Lateral Loads on Micropiles

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Micropile Names

Micropile (DFI & FHWA)
= Pin Pile^{SM} (Nicholson)
= Minipile (previously used by Hayward Baker and used in UK)
= Bored-in Pile (NYSDOT)
= Small Diameter Grouted Piles (Mass. Building Code)
= <12” diameter drilled and grouted

Introduction

Lateral load performance and design of Pin Piles

- results of lateral load tests including load and deflection
- comparison of lateral tests results to predictions using LPILE, NAVFAC, and Characteristic Load Method (CLM)
- combined stresses
- options for increasing lateral resistance
- analysis for battered piles

The intent is to demonstrate that micropiles and micropile groups can be designed to support lateral loads

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Lateral Load Test – Site C

They are “two for the price of one”.

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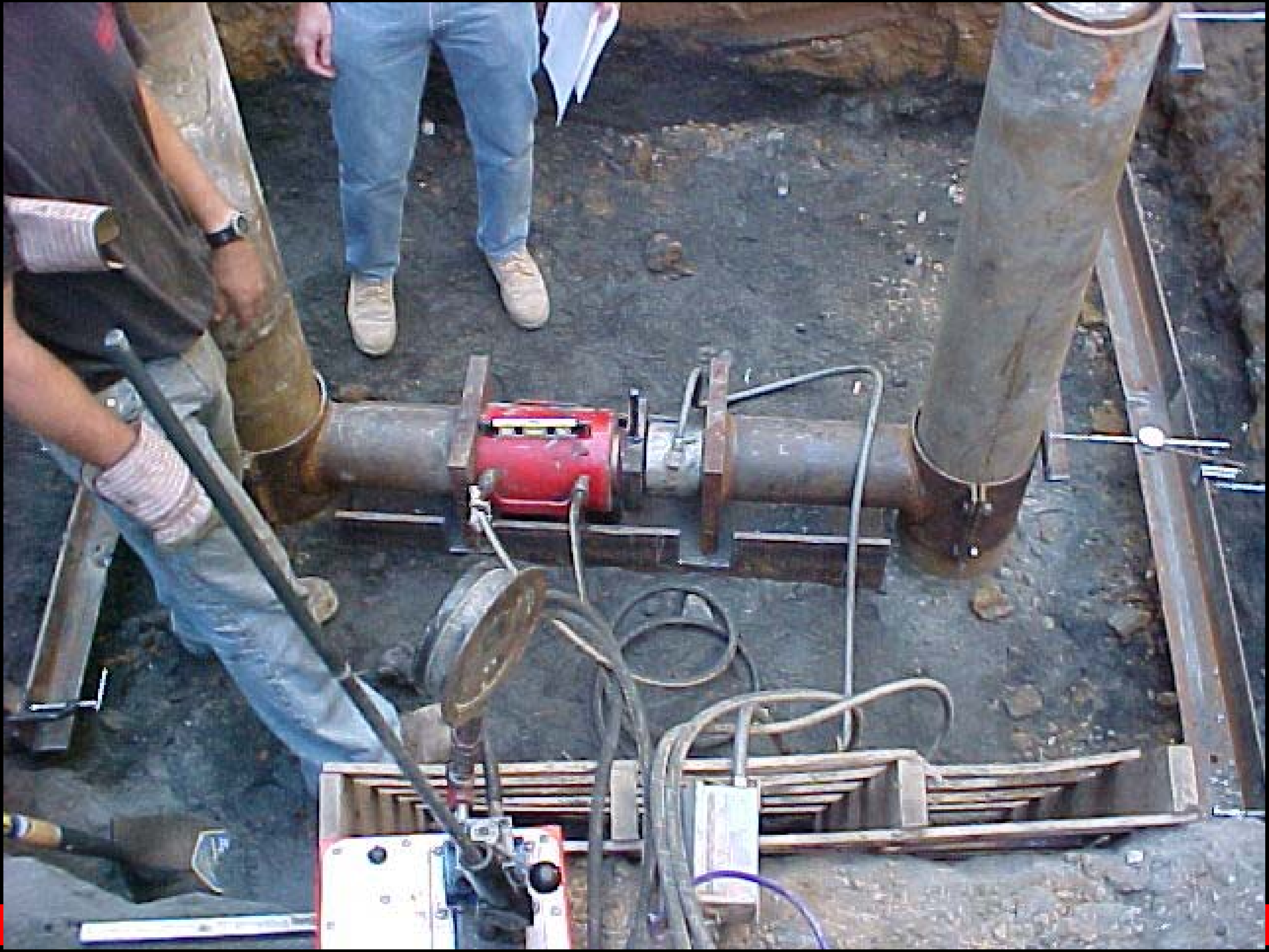

PILE	PILE PROPERTIES			SOIL PROPERTIES					ASSIGNED SOIL PARAMETERS						TEST	
	D mm	EI kN mm^2	DRILL METHOD	TYPE	N min	N max	N typ.	Dw M	Su kPa	F deg	g kN/m^3	g'avg kN/m^3	f kN/m^3	kh kPa	zP cm	dpit cm
A1	244	1.914E+10	Rotary Duplex with water	Sandy Lean Clay	12	25	19.0	6.7	129	0	19.6	19.6		4525	18	122
A2	244	1.914E+10			12	25	19.0	6.7	129	0	19.6	19.6		4525	24	122
C1	244	1.914E+10	Rotary Duplex with water	Sandy clay or silty clay	8	15	13.3	8.7	86	0	18.9	18.9		3016	24	137
C2	244	1.929E+10			8	15	13.3	8.7	86	0	18.9	18.9		3016	21	134
MR1	244	1.914E+10	Rotary Duplex with water	Flyash	4	4	4.0	3.0	0.0	25	14.1	13.8	1923		30	131
MR2	244	1.927E+10			4	4	4.0	3.0	0.0	25	14.1	13.8	1923		30	131
Z1	244	2.056E+10	Rotary Duplex with water	Silty sand with gravel	41	61	50.3	13.3	0.0	35	19.6	19.6	15043		30	107
Z2	244	2.058E+10			41	61	50.3	13.3	0.0	35	19.6	19.6	15043		27	107
G1	244	1.929E+10	Rotary Eccentric Percussive Duplex with Air	silt & sand to 2.4 m, then dense sand with silt & gravel	3	57	13.4	0.0	0.0	30	19.6	9.8	8014		18	130
G2	244	1.929E+10			3	57	13.4	0.0	0.0	30	19.6	9.8	4701		15	130
MC1	197	7.662E+09	Rotary Duplex with water	Fill – Silty Clay with sand	5	12	9.3	21.5	100	0	19.6	19.6		4525	52	134
MC2	197	7.662E+09			5	12	9.3	21.5	100	0	19.6	19.6		4525	55	137
MC3	197	7.662E+09			5	12	9.3	21.5	100	0	19.6	19.6		4525	40	143
MC4	197	7.662E+09			5	12	9.3	21.5	100	0	19.6	19.6		4525	27	131
B1	254	4.718E+09	Single Tube = Ext Flush	Fill – silty sand to silty sandy gravel	3	16	8.0	2.4	0.0	30	18.9	16.9	2645		15	122
B2	254	4.718E+09			3	16	8.0	2.4	0.0	30	18.9	16.9	2645		15	122
O1	381	4.348E+10	Open Hole with Air	Stiff silty clay/ clayey silt with chert fragments	12	44	24.5	15.2	96	0	17.3	17.3		3352	23	76
O2	381	5.051E+10			12	44	24.5	15.2	96	0	17.3	17.3		3352	23	76
O3	381	4.348E+10			12	44	24.5	15.2	96	0	17.3	17.3		3352	23	76
O4	381	5.051E+10			12	44	24.5	15.2	96	0	17.3	17.3		3352	23	76

