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PREFACE

The activities of the International Conference is in line and very appropriate with the vision and mission of the UBL to promote training and education as well as research in these areas.

On behave of the First International Conference of Engineering and Technology Development (ICETD 2012) organizing committee; we are very pleased with the very good responses especially from the keynote speakers and from the participants. It is noteworthy to point out that about 45 technical papers were received for this conference

The participants of conference come from many well known universities, among others: Universitas Bandar Lampung, International Islamic University Malaysia, University Malaysia Trengganu, Nanyang Technological University, Curtin University of Technology Australia, University Putra Malaysia, Jamal Mohamed College India, ITB, Mercu Buana University, National University Malaysia, Surya Institute Jakarta, Diponogoro University, Unila, Universitas Malahayati, University Pelita Harapan, STIMIK Kristen Newmann, BPPT Lampung, Nurtanio University Bandung, STIMIK Tarakanita, University Sultan Ageng Tirtayasa, and Pelita Bangsa.

I would like to express my deepest gratitude to the International Advisory Board members, sponsors and also welcome to all keynote speakers and all participants. I am also grateful to all organizing committee and all of the reviewers which contribute to the high standard of the conference. Also I would like to express my deepest gratitude to the Rector which give us endless support to these activities, such that the conference can be administrated on time.

Bandar Lampung, 20 Juni 2012

Mustofa Usman, Ph.D
ICETD Chairman

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Validation Study of Simplified Soil Mechanics Method Design with Kentledge Pile Loading Test of Bored Pile

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Abstract: **The use of static load testing in optimising design and providing verification of suitability and constructability continues to be unsurpassed in the foundation industry.**
Another purpose of that load testing is either to validate the design before construction and/or to check compliance with the specification during construction The aim of this paper is to validate the design by comparing the ultimate load of Kentledge bored pile loading test with the its design, using Simplified Soil Mechanics method. The difference between Kentledge loading test and Simplified Soil Mechanics method are 13 % and 25%, if the pile bearing ultimate capacity of Kentledge loading test is computed using Mazurkiewicz and Davisson MT method.

Keywords : **Kentledge bored pile loading test , Simplified Soil Mechanics Method Bore Pile Design, Mazurkiewicz and Davisson MT method**

I. INTRODUCTION

Producing competent bored piles is one of the most difficult tasks facing a civil engineer. Since their production process is carried out in a hostile underground environment and is largely invisible,bored piles unavoidably contain flaws. On the otherhand, replacement of faulty foundation piles is at best impractical. This is the reason why quality control of finished foundation piles grew to rely on various methods. A flaw is any deviation from the planned shape and/or material of the pile (Amir, 2002). It may thus involve inclusions of foreign material, necking,bulging and also piles that are too short. Table 1 shows the flaw occurrence ratio. Pile load testing provides an opportunity for continuous improvement in foundation design and construction practices, while at the same time fulfilling its traditional role of design validation and routine quality control of the piling works.

The strategy for pile testing needs to be established at the time the piles are being designed. For most projects the main purpose of pile testing is either to validate the design before construction and/or to check compliance with the specification during construction. However in some cases there are benefits in using testing for design development or research to provide the best solution. Testing strategies can therefore be divided into four main categories: (1) Design validation, (2) Quality control, (3) Design development and (4) Research.

The scope of testing will depend on the complexity of the foundation solution, the nature of the site and the consequences if piles do not meet the specified requirements. The pile designer therefore needs to assess the risks and develop the testing regime accordingly.

The main risks are: (1) Insufficient site investigation,(2) Lack of experience of similar piles in similar ground conditions,(3) Insufficient time to verify the pile design and realise any savings (4) Cost and programme implications of undertaking the pile tests,(5) Cost and programme implications of a foundation failure for simple structures on a site where the ground conditions are well understood and there is pile test data from adjacent sites that have used similar piling solutions, then the risks are low and pile load testing can usually be restricted to routine checks for compliance or can even be omitted.

For situations where the ground conditions or structural requirements are complex, or there is little experience of similar piling work, then careful evaluation of the piling proposals is essential prior to embarking on the main piling works. Here the testing regime may need to be considered in two phases comprising preliminary pile testing before the main piling works and then proof testing of working piles.

