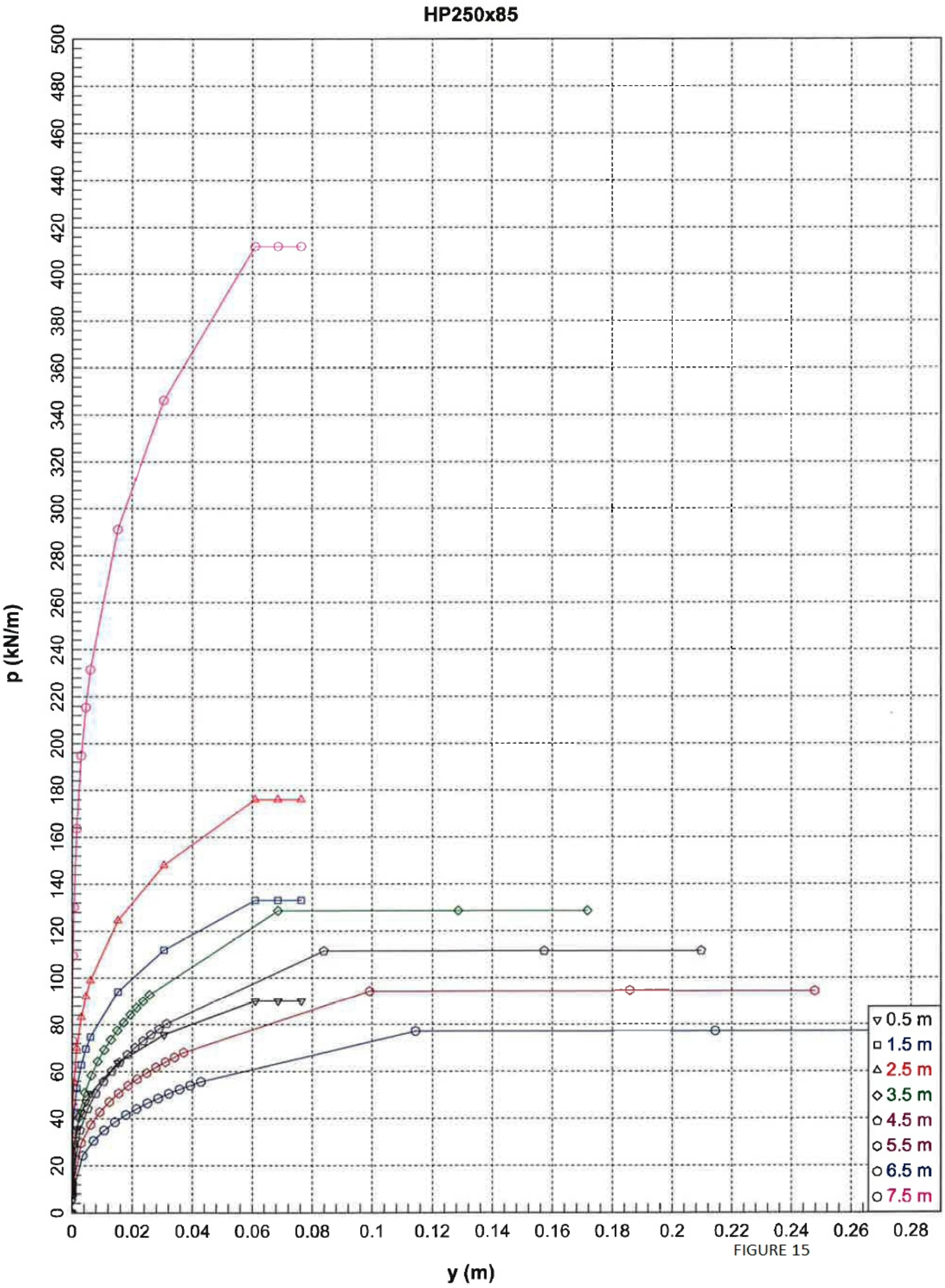

HP250x85
p (kN/m)
y (m)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14
0.16
0.18
0.2
0.22
0.24
0.26
0.28
▽ 0.5 m
□ 1.5 m
△ 2.5 m
◇ 3.5 m
⬠ 4.5 m
○ 5.5 m
○ 6.5 m
○ 7.5 m

FIGURE 15

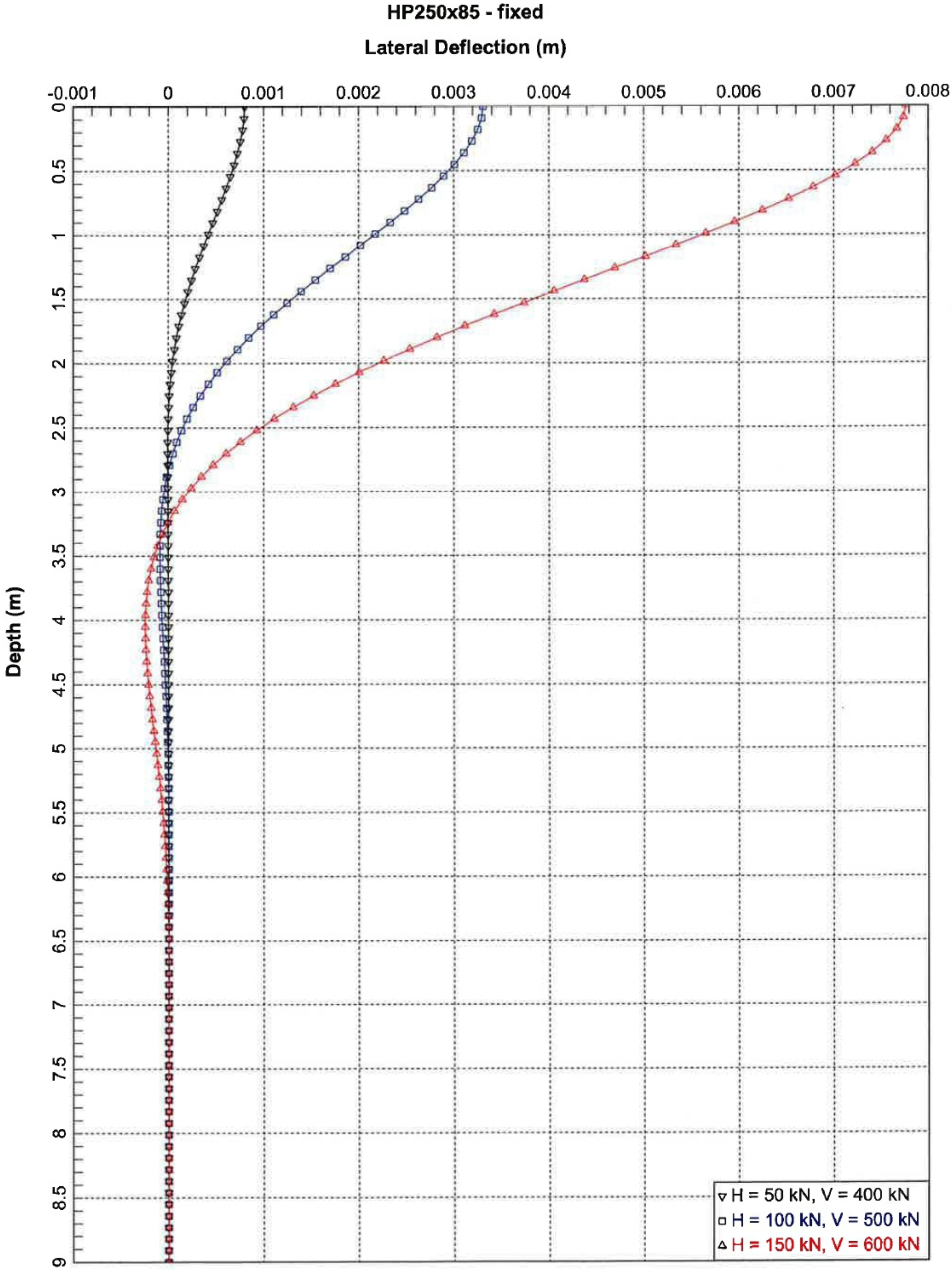

HP250x85 - fixed
Lateral Deflection (m)
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
0.008
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 16