TABLE No. 1 Summary of Test Pile Data

PILE AND SOIL SUMMARY

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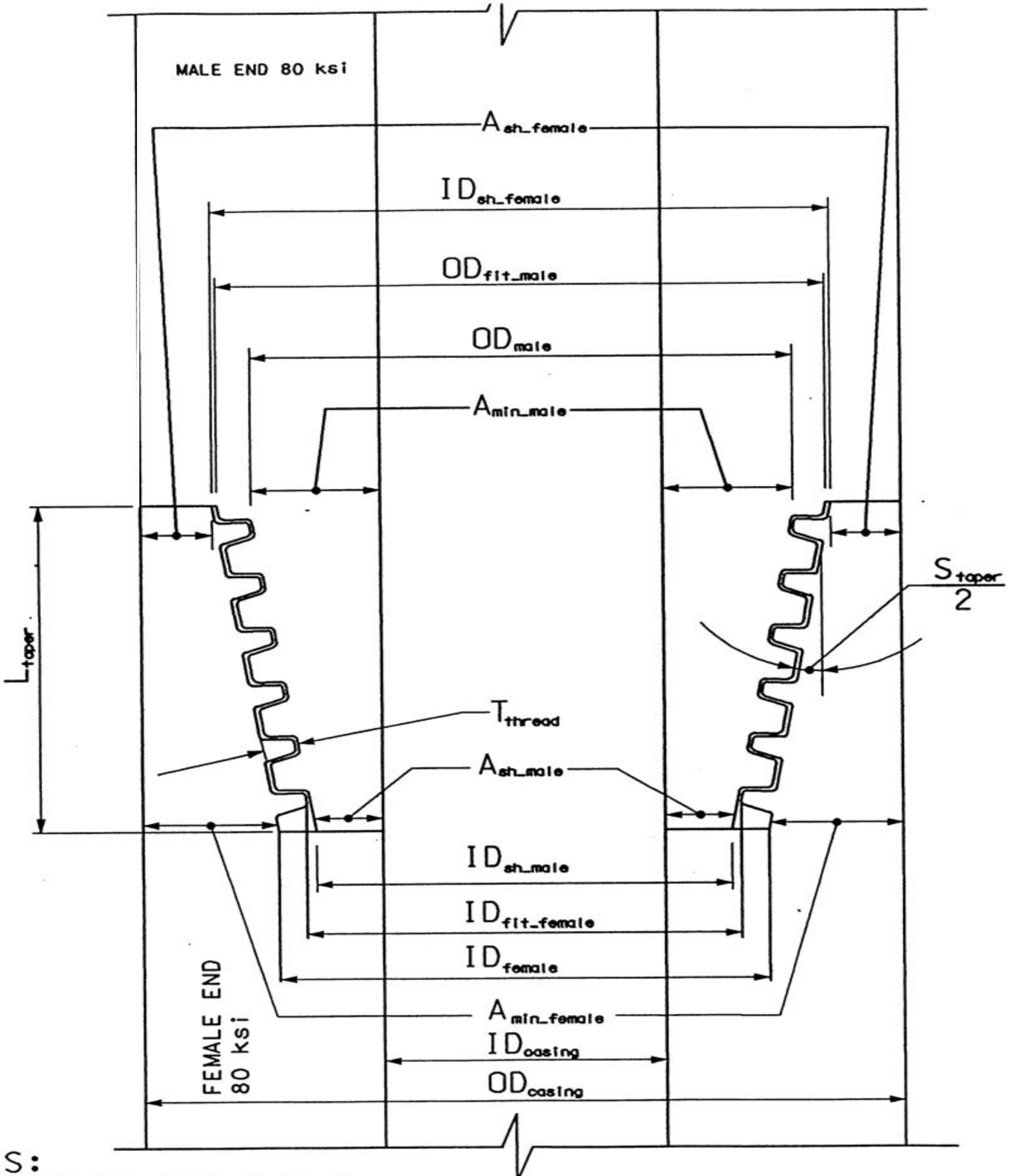
Typical Casing Joint



NOTES:
1) THREADS ARE N.T.S.
2) TAPER AND WALL ARE EXAGGERATED

Transformed Section

- For consistency and to eliminate a source of difference, the composite pile stiffness (EI) was determined using the LPILE program.
- The result was typically near the average of the uncracked transformed section and the steel only section.
- All analysis neglected the reduced EI over discrete lengths at the threaded joints of the drilled pipe. The only method that would be able to consider this is LPILE by using variable EI along the pile length. The effect of this unconservative assumption is discussed in the “Comparison of Results” section below.

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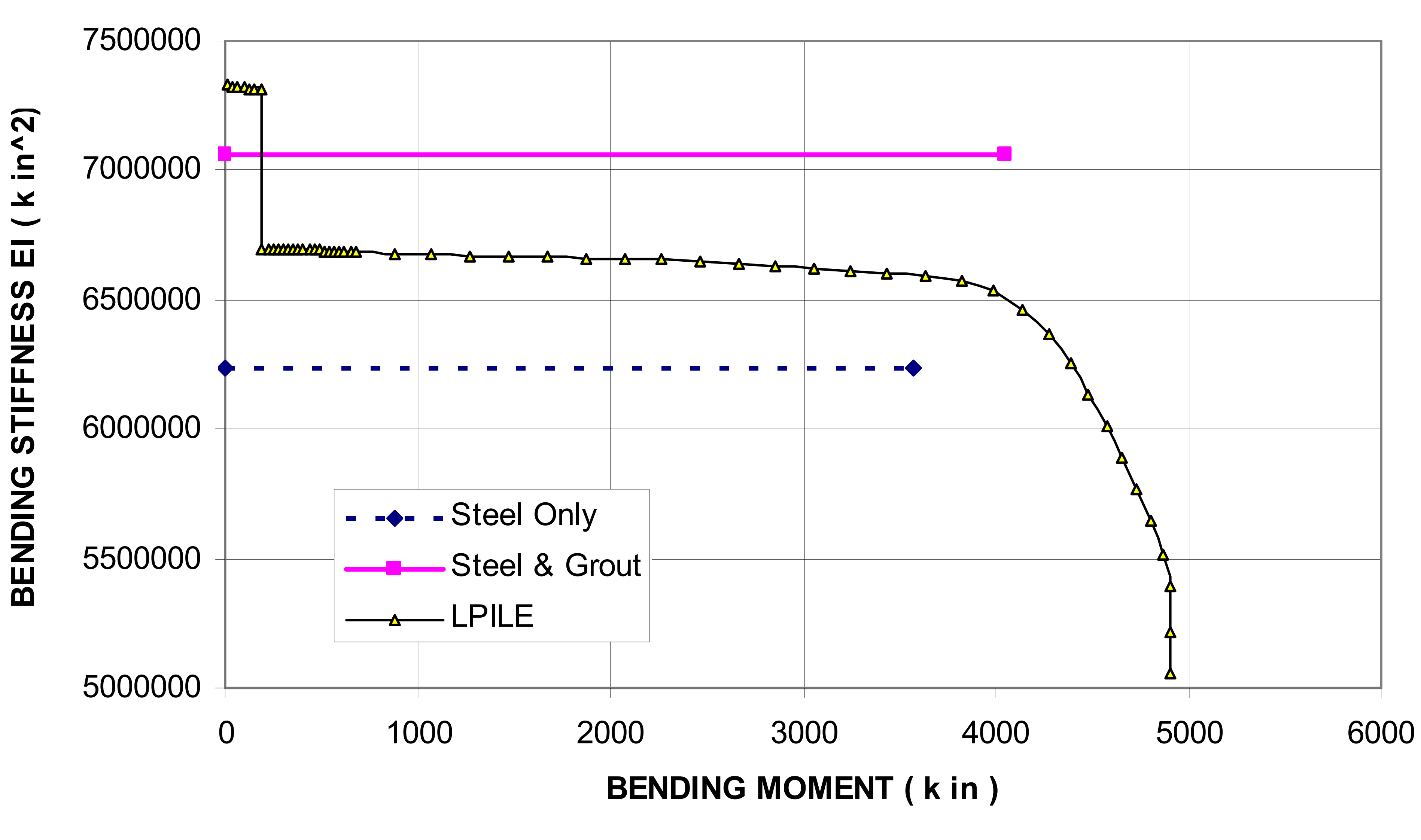

7500000
7000000
6500000
6000000
5500000
5000000
BENDING STIFFNESS EI (k in^2)
0
1000
2000
3000
4000
5000
6000
BENDING MOMENT (k in)
Steel Only
Steel & Grout
LPILE

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Characteristic Load Method (CLM)

This method is available as a spreadsheet from the Virginia Tech, Center for Geotechnical Practice and Research.