The testing strategy for pile testing should address a project-specific set of stated objectives, which should include the following: (1) To minimise risk by investigating any uncertainties about the ground conditions, contractor's experience or new piling techniques (2) To optimise the pile design in terms of size, length and factor of safety to confirm any pile installation criteria such as founding strata identification, pile set or pile refusal criteria (3) To assess buildability, site variability, pile uplift, soil remoulding along the pile shaft or relaxation at the pile toe,(4) To check that the pile performance meets the required load/settlement behaviour during loading (Federation of Piling Specialists, 2006).

This paper is intended to present the study of design validation by comparing the ultimate load of loading test with the its design.

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TABLE 1
FLAW OCCURRENCE RATIO

Location	No.of piles	Testing	Piles with		Ref.
			Nu	[	
United Kingdom	9,550	Sonic (analog)	161	1.7	1
California	2,986	Mostly radioactive	-	20	2
US site X	470	Visual inspection	-	64?	2
US site Y	171	Visual inspection	-	76?	2
Asia	300	Visual	-	>20	2
Italy	6,865	Ultra		12	3
Israel site"R"	253	Sonic (digital)	57	22.5	4
	40	Ultrasonic	26	65	4
Israel	65	Ultra	28	4	4

1. Fleming et al. (1992)
2. O'Neill & Sarhan (2004)
3. Faiella & Suprebo (1998)
4. Piletest.com files

2. Kentledge Test.

Should the ground conditions or site constraints preclude the use of reaction piles, the alternative is to use kentledge. A frame is assembled over the pile to be tested on top of which an amount of weight (a minimum 110 to 120% of maximum test load) is safely stacked. This takes the form of concrete blocks of regular dimensions and weight although steel ingots can be used provided that their weight can be assessed with reasonable accuracy. The size of the testing apparatus is generally a function of the pile size and loading to be applied.At the time of assembly, the presence of the additional cranes and associated transport deliveries will increase this working area.

The test used type of static load testing, that is the Maintained Load Test (MLT).In the MLT, the load is applied to the pile in discrete increments and the resulting pile movement/ settlement monitored. Subsequent load increments are only applied when the minimum specified time period has elapsed and the rates of induced settlement are below the specified criteria. The normal Indonesia practice is to load the pile up to design verification load (DVL), then to unload back to zero loading. Subsequent load cycles are applied, taking the loading to specified values above the DVL depending on the requirements of the test. The test is conducted from January 9, 2012 until January 11, 2012 and determining the load/settlement performance of a pile under working loads up to 2 times design working load conditions.

3. Case Study.

3.1. Site Geology

The project is located in central of Jakarta. As is generally the ground in Jakarta,the upper layer, consisting of clay that comes from the sea, so it called marine clay. If that soil is tested, it was found that they content high levels of salt. In this project that soil is located at a depth of -0.50 m to a depth of -15.00 m, so thick as 14.5 m. From the drill logs, soil type is dominated by silty clay soil, with a value of N such as 6-10, medium consistency.

Under this layer, there is a lens of sandy soil from -15.00m to a depth -19.00 m. This soil is the result of lava flows from a volcano that erupted thousands years ago. This soil is usually used as the foundation of story building. The soil is hard and dense with a value of N spt = 50.The problem is, there are underground layers of clay / silt that has a value of N spt = 28 to 21, medium to stiff consistency, located at a depth of -19.00 m to -30.00 m, thickness of 11m. This clay type over consolidated (OC).

Under this layer, there is a layer of dense sand, with a value of N = 50. This soil also from the lava of a volcano, located at a depth of - 30.00 m to -50.00 m, 20 m thick. So, the tip of pile will lay on this layer.While in the depth of - 50 m to -60 m the layer of clay is found.

See Fig. 1 to 3.