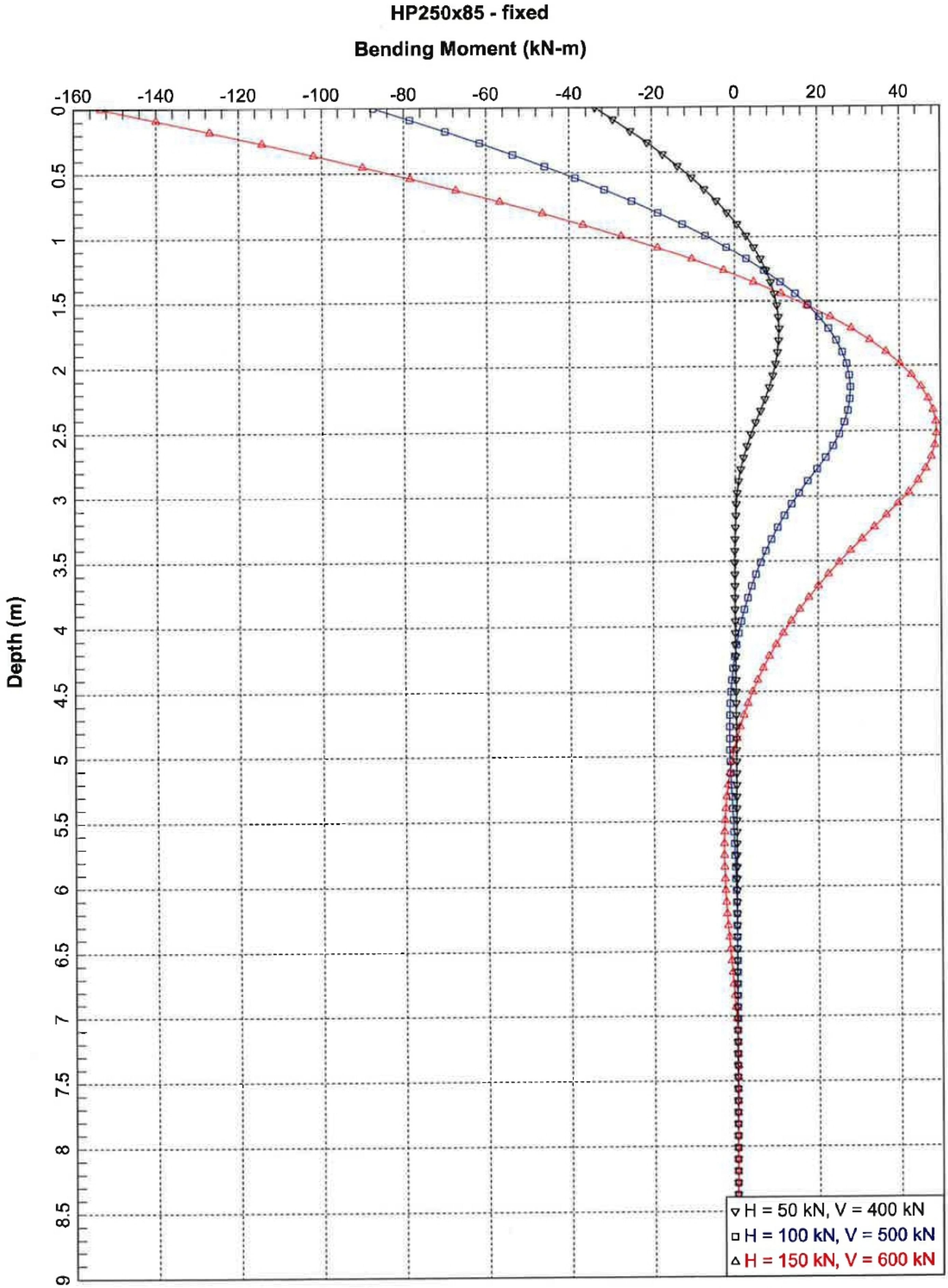

HP250x85 - fixed
Bending Moment (kN-m)
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 17

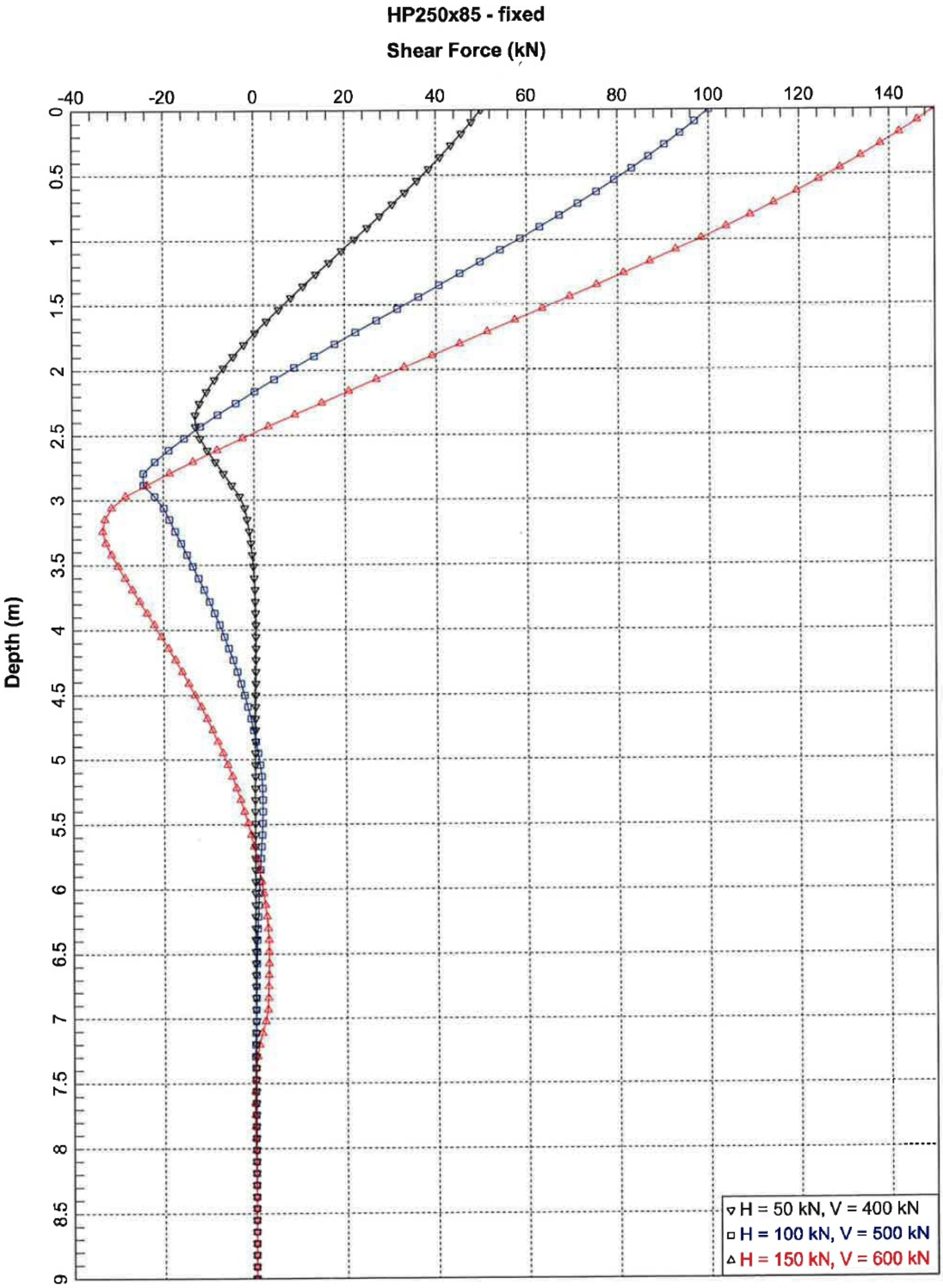

HP250x85 - fixed
Shear Force (kN)
-40
-20
0
20
40
60
80
100
120
140
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 18