Per Clarke and Duncan (2001), "*The characteristic load method (CLM) of analysis of laterally loaded piles (Duncan et al.,1994) was developed by performing nonlinear p-y analyses for a wide range of free-head and fixed-head piles and drilled shafts in clay and sand. The results of the analyses were used to develop nonlinear relationships between dimensionless measures of load and deflection. These relationships were found to be capable of representing the nonlinear behavior of single piles and drilled shafts quite accurately, producing essentially the same values of deflection and maximum moment as p-y analysis computer programs like COM624 and Lpile Plus 3.0. The principal limitation of the CLM method is that it is applicable only to uniform soil conditions.*"

When the water table was within 3 meters of pile subgrade, the weighted average effective unit weight (g'avg) was used as suggested in the CLM Manual, Clarke and Duncan (2001) and as shown in Table 1.

The deflections were determined both with the applied moment from the point of load application above the ground surface. This method does not provide rotations or bending moments versus depth.

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NAVFAC Method

The "NAVFAC" method is from NAVFAC (1986) and based on Reese and Matlock (1956). This method uses linear elastic coefficient of subgrade reaction and assumes "*that the lateral load does not exceed about 1/3 of the ultimate lateral load capacity*." For granular soil and normally to slightly overconsolidated cohesive soils, NAVFAC states "*the coefficient of subgrade reaction, Kh, increases linearly with depth in accordance with:*

(1)

where: ***Kh = coefficient of lateral subgrade reaction [F/L^3]***

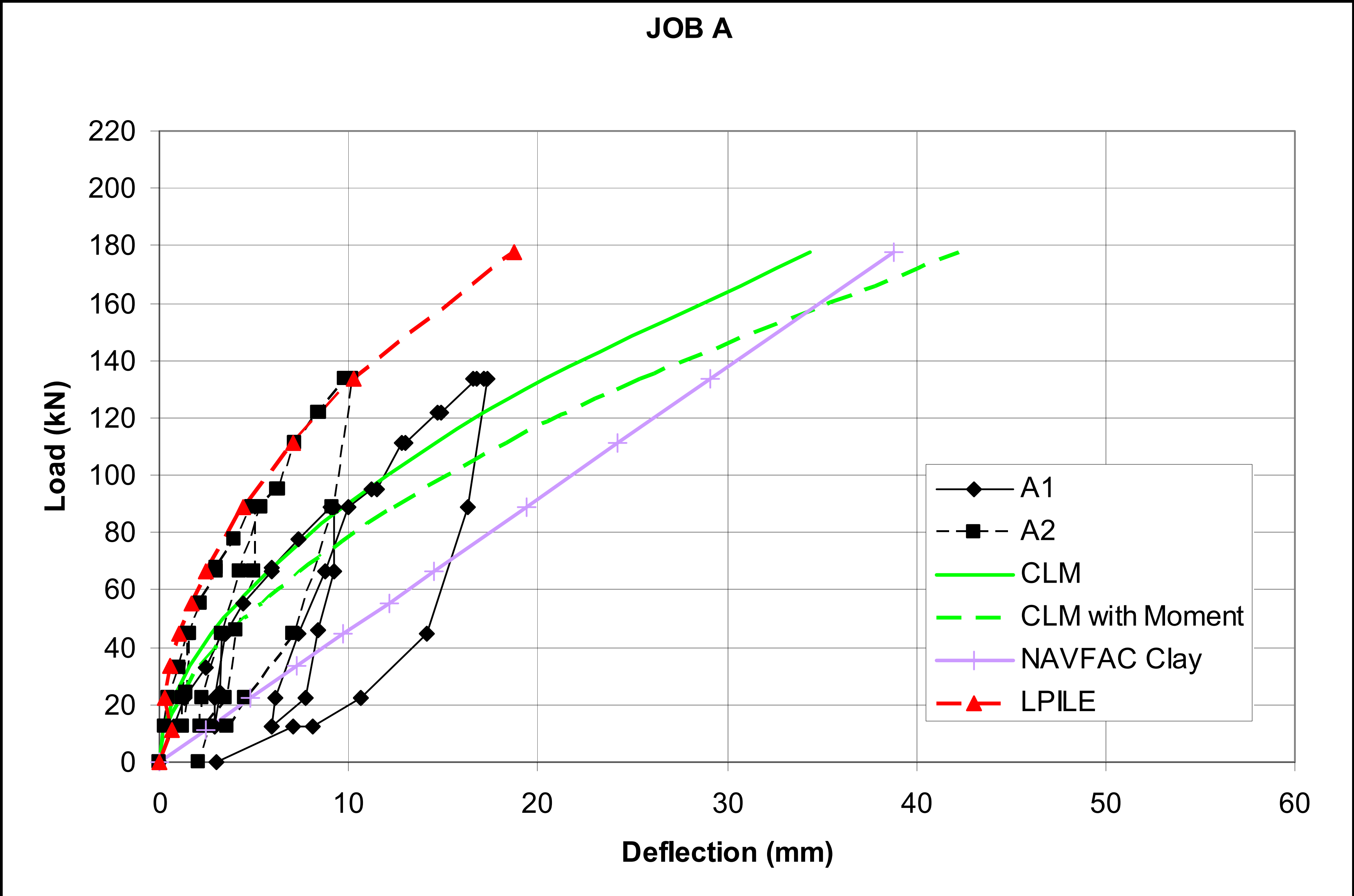
f = coefficient of variation of lateral subgrade reaction [F/ L^3]

z = depth [L]

***D = width/diameter of loaded area [L]*"**

For overconsolidated cohesive soils, NAVFAC states "*for heavily overconsolidated hard cohesive soils, the coefficient of lateral subgrade reaction can be assumed to be constant with depth. The methods presented in Chapter 4 can be used for the analysis; Kh, varies between 35c and 70c (units of force/length^3) where c is the undrained shear strength*." NAVFAC Chapter 4 presents traditional elastic modulus of subgrade reaction equations. The "free end, concentrated load" case was used. The units of 35c appear to be force/length^2. Therefore, the modulus of subgrade reaction used was Kb = 35su/b where b = pile diameter.

This method estimates the moment diagram versus depth and does not consider the effect of passive surcharge. This method does not easily deal with the applied moment from the applied load being above the ground surface and this was not considered.



JOB A
Load (kN)
220
200
180
160
140
120
100
80
60
40
20
0
0
10
20
30
40
50
60
Deflection (mm)
A1
A2
CLM
CLM with Moment
NAVFAC Clay
LPILE