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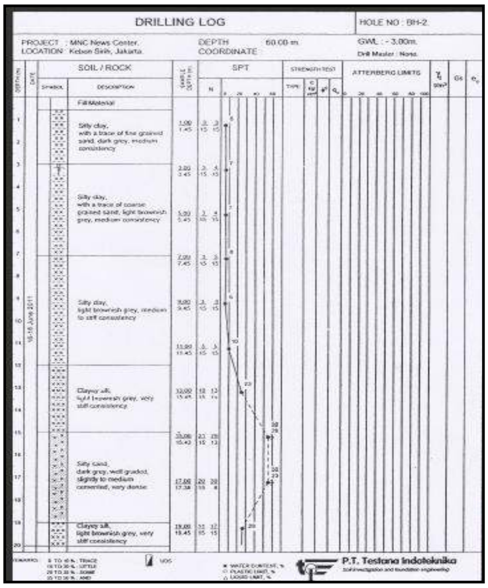

Fig. 1 Drilling Log and SPT from 0.00- 20.00m

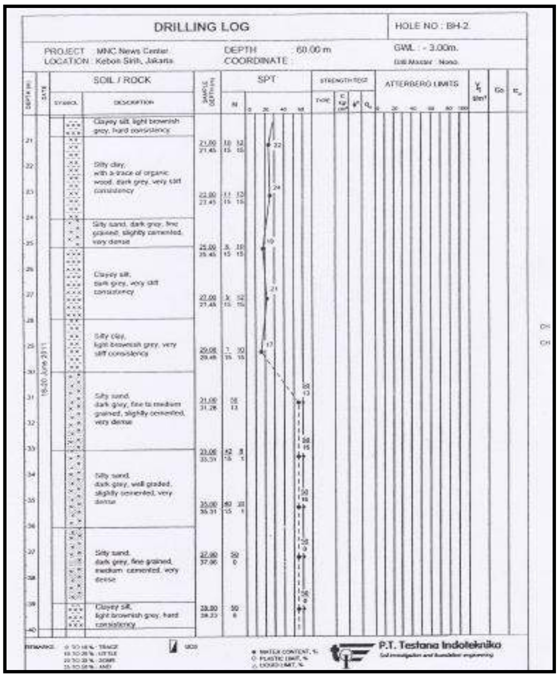

Fig. 2 Drilling Log and SPT from 20.00- 40.00m

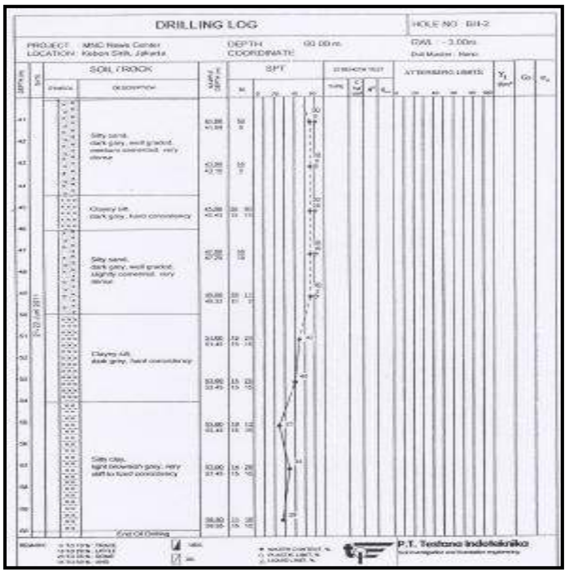

Fig. 3 Drilling Log and SPT from 40.00- 60.00m

3.2. Simplified Soil Mechanics Method Design

To calculate the capacity of pile , the Simplified Soil Mechanics method is used. Semi-empirical correlations have been extensively developed relating both shaft resistance and base resistance of bored piles to N-values from Standard Penetration Tests (SPT"N" values).This study use that correlation for computing bearing capacity of pile.

3.2.1. Ultimate shaft capacity

Undrained Shear Strength (Su), and adhesion factor α of cohesive soil are computed using empirical correlation of Nspt from Terzaghi dan Peck ,(1967), Sowers, (1979) and Kulhawy, (1991). See Fig. 4.and 5