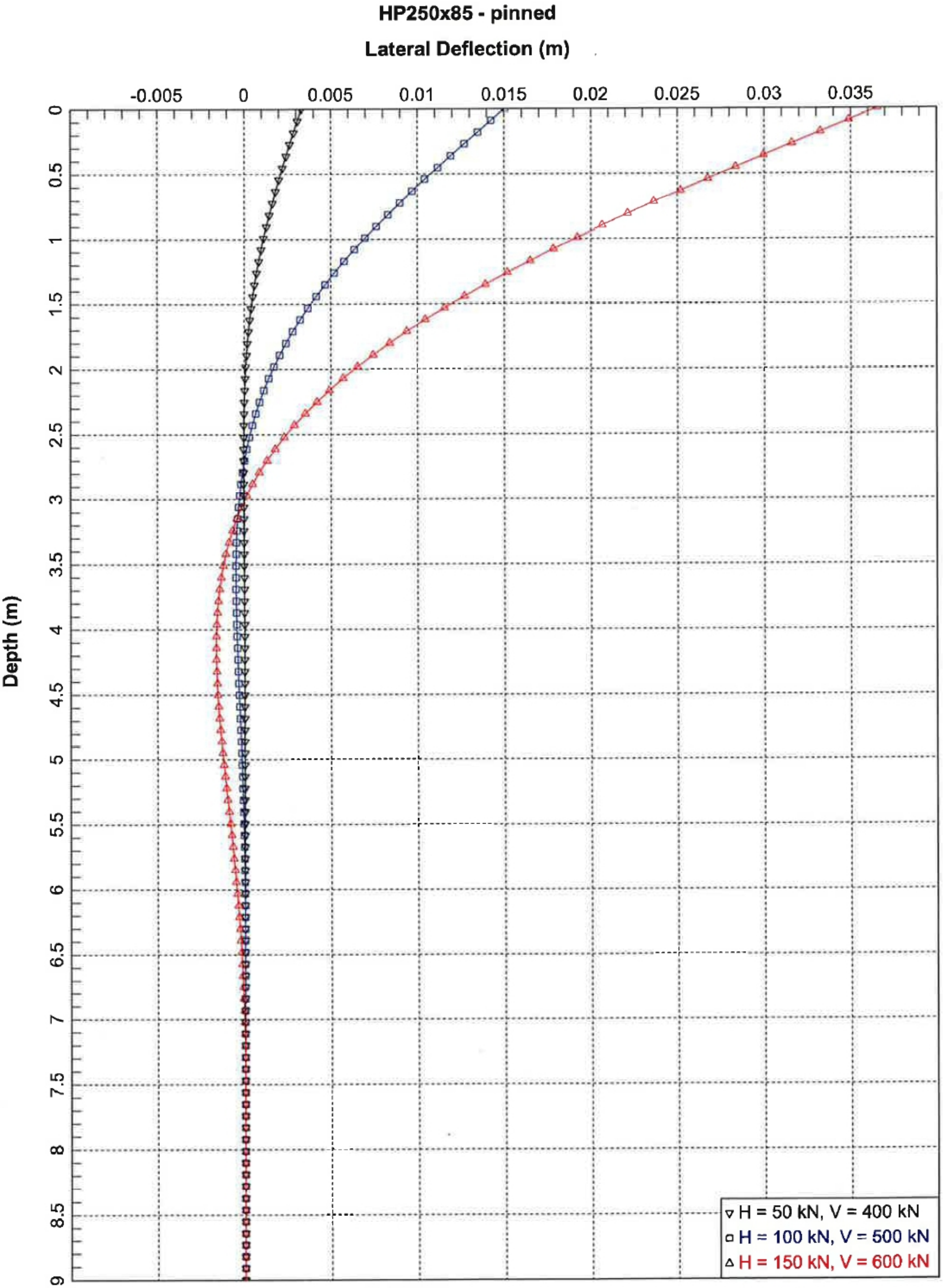

HP250x85 - pinned
Lateral Deflection (m)
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 19

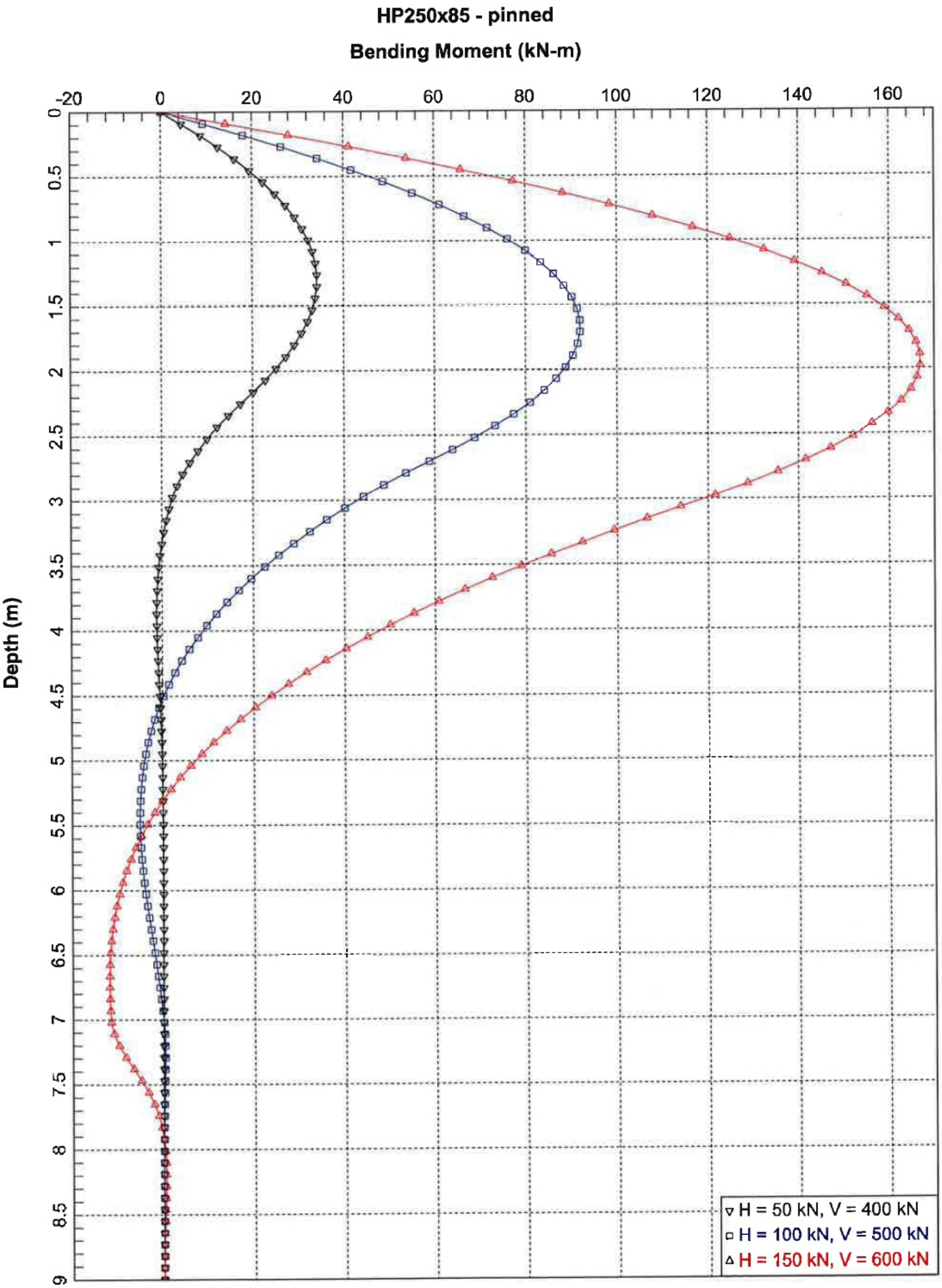

HP250x85 - pinned
Bending Moment (kN-m)
-20
0
20
40
60
80
100
120
140
160
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 20