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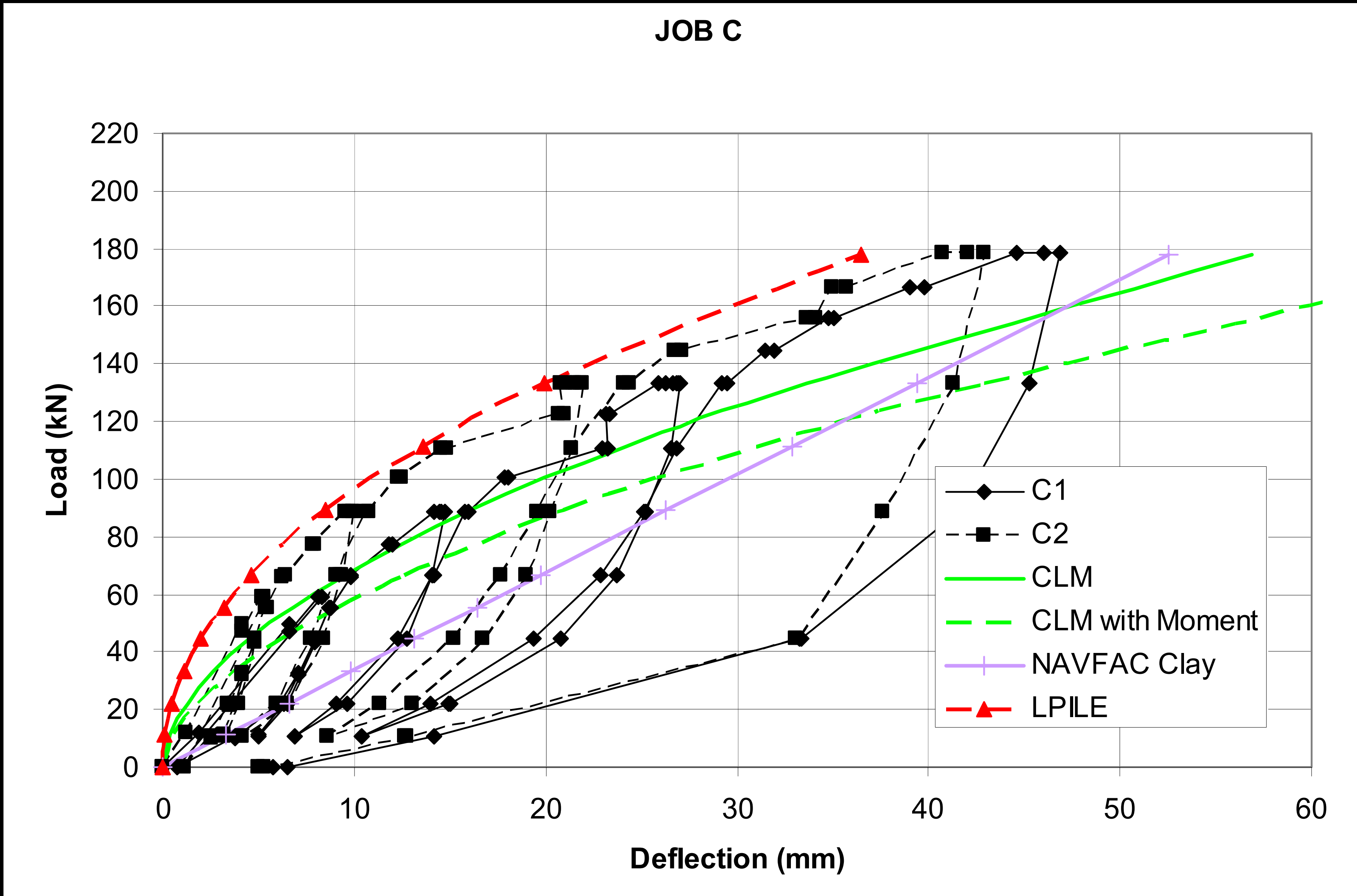
JOB C
Load (kN)
Deflection (mm)
220
200
180
160
140
120
100
80
60
40
20
0
0
10
20
30
40
50
60
C1
C2
CLM
CLM with Moment
NAVFAC Clay
LPILE

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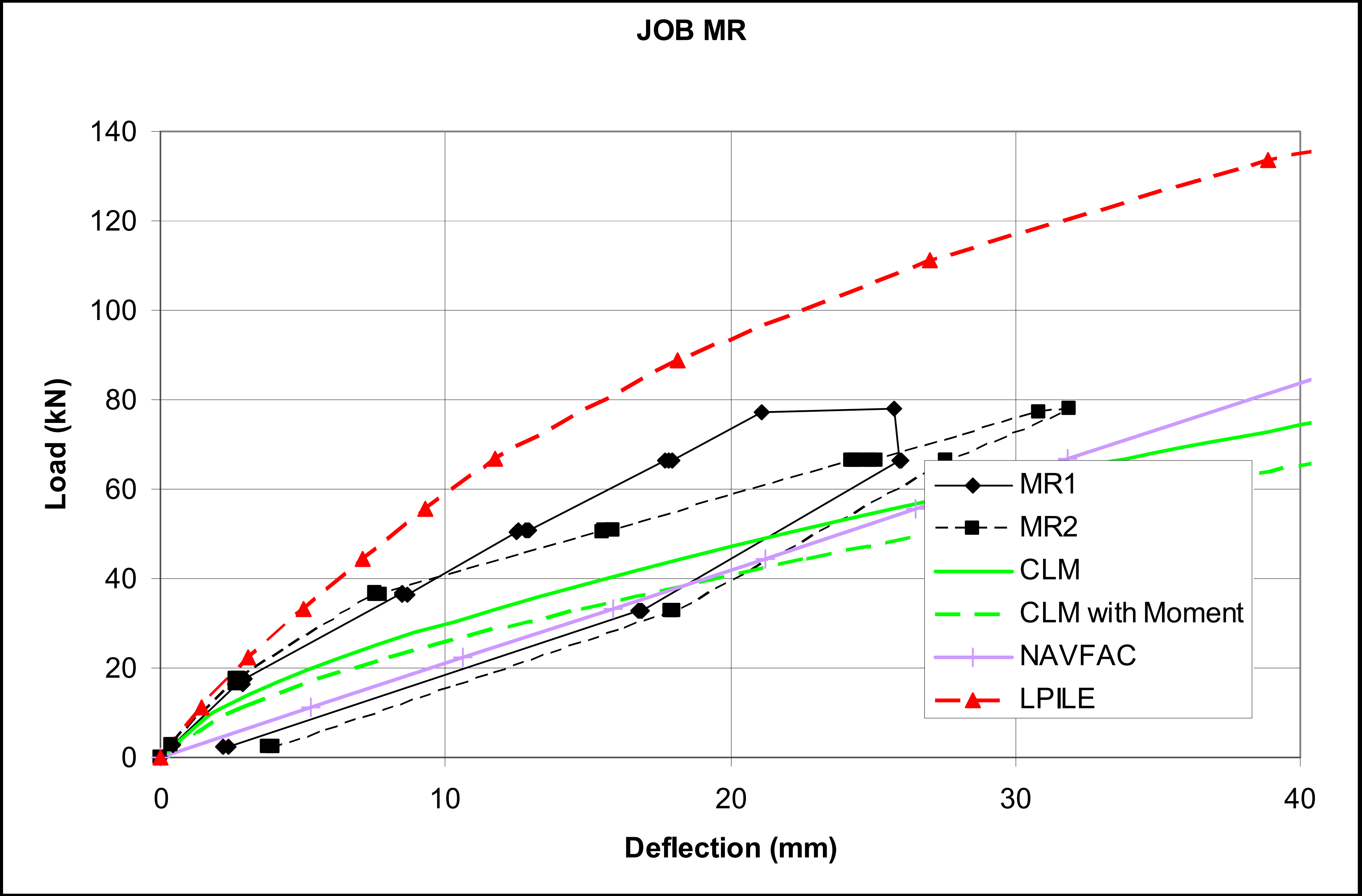
JOB MR
Load (kN)
Deflection (mm)
0
20
40
60
80
100
120
140
0
10
20
30
40
MR1
MR2
CLM
CLM with Moment
NAVFAC
LPILE