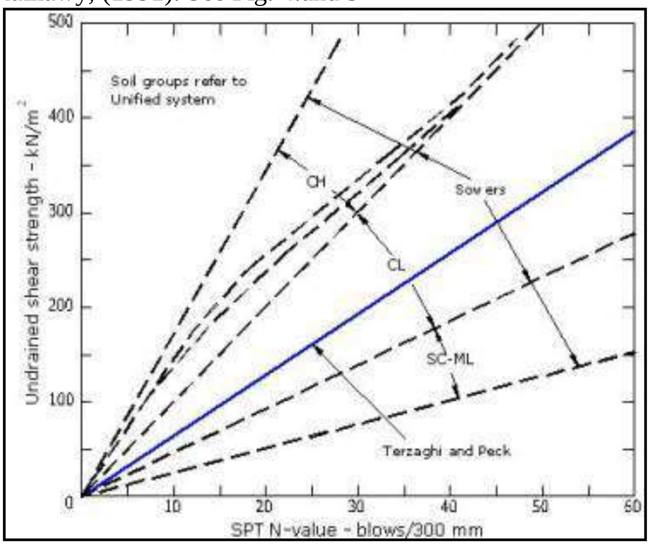

Fig. 4 Relationship between Undrained Shear Strength (Su) and N spt for cohesive soil
(Terzaghi dan Peck ,(1967), Sowers, (1979)

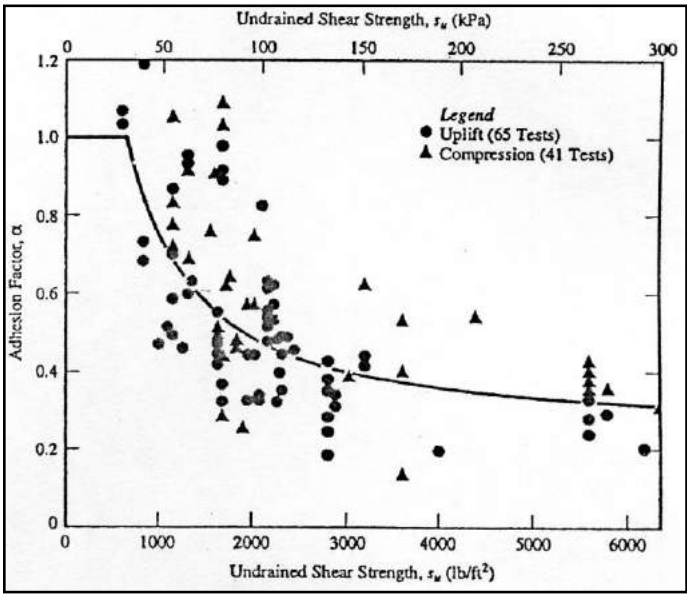

Fig. 5 Relationship between Undrained Shear Strength (Su) and Adhesion Factor for cohesive soil (Kulhawy, (1991)

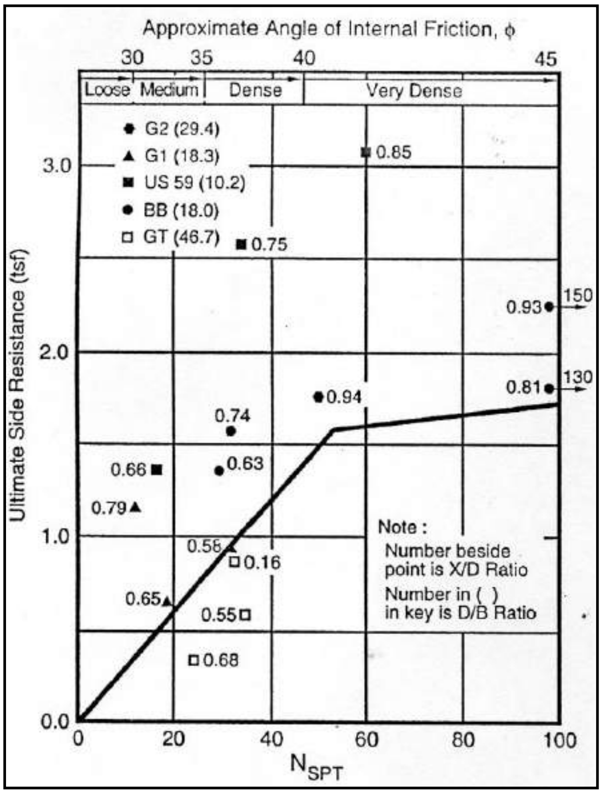

Fig. 6 Relationship between Ultimate Side Resistance and N spt for cohesionless soil (Wright, (1977))

For non cohesive soil, the ultimate side resistance are computed using Wright, (1977) chart . See Fig. 3.

The ultimate shaft capacity is computed using Qsu = Σi fsu * Ps

Where :

fsu = Unit shaft resistance for each layer of embedded soil

Ps = Perimeter of pile.

i = = Number of soil layers

The calculation is shown in table 1.