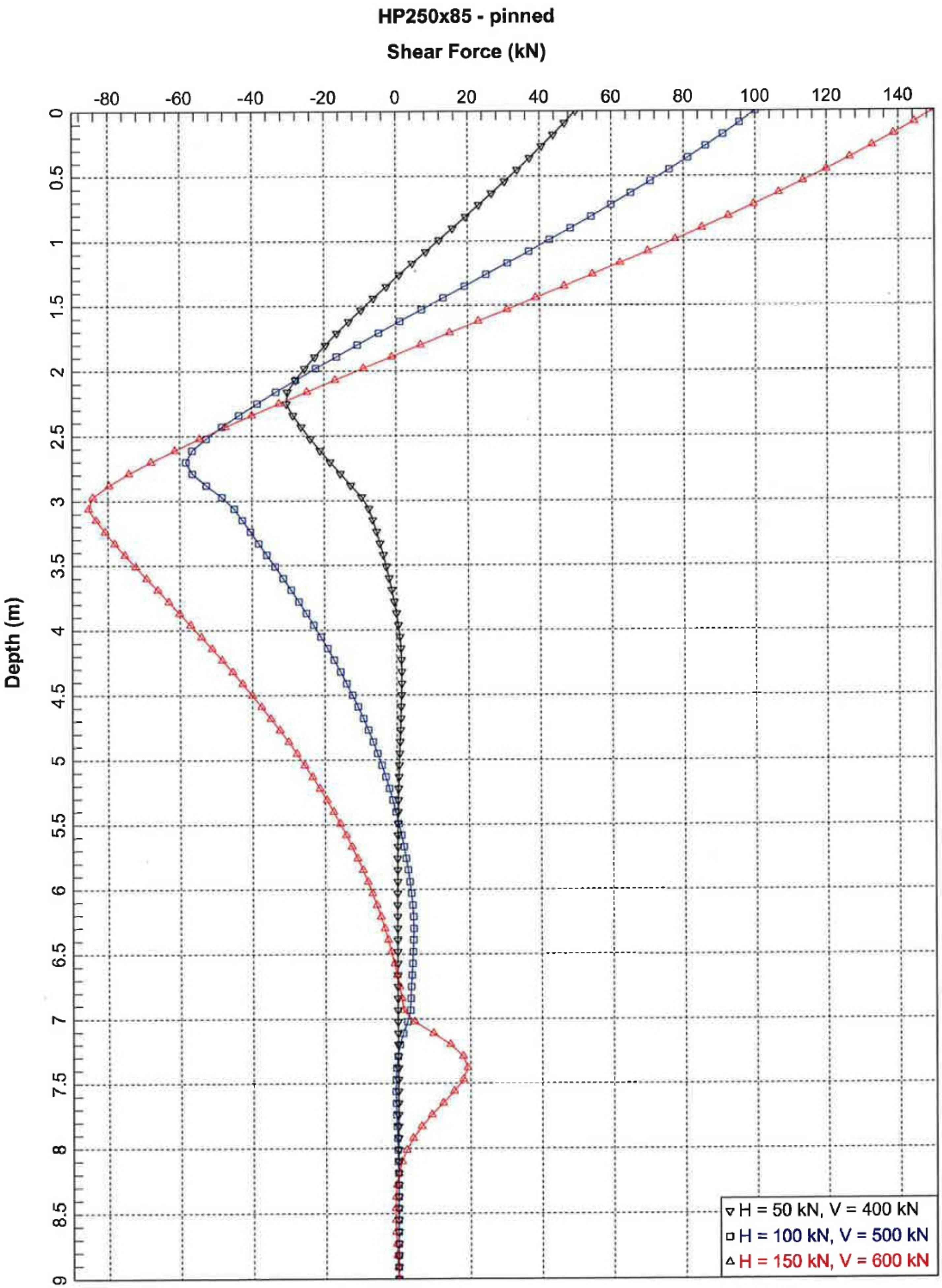

HP250x85 - pinned
Shear Force (kN)
-80
-60
-40
-20
0
20
40
60
80
100
120
140
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 21

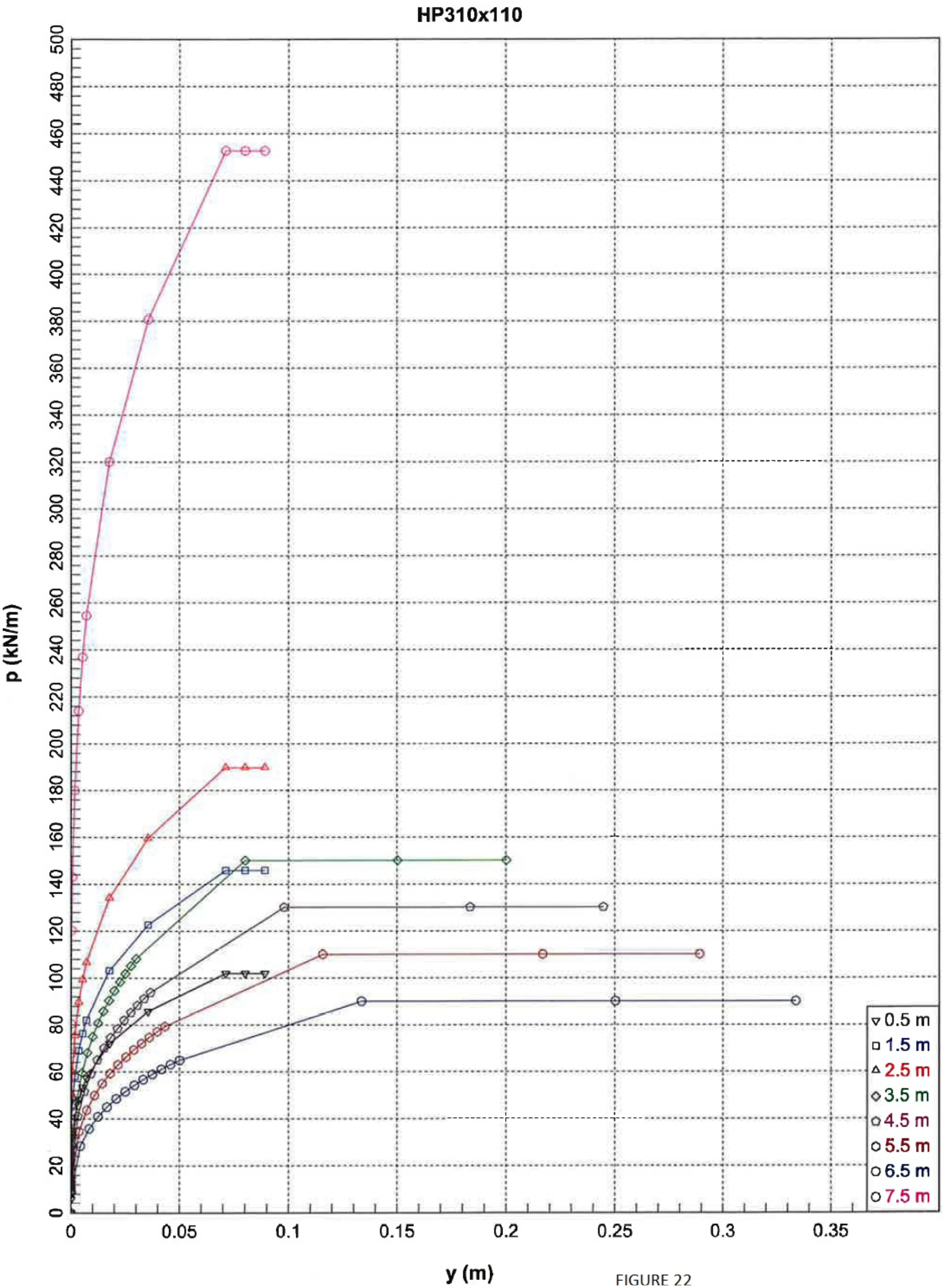
HP310x110
p (kN/m)
y (m)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
▽ 0.5 m
□ 1.5 m
△ 2.5 m
◇ 3.5 m
⬠ 4.5 m
○ 5.5 m
○ 6.5 m
○ 7.5 m