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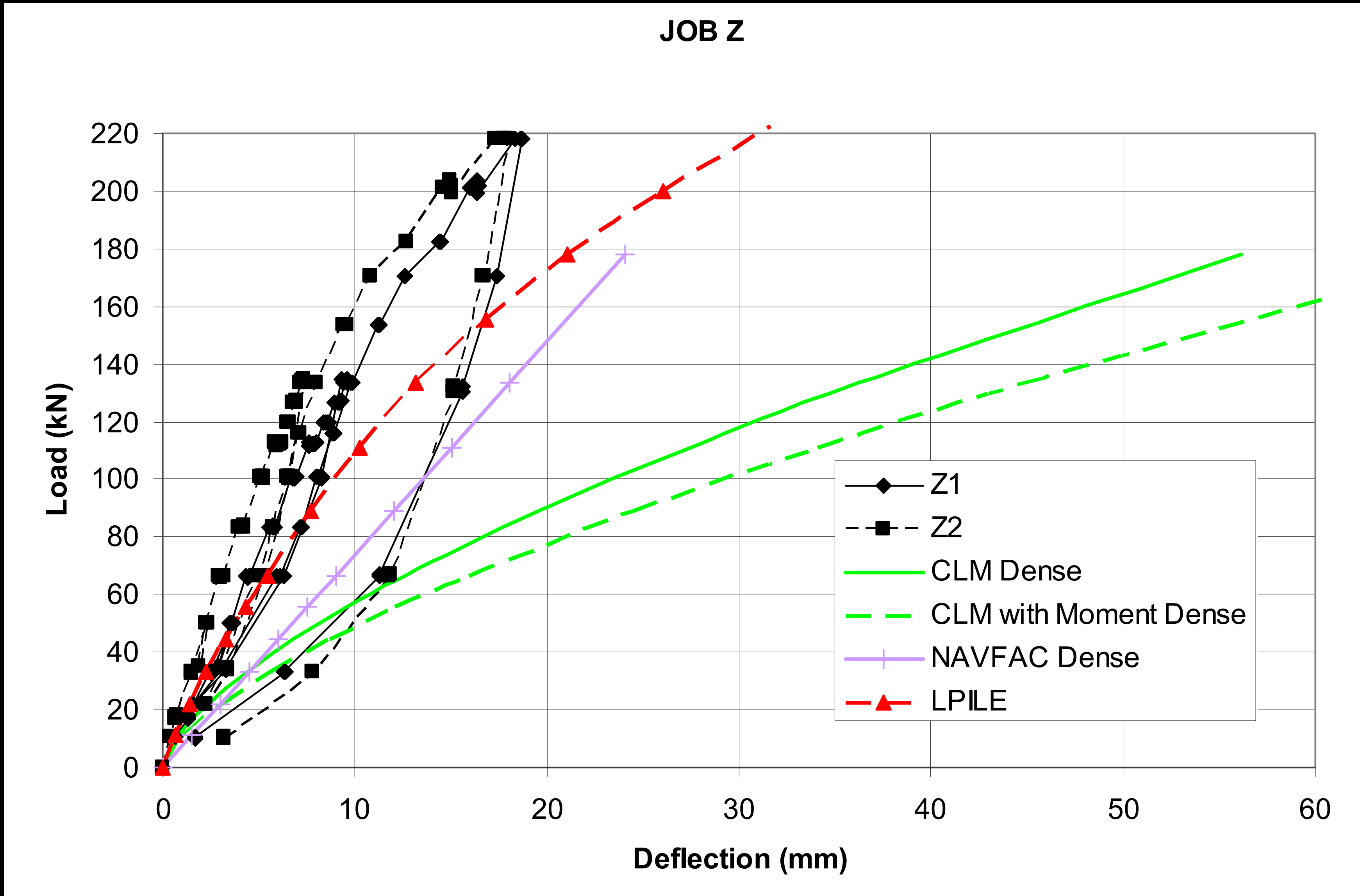

JOB Z
Load (kN)
0
20
40
60
80
100
120
140
160
180
200
220
Deflection (mm)
0
10
20
30
40
50
60
Z1
Z2
CLM Dense
CLM with Moment Dense
NAVFAC Dense
LPILE

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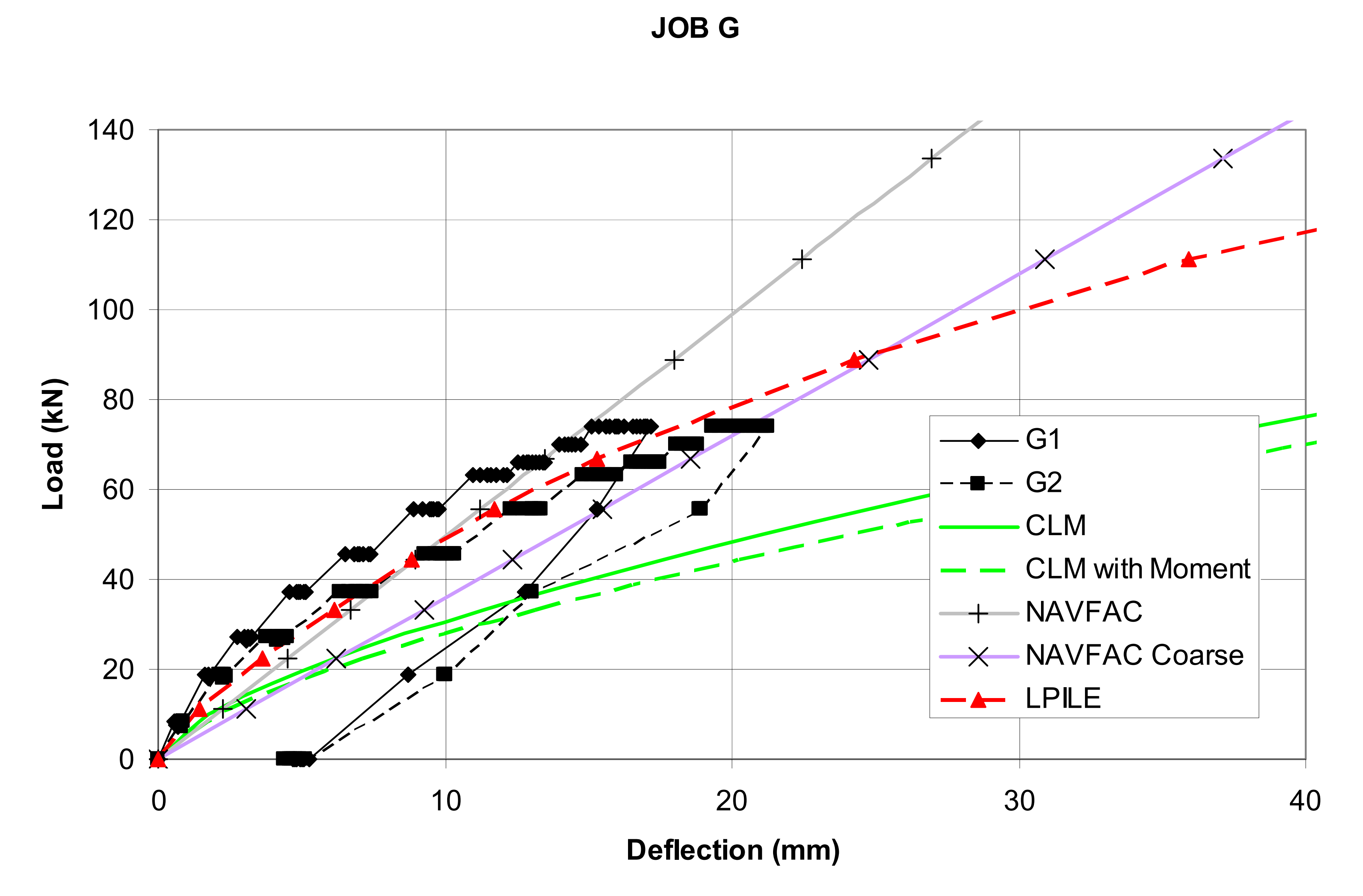
JOB G
Load (kN)
Deflection (mm)
0
20
40
60
80
100
120
140
0
10
20
30
40
G1
G2
CLM
CLM with Moment
NAVFAC
NAVFAC Coarse
LPILE