3.2.2. Ultimate base capacity.

Pile tip is at elevation of -39.00 m, in hard silty sand ,dark gray, medium and very dense cemented. The value N spt = 50/0, meaning the soil has been hit 50 times, and no decrease. Such soil is found begin -31.00 m , so, the depth is 8 m (10 times the diameter of the pile) on the pile tip. This is ensured that the hard layer can clamp the pile, so that the custody of pile tip can work well. Under the pile tip there is a hard clayey silt (N = 50/8) as deep as 1m, followed by silty hard sand (N = 50). So the pile tip is supported also by the hard ground. Ultimate side resistance in non-cohesive soil (qp) is calculated by using graphs of Reese & Wright, (1977). See Fig. 7.

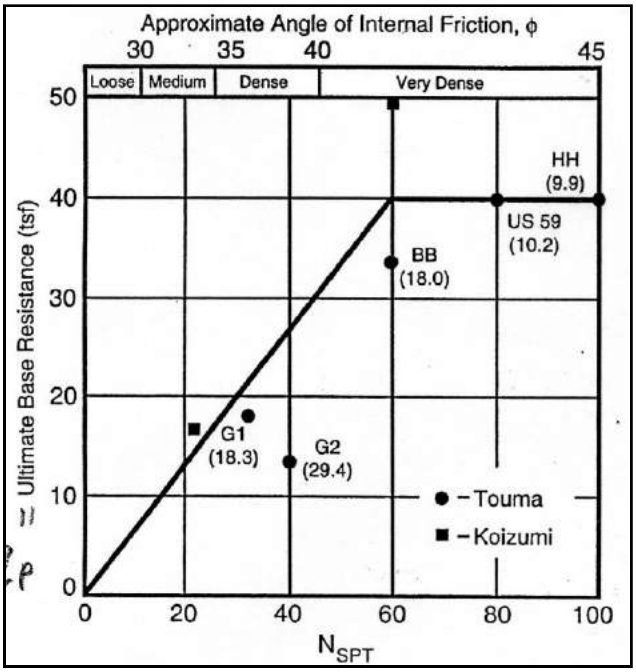

Fig. 7 Relationship between Ultimate Base Resistance and N spt for cohesionless soil (Reese & Wright, (1977))

The ultimate base capacity is computed using Qbu = Ap.qpu

Where :

qpu = Unit base resistance for the bearing layer of soil.

Ap = Area of pile.