FIGURE 22

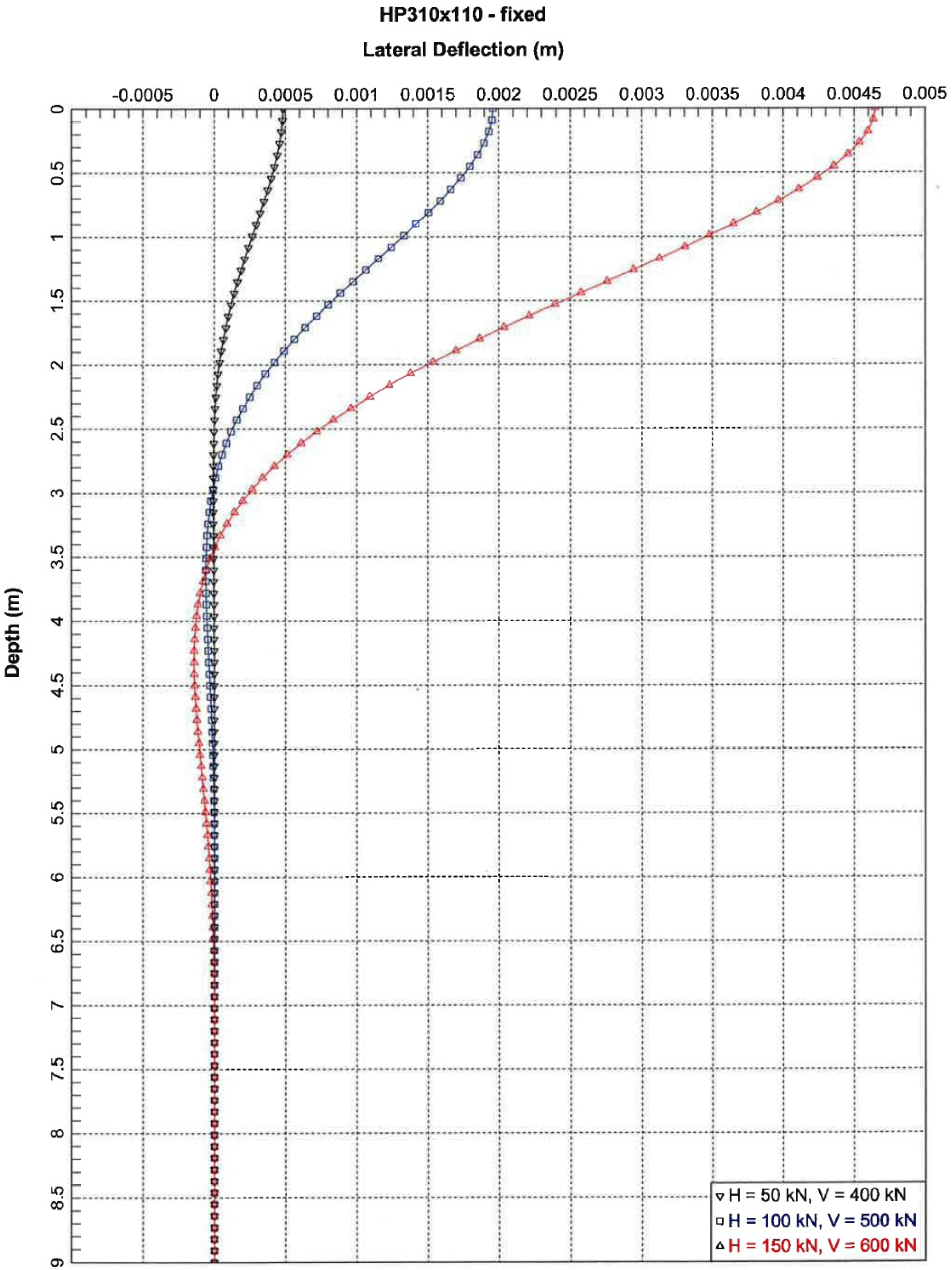

HP310x110 - fixed
Lateral Deflection (m)
-0.0005
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
0.0035
0.004
0.0045
0.005
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 23

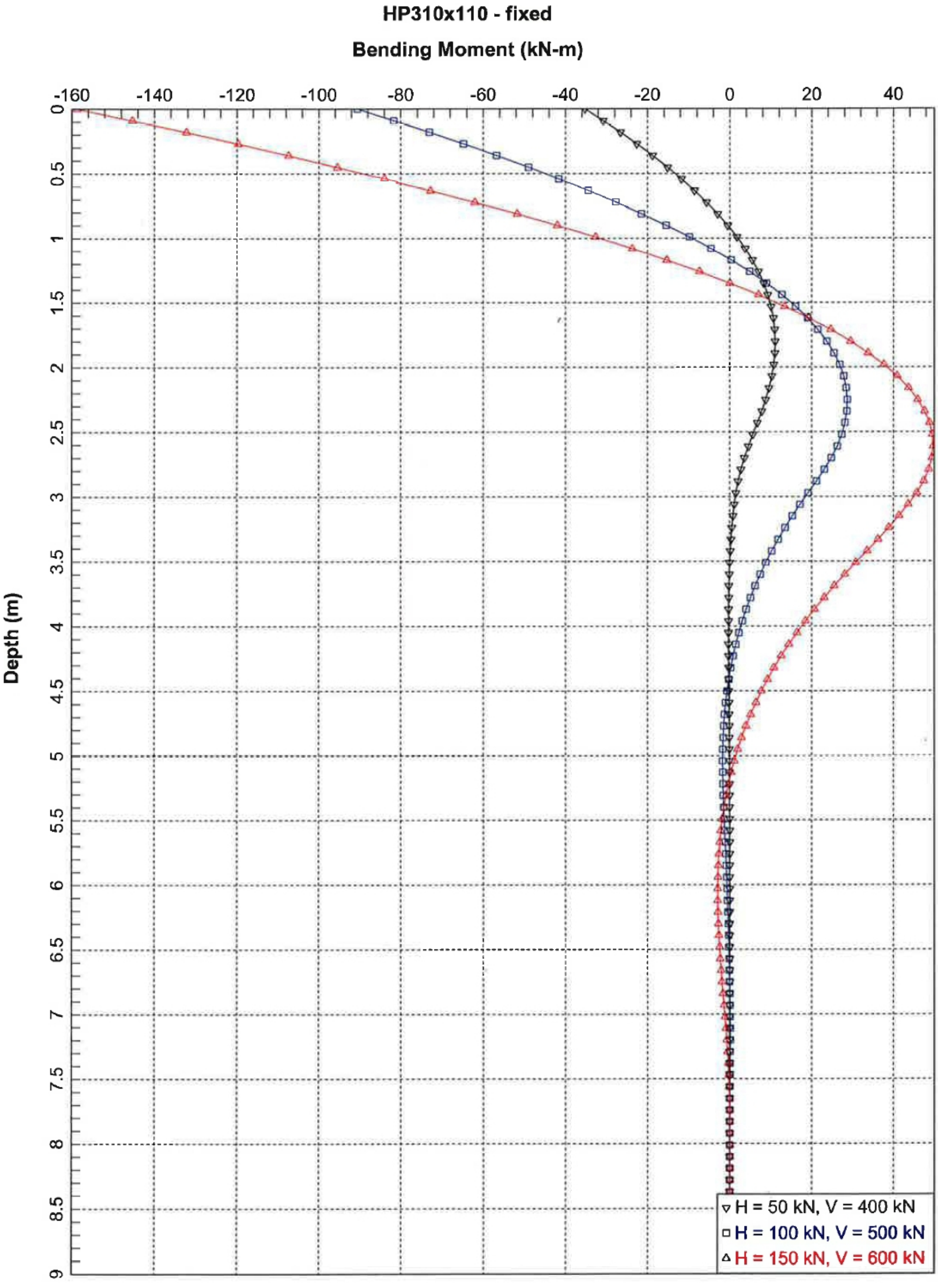

HP310x110 - fixed
Bending Moment (kN-m)
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 24

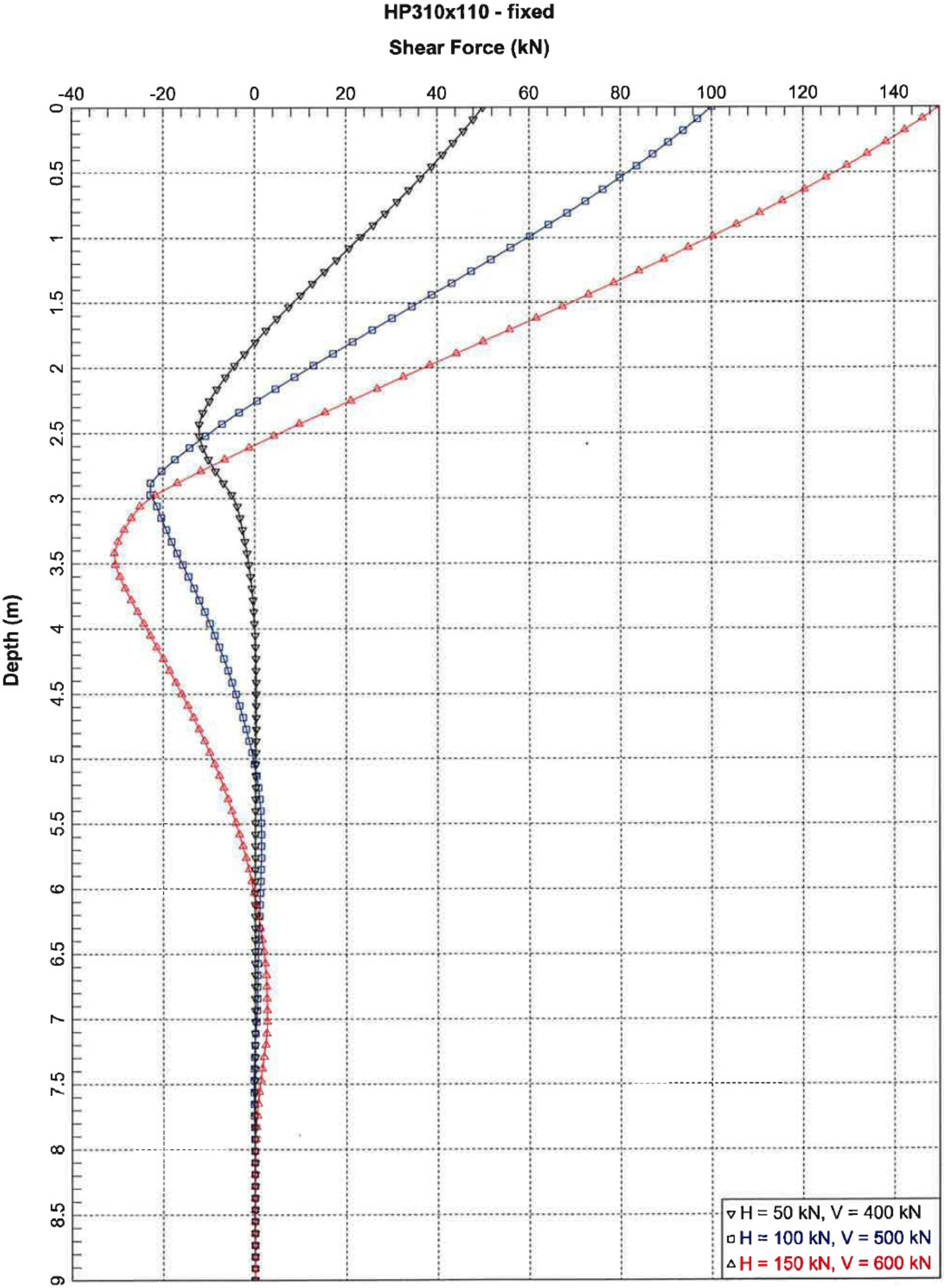

HP310x110 - fixed
Shear Force (kN)
-40
-20
0
20
40
60
80
100
120
140
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 25