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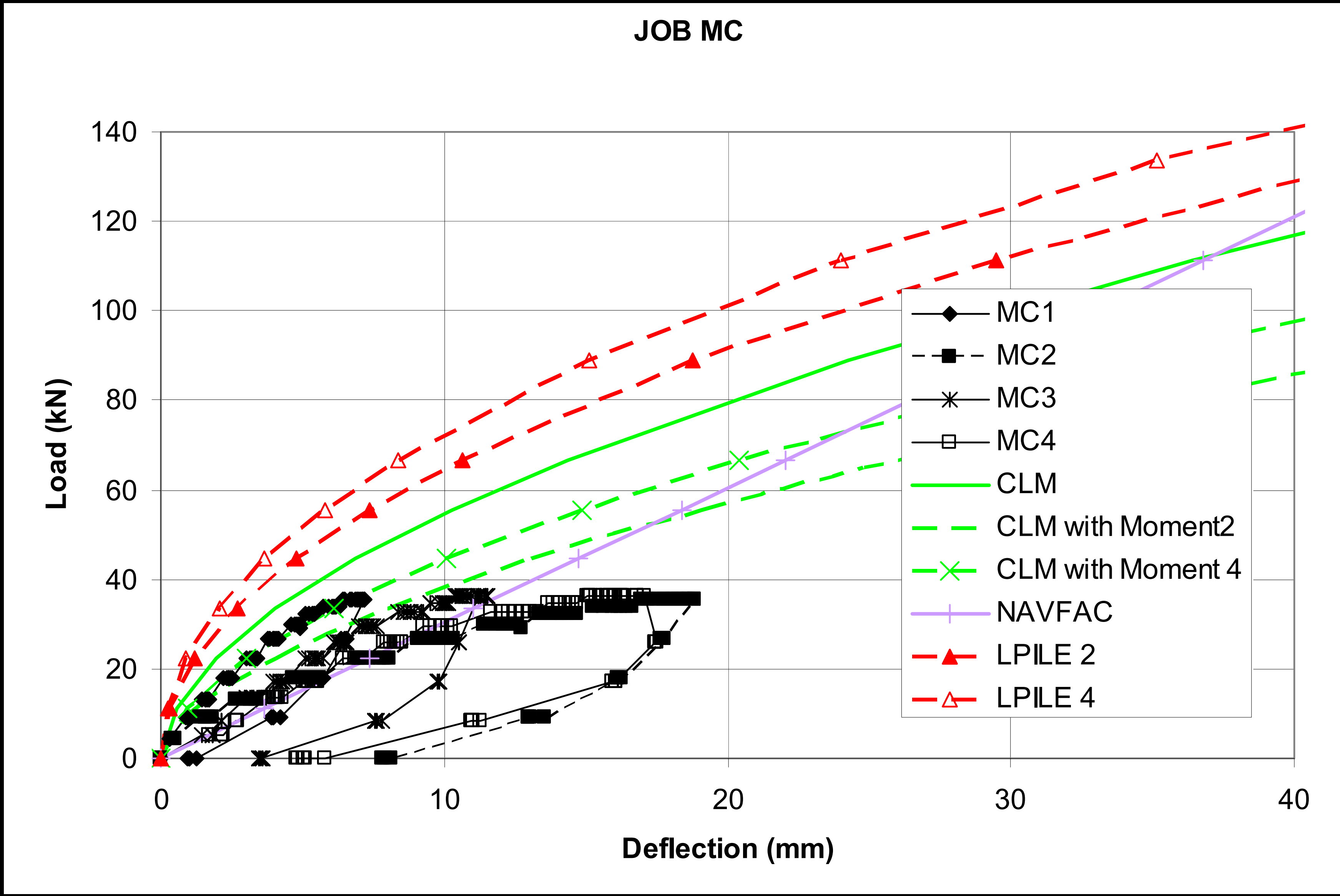
JOB MC
Load (kN)
Deflection (mm)
140
120
100
80
60
40
20
0
0
10
20
30
40
MC1
MC2
MC3
MC4
CLM
CLM with Moment2
CLM with Moment 4
NAVFAC
LPILE 2
LPILE 4

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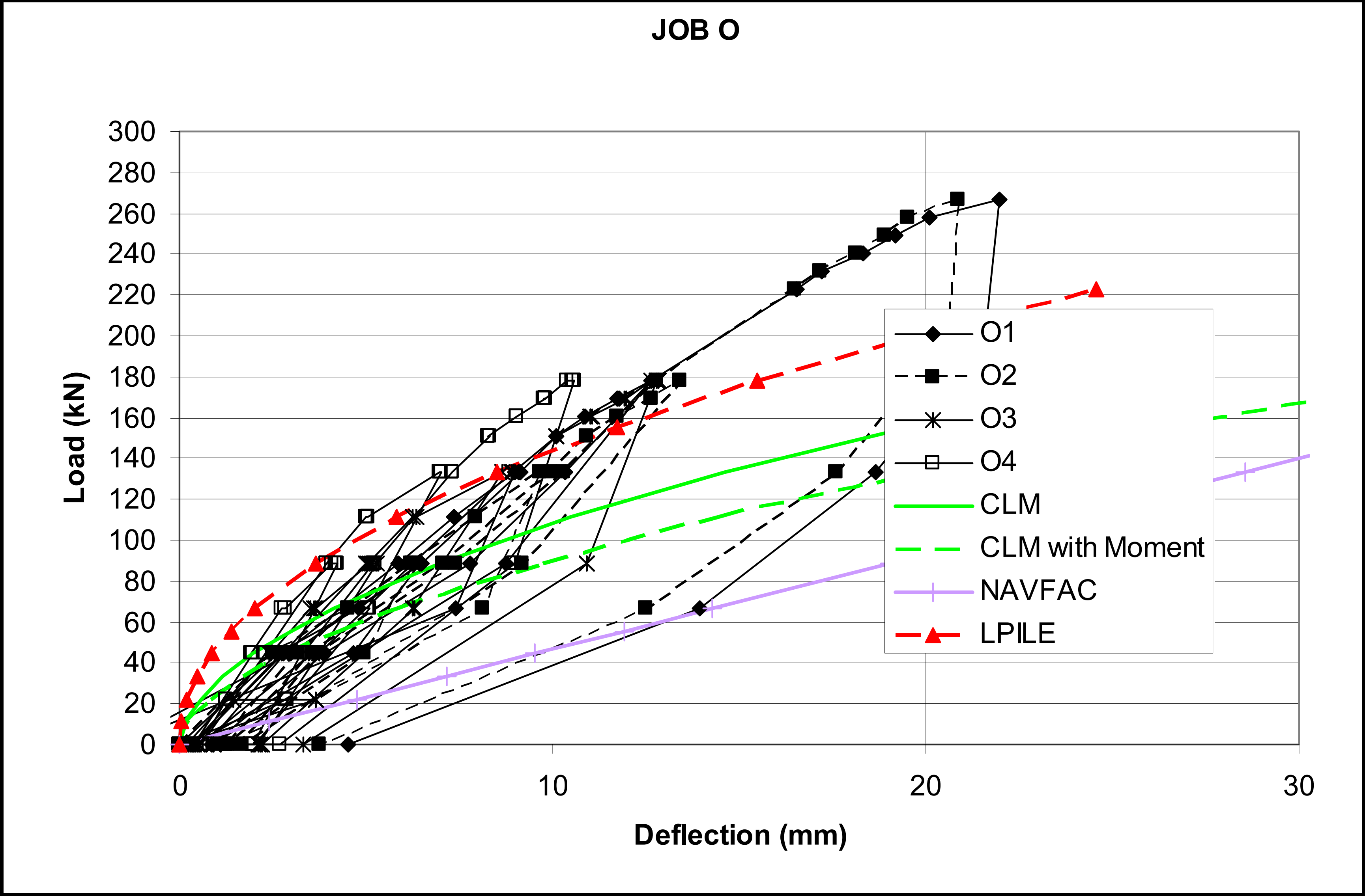
JOB O
Load (kN)
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
0
10
20
30
Deflection (mm)
O1
O2
O3
O4
CLM
CLM with Moment
NAVFAC
LPILE