The calculation is shown in Table II

TABLE II
ULTIMATE SHAFT RESISTANCE USING SIMPLIFIED SOIL MECHANICS METHOD

			Ultimate Shaft Resistance (Qs)						
			Adhesion factor			D = 0,8 m			Type of Soil
Depth	N spt	Su *)	α**)	fs		Pile perimeter	q s	Qs cum ult	
(m)		(kg/cm2)	(-)	(kg/cm2)	(t/m2)	(m)	(ton)	(ton)	
		(1)	(2)	(3)=(1)*(2)	(4)= (3)*10	(5)	(6)=(4)*(5)	(7)= cum (6)	
0	Fill Material								
-1	6	0,40	0,87	0,35	3,48	2,51	8,74	8,74	
-2	6	0,40	0,87	0,35	3,48	2,51	8,74	17,48	
-3	7	0,47	0,79	0,37	3,69	2,51	9,26	26,74	silty clay
-4	7	0,47	0,79	0,37	3,69	2,51	9,26	36,01	CL
-5	7	0,47	0,79	0,37	3,69	2,51	9,26	45,27	
-6	7	0,47	0,79	0,37	3,69	2,51	9,26	54,53	
-7	8	0,53	0,69	0,37	3,68	2,51	9,24	63,77	
-8	8	0,53	0,69	0,37	3,68	2,51	9,24	73,02	
-9	6	0,40	0,87	0,35	3,48	2,51	8,74	81,76	
-10	6	0,40	0,87	0,35	3,48	2,51	8,74	90,50	
-11	10	0,67	0,59	0,39	3,93	2,51	9,88	100,38	
-12	15	0,50	0,71	0,36	3,55	2,51	8,92	109,30	
-13	23	0,77	0,55	0,42	4,22	2,51	10,59	119,89	clayely silt
-14	37	1,22	0,44	0,54	5,35	2,51	13,45	133,34	ML
-15	50			1,50	15,00	2,51	37,68	171,02	
-16	50			1,50	15,00	2,51	37,68	208,70	silty sand
-17	50			1,50	15,00	2,51	37,68	246,38	SM
-18	44			1,32	13,20	2,51	33,16	279,54	
-19	28			0,84	8,40	2,51	21,10	300,64	
-20	25	0,83	0,53	0,44	4,42	2,51	11,09	311,73	clayely silt,ML
-21	22	1,47	0,4	0,59	5,87	2,51	14,74	326,47	
-22	23	1,53	0,4	0,61	6,13	2,51	15,41	341,87	silty clay
-23	24	1,60	0,39	0,62	6,24	2,51	15,67	357,55	CL
-24	22	1,43	0,41	0,59	5,88	2,51	14,76	372,31	
-25	19			0,57	5,70	2,51	14,32	386,63	silty sand,SM
-26	20	0,67	0,59	0,39	3,93	2,51	9,88	396,51	
-27	21	0,70	0,58	0,41	4,06	2,51	10,20	406,71	clayely silt
-28	19	0,63	0,61	0,39	3,86	2,51	9,70	416,41	ML
-29	17	1,13	0,45	0,51	5,10	2,51	12,81	429,23	silty clay,CL
-30	39			1,16	11,55	2,51	29,01	458,24	
-31	50			1,50	15,00	2,51	37,68	495,92	
-32	50			1,50	15,00	2,51	37,68	533,60	
-33	50			1,50	15,00	2,51	37,68	571,28	silty sand
-34	50			1,50	15,00	2,51	37,68	608,96	SM
-35	50			1,50	15,00	2,51	37,68	646,64	
-36	50			1,50	15,00	2,51	37,68	684,32	
-37	50			1,50	15,00	2,51	37,68	722,00	
-38	50			1,50	15,00	2,51	37,68	759,68	
-39	**50**			**1,50**	**15,00**	**2,51**	**37,68**	**797,36**	

TABLE III.
ULTIMATE BASE CAPACITY USING SIMPLIFIED SOIL MECHANICS METHOD

		Ultimate Base Capacity (Qp)		
		Non cohesive		
Depth	**N spt**	**Qp**	**Pile Area**	**Qp ult**
(m)		**(ton/m2)**	**(m2)**	**(ton)**
		(1)	**(2)**	**(3)=(1)*(2)**
-39	50	365,05	0,50	183,40

3.2.3. Ultimate bearing capacity.

So, the ultimate bearing capacity of pile is Qult = Qs + Qp = 797,36 + 183,40 = 980 ton.
The design working load of pile is half of ultimate bearing capacity that is 400 ton.

3.3. Kentledge Pile Loading Test.

3.3.1.Date of test.

The pile loading test performed by the system Kentledge D 1143-81 section 5.2. Pile is casted on October 18, 2011 and the test on 9 to 11 of January, 2012. So, between the casting and the test, there is time of 82 days. So the age of the concrete has reached more than 28 days. Predicted compressive strength is above 42 MPa, so that is strong enough to be burdened.
Within 82 days,it expected that the soil damaged caused by drilling and casting has been improved and the water table has returned to its original position. So that the loading test results did not change with time and has a long-term results.See Fig. 8.

3.3.2. Loading

The maximum consecutive loading are 200 tonnes (50%), 400 tons (100%), 600 tons (150%) and 800 tons (200% of design working load).In this test, the pile is loaded and unloaded in unequal increments.The load is maintained under each increment until the rate of settlement is acceptably small. At two times the design load, the load is maintained for 12 hours. After the required holding time, the loading is added in increment of 50%, 33%-25%, 50%-17%, 50%-13% with 1 hour between increments, in maximum load of 200 ton, 400 ton, 600 ton and 800 ton. As well as reducing load during unloading. The greater the maximum load, the smaller the load change. This is done to save time without sacrificing testing accuracy. Relationship between load and settlement can be seen in Table IV. and Fig 9.