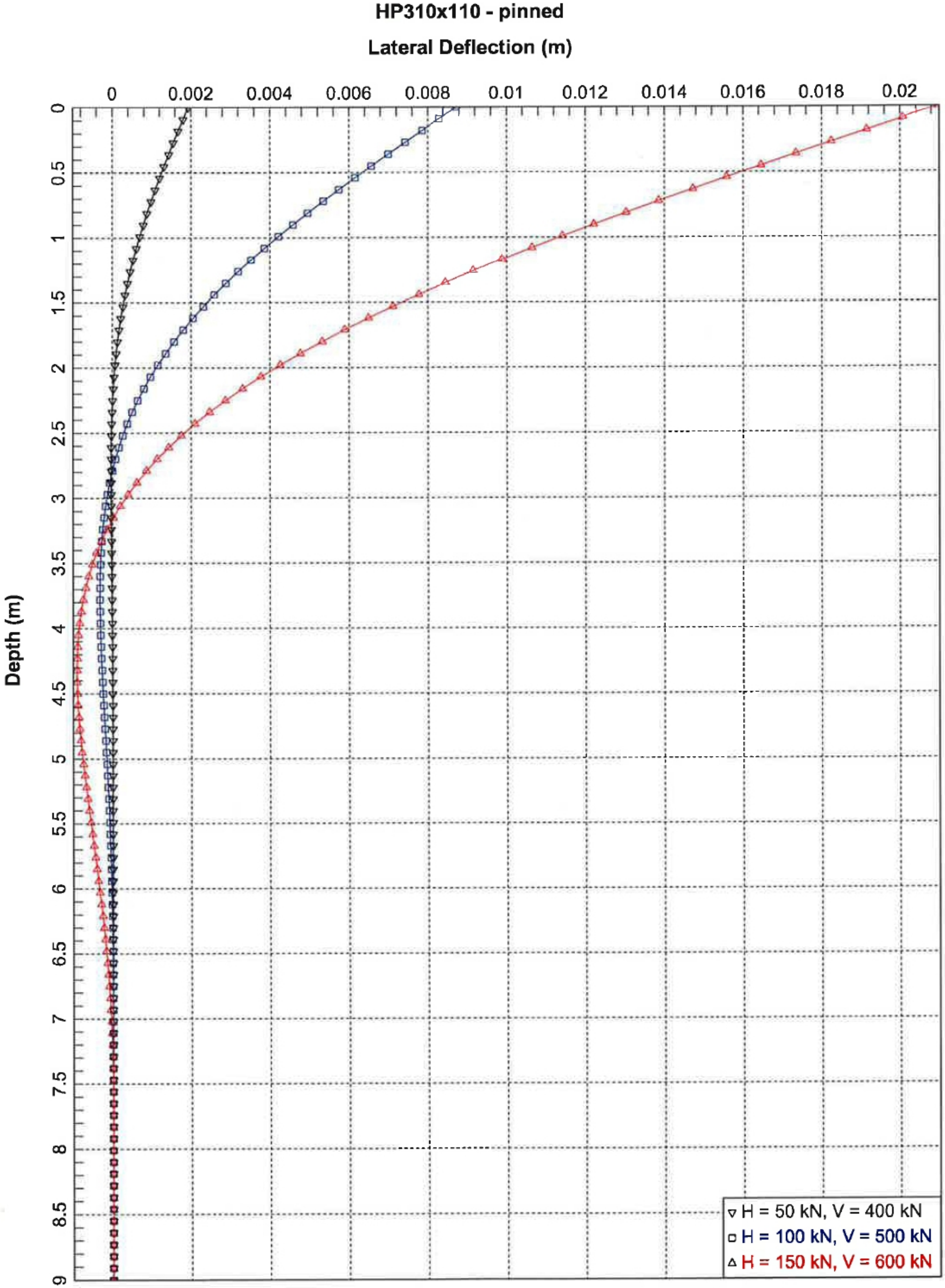

HP310x110 - pinned
Lateral Deflection (m)
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 26

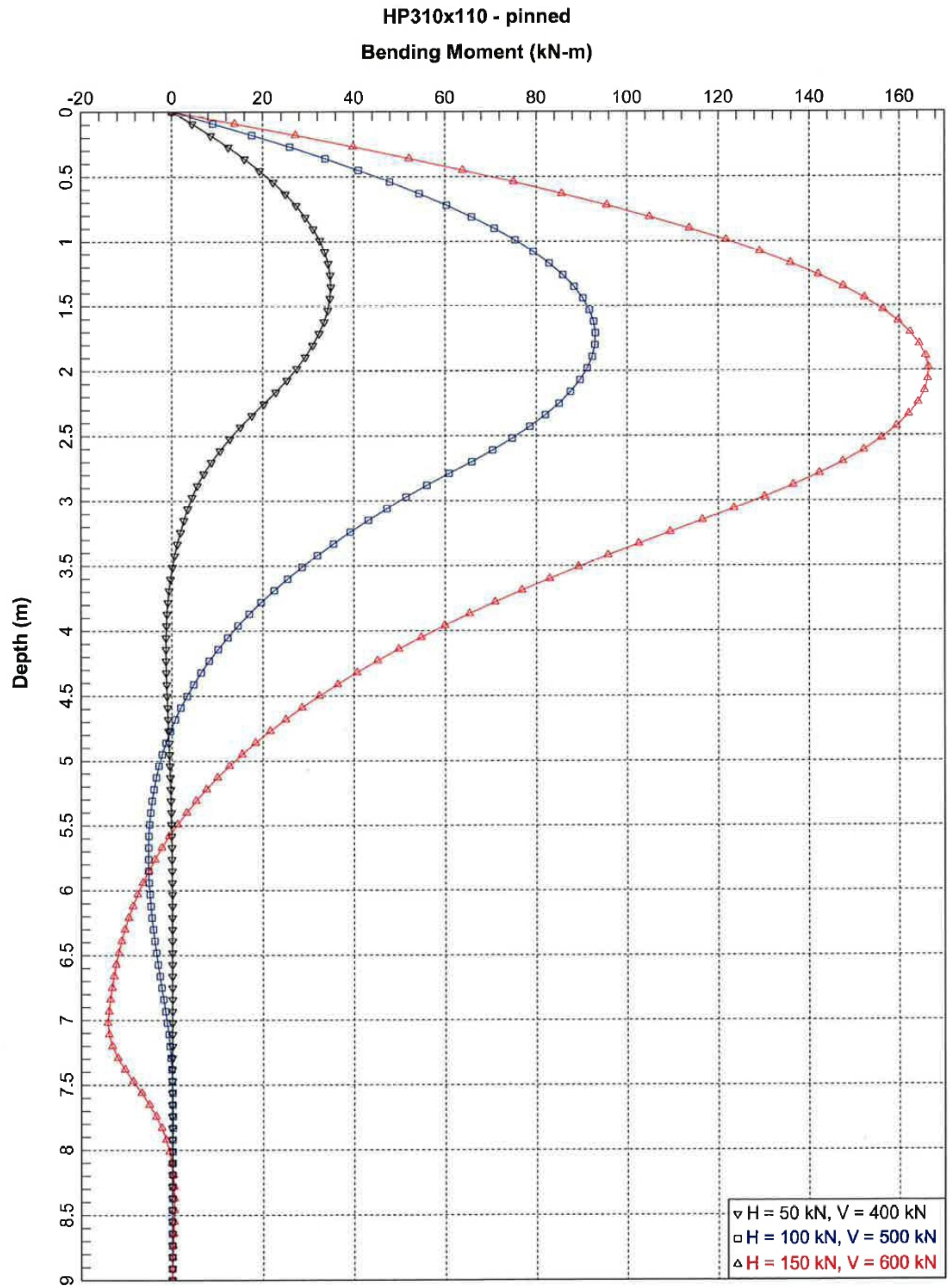

HP310x110 - pinned
Bending Moment (kN-m)
-20
0
20
40
60
80
100
120
140
160
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 27