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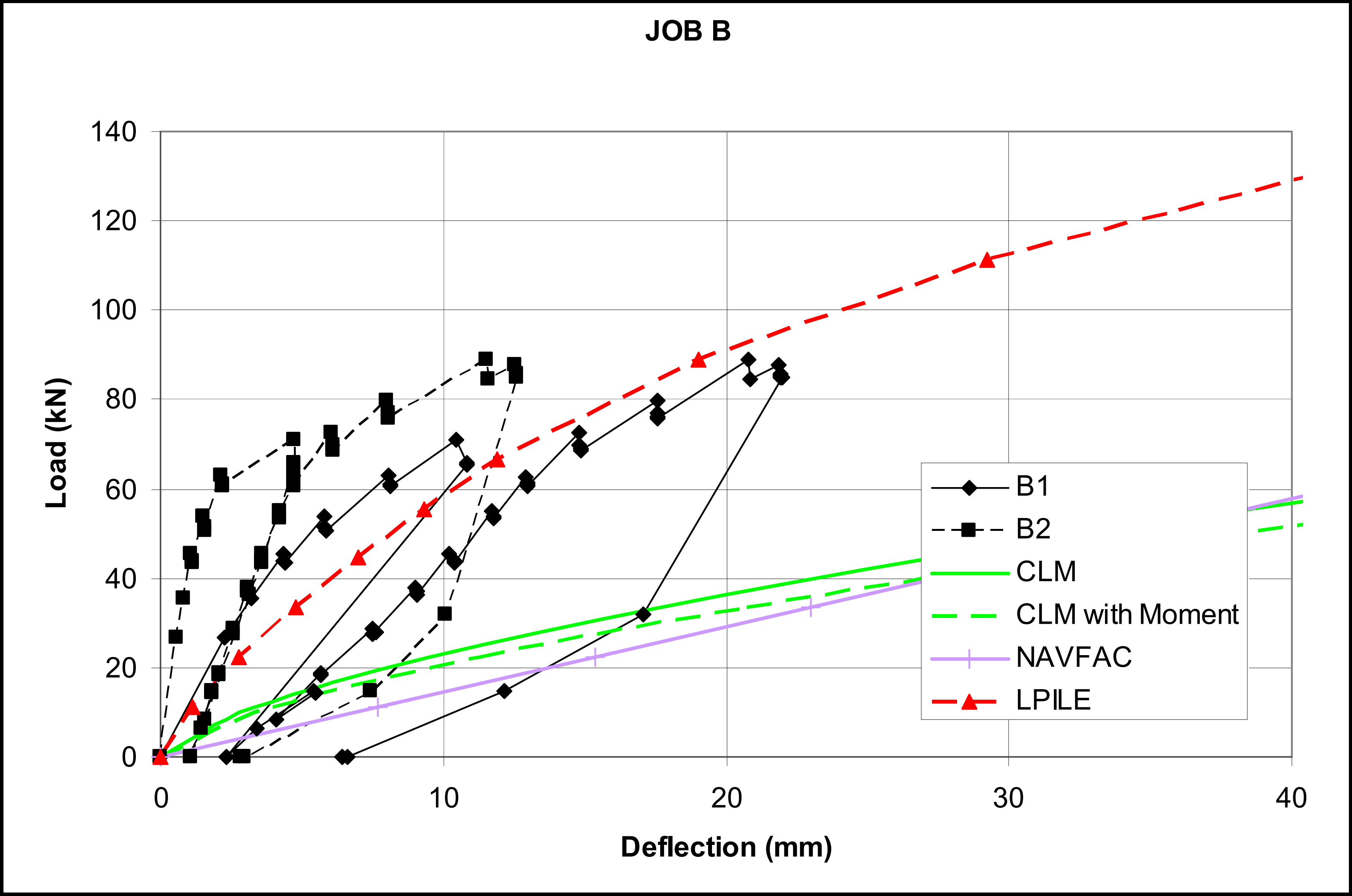
JOB B
Load (kN)
Deflection (mm)
140
120
100
80
60
40
20
0
0
10
20
30
40
B1
B2
CLM
CLM with Moment
NAVFAC
LPILE

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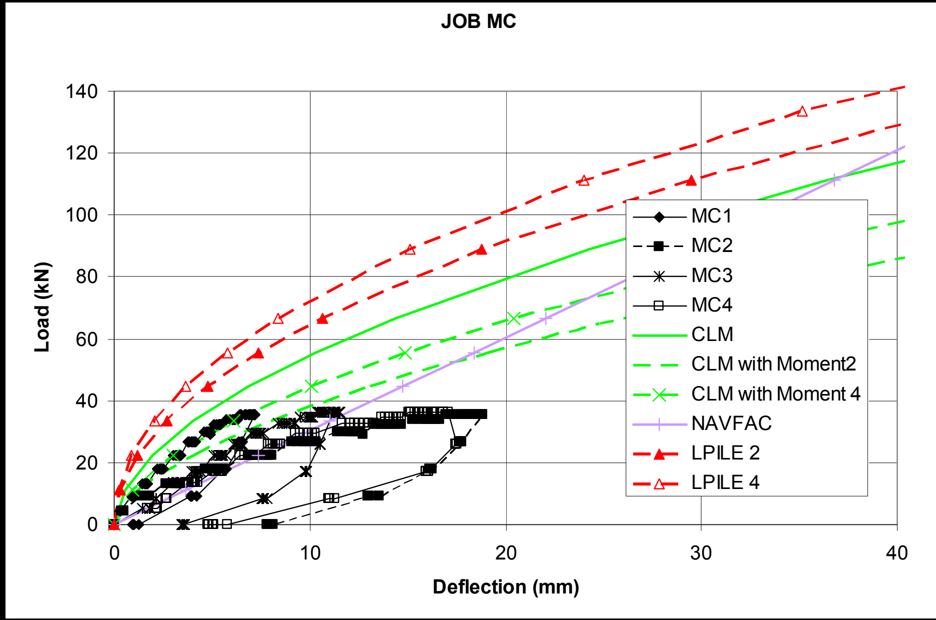
Comparison of Results

- Generally, the measured deflections were typically significantly less than predicted by CLM or NAVFAC.
- The LPILE analysis tended to provide the best fit. However, the measured deflections often exceeded the LPILE predictions, due primarily to the "passive surcharge" considered in LPILE. By comparing LPILE to CLM curves, the impact of this surcharge is significant even on clay sites. The pits did not provide a pure surcharge and were typically often 0.6 meters beyond the edge of the pile.
- The underestimated predictions with LPILE were also due to the fairly high undrained shear strengths, especially at site MC
- Since the measured deflections were close typically close to predicted, ignoring the reduction in EI of the threads in predicting deflections appears appropriate.
- The performance is judged to be dominated more by the soil strength than small sections with lower EI and than the initial soil stiffness chosen

Comparison of Results Cont Exception at Site MC

Measured deflections significantly exceeded calculations by LPILE and were near NAVFAC & CLM predictions. This is caused by

- fairly high undrained shear strength used when compared to the blow count
- the soil being a clayey fill, therefore pocket penetrometer readings may represent “chunks” versus the mass
- the limits of the pit excavation was approximately 2 meters beyond the piles. LPILE analysis without the surcharge would be similar to CLM
- perched water near the bottom of the pit



Combined Stresses

The simple method to determine the combined stresses is:

$$\frac{P}{Pall} + \frac{M}{Mall} < 1$$

Where: *P* = applied axial load

Pall = allowable axial structural load of pile

M = bending moment from analysis

Mall = allowable bending strength of the pile

The allowable bending moment must consider the threaded joint section of the pile. An approximation for the section modulus of the flush joint thread length is 50% of the section modulus of the solid pipe.

Often designers allow higher bending stresses than axial stresses. This is not clear in various Codes.

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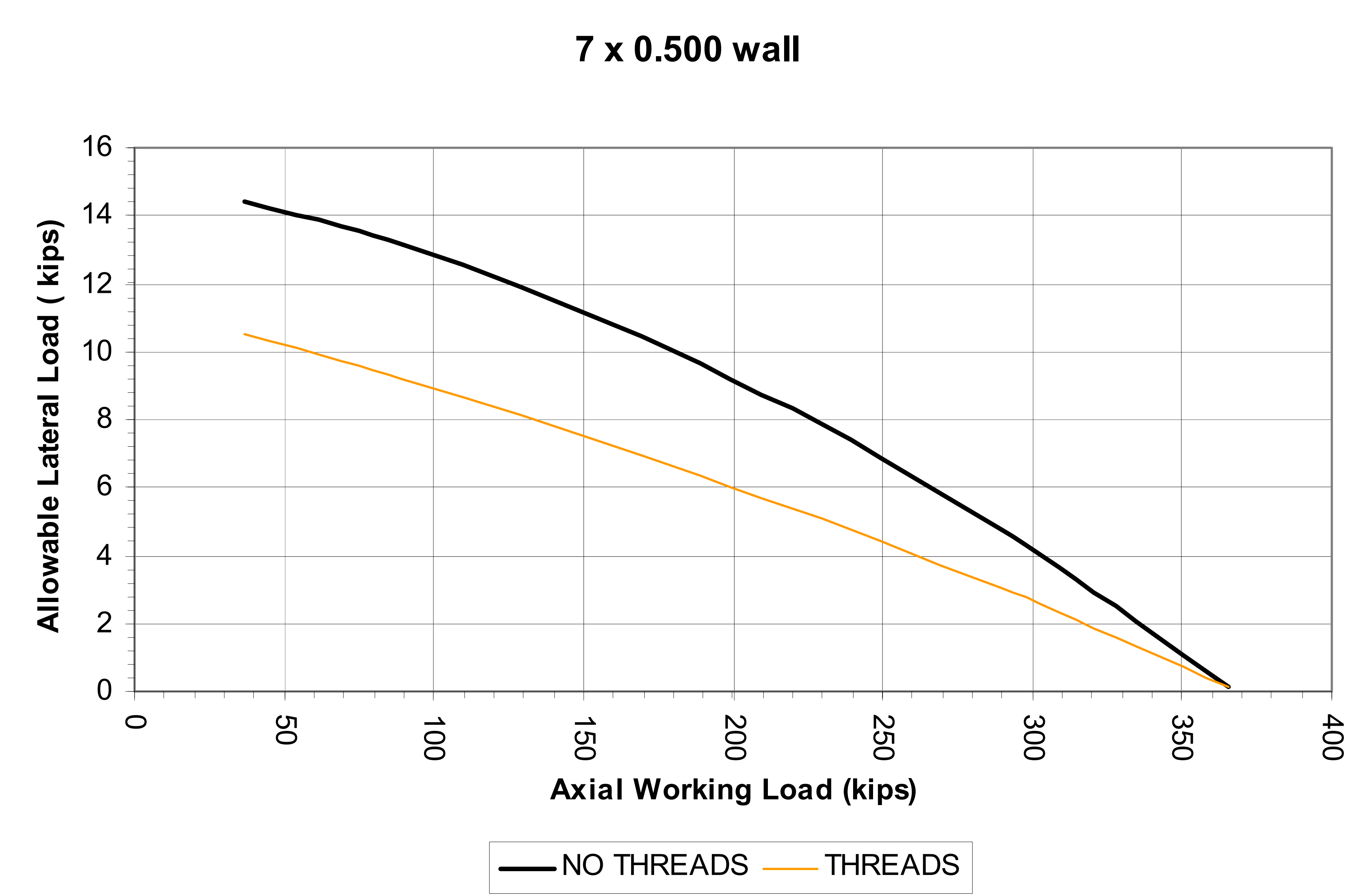

Combined Stress

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OPTIONS FOR INCREASING LATERAL RESISTANCE OF PILES OR PILE GROUPS

Lateral capacity of an individual micropile or a micropile group can be increased by

- installing an oversized casing in the top portion of the pile where moments are high,
- constructing a larger pile diameter at the top (bending moment decreases with increased diameter and passive resistance),
- embedding the pile cap deeper, or
- creating a "fixed" connection. Although pure fixity between the pile and pile cap with zero rotation is unrealistic.

Lateral capacity of a pile group can also be increased by battering piles or making the group larger, i.e. increasing the pile spacing to decrease the group reduction effects.

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ANALYSIS FOR BATTERED PILES

- A simple graphical procedure for estimating the compressive and tensile forces in micropile groups containing not more than three rows of micropiles is described in Tomlinson (1987) and Teng (1962).
- For analysis of three-dimensional pile groups that considers nonlinear soil response and micropile-soil-micropile interaction, the GROUP 6 program can be used.

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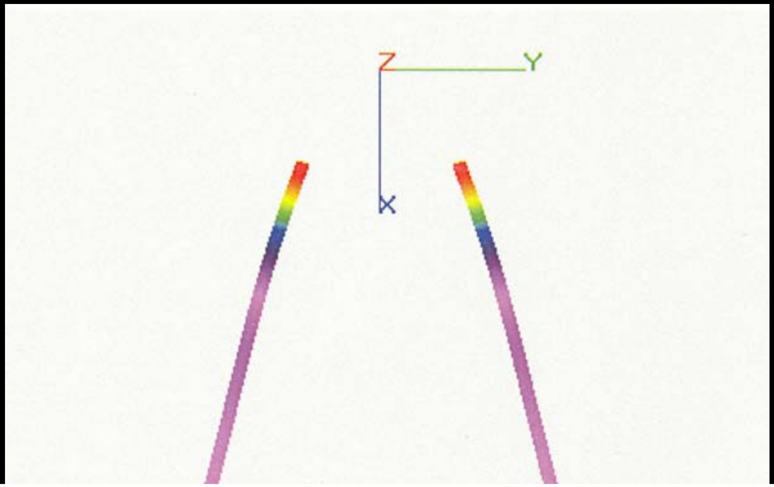
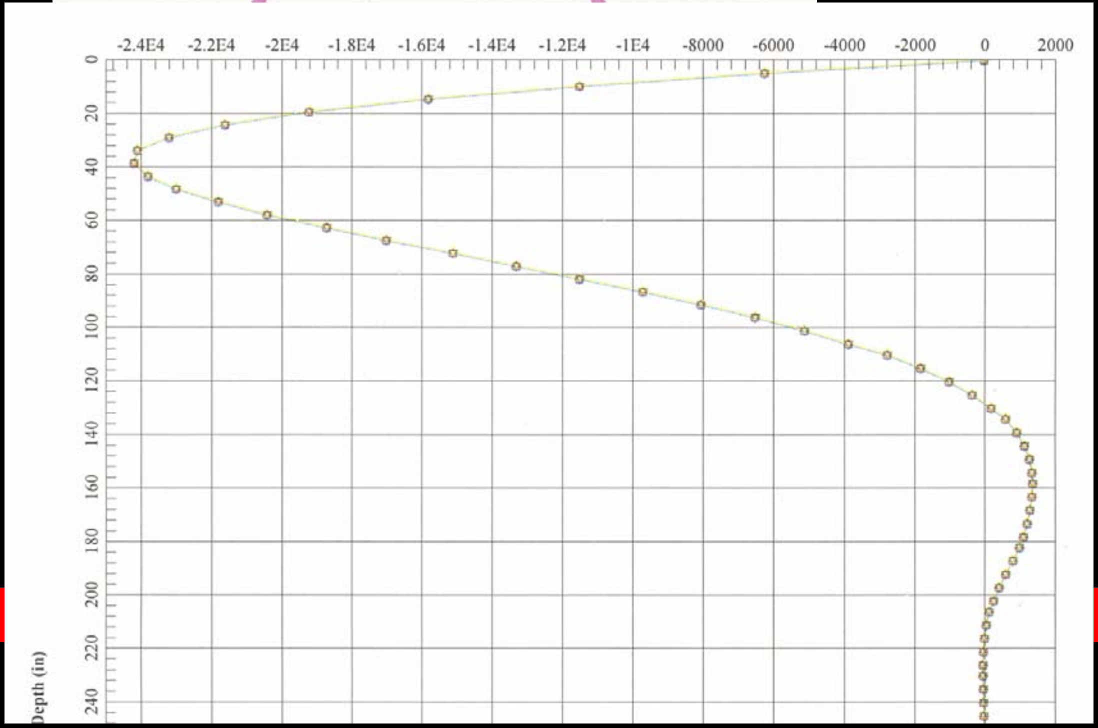
ANALYSIS FOR BATTERED PILES





- An interesting outcome from working with the GROUP program is the realization that even battered pile groups have bending moments in the piles.
- Battered piles can substantially reduce and balance, but not eliminate, the bending moments in the piles.
- Piles should have pipe at the top

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CONCLUSIONS

- Micropile foundations can be and have been designed to carry substantial lateral loads. The loads can be resisted by the lateral load resistance of the micropile and/or by battering the piles. In either case, the micropiles must be designed for the resulting combined stresses often resulting in the need to include casing near the top of the pile for bending strength.
- Lateral tests on micropiles have generally shown less deflection than predicted due to typical conservatism in assigned soil parameters or neglecting "passive surcharge" due to the top of the pile being below ground surface. The elastic solutions generally greatly overestimate deflection.
- The tests and analyses as well as other literature show that the lateral load performance is very sensitive to the soil type and shear strength in the upper 2 to 5 meters of the pile. Therefore, this zone should be well sampled and characterized in subsurface investigations including laboratory testing for projects expecting deep foundations.

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