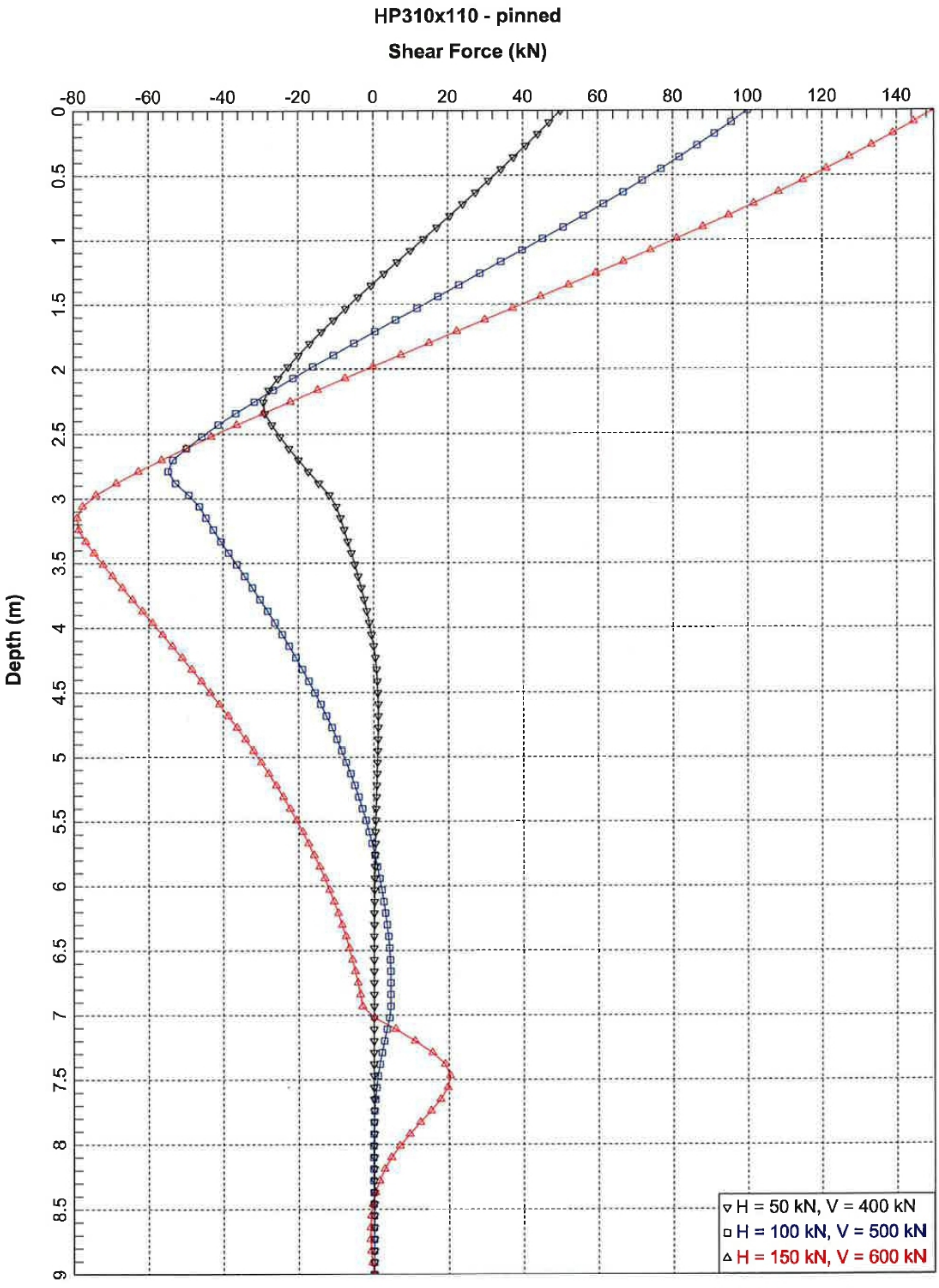
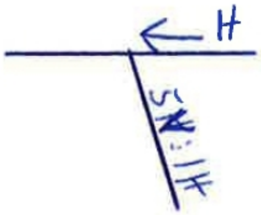

HP310x110 - pinned
Shear Force (kN)
-80
-60
-40
-20
0
20
40
60
80
100
120
140
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 150 kN, V = 600 kN

FIGURE 28


H
5V:1H

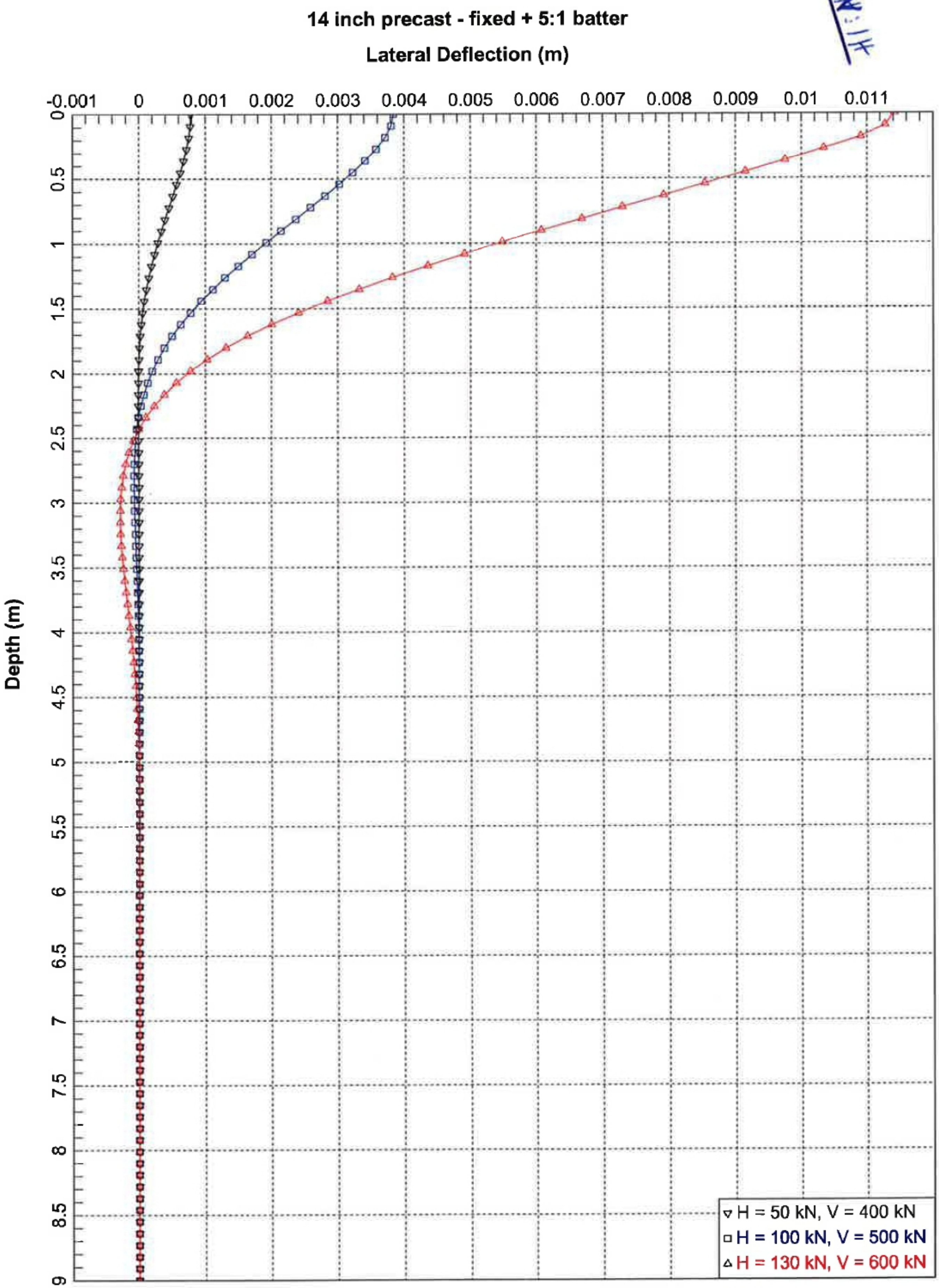

14 inch precast - fixed + 5:1 batter
Lateral Deflection (m)
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
0.008
0.009
0.01
0.011
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 130 kN, V = 600 kN

FIGURE 29

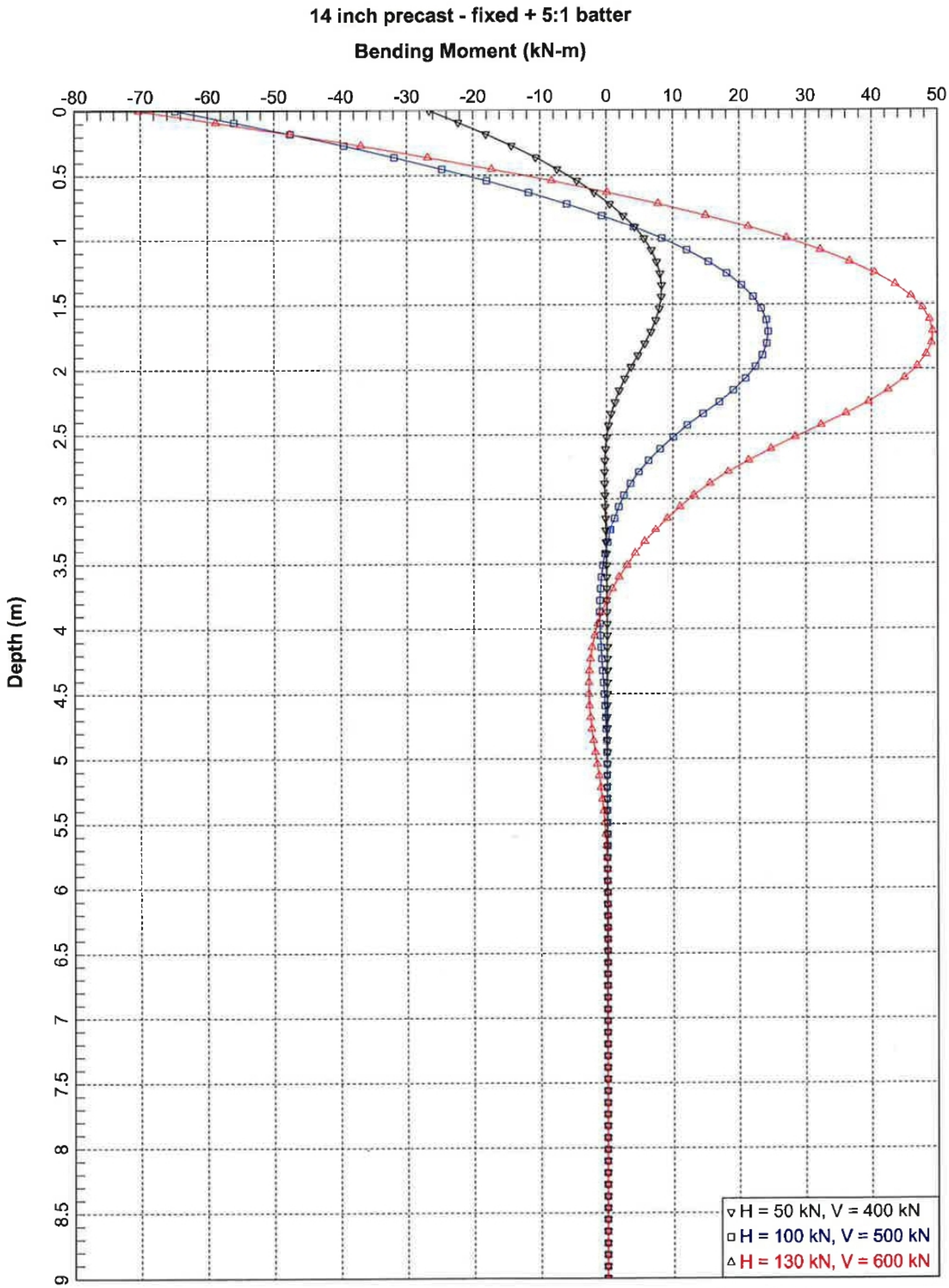
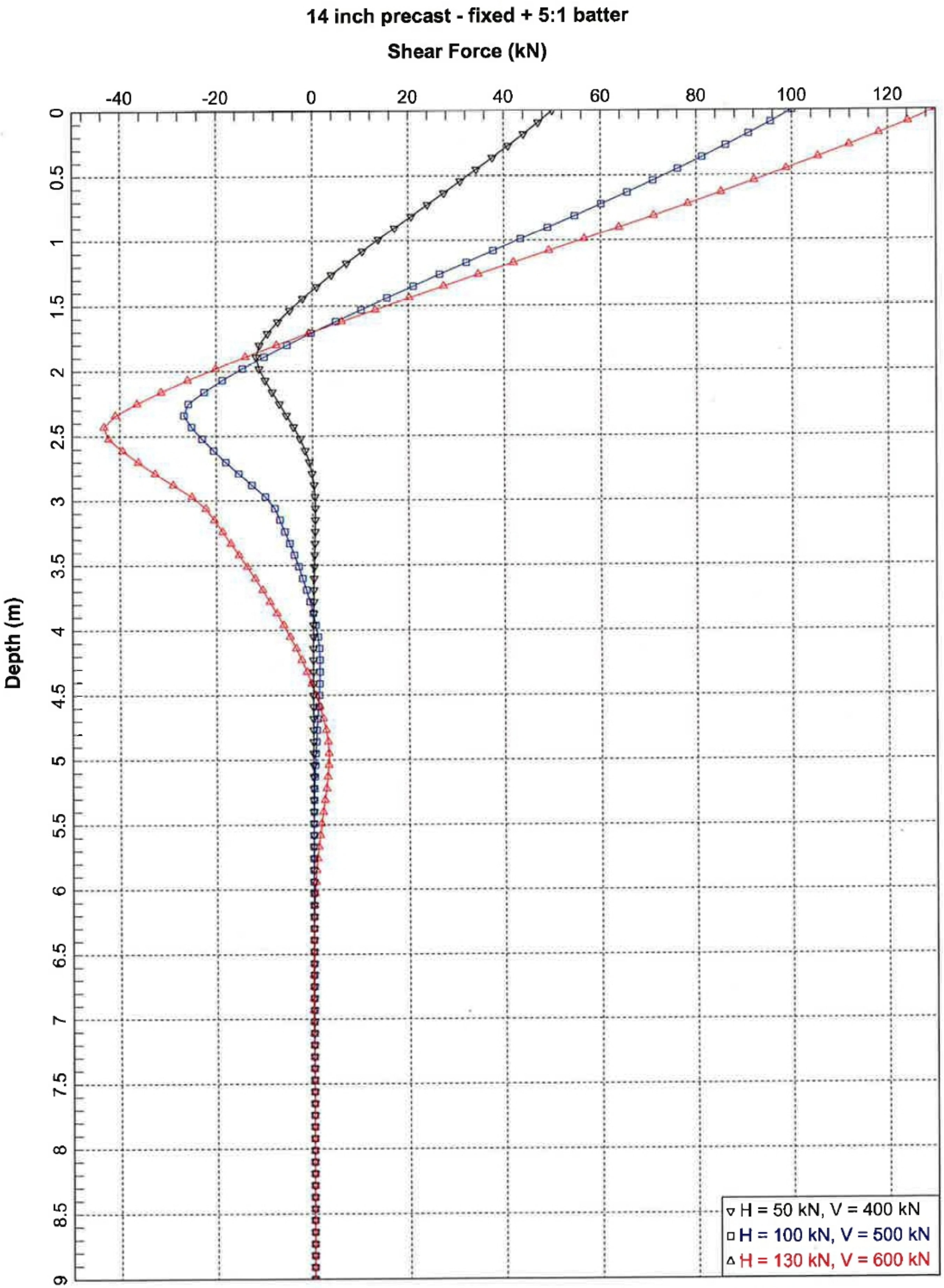

14 inch precast - fixed + 5:1 batter
Bending Moment (kN-m)
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▿ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 130 kN, V = 600 kN

FIGURE 30



14 inch precast - fixed + 5:1 batter
Shear Force (kN)
-40
-20
0
20
40
60
80
100
120
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
▽ H = 50 kN, V = 400 kN
□ H = 100 kN, V = 500 kN
△ H = 130 kN, V = 600 kN

FIGURE 31