TABLE IV
RELATIONSHIP BETWEEN LOAD AND SETTLEMENT IN MAXIMUM LOAD OF 200 TON, 400 TON, AND 800 TON (LOADING AND UNLOADING)

Design Working Load 400 ton							
Max 200 ton (50%)		Max 400 ton (100%)		Max 600 ton (150%)		Max 800 ton (200%)	
Load	Settlement	Load	Settlement	Load	Settlement	Load	Settlement
(ton)	(mm)	(ton)	(mm)	(ton)	(mm)	(ton)	(mm)
0	0,000	0	0,01	0	1,46	0	3,49
100	0,510	200	1,56	200	3,07	200	5,18
200	**1,300**	300	3,09	400	5,28	400	7,55
100	0,980	**400**	**4,87**	500	7,30	600	10,73
0	0,010	300	4,26	**600**	**10,12**	700	14,36
		200	3,75	500	9,84	**800**	**21,78**
		0	1,46	400	9,25	**800**	**24,48**
				200	7,01	600	23,85
				0	3,49	400	21,54
						200	18,79
						0	14,04

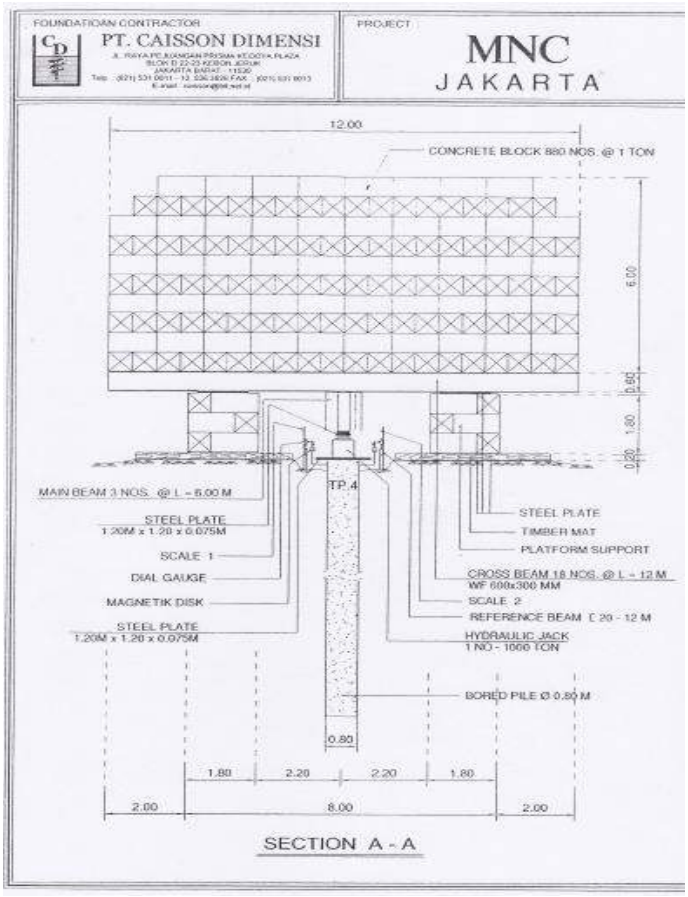

Fig. 8.Kentledge test

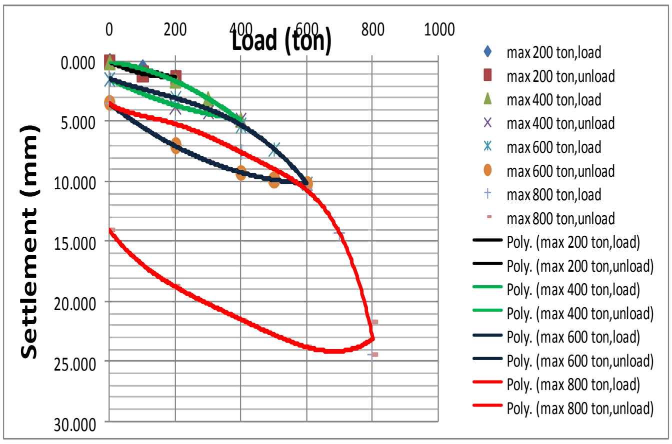

Fig. 9 Relationship between load and settlement in maximum load of 200 ton, 400 ton,600 ton and 800 ton (loading and unloading)

3.3.3. Calculation of ultimate bearing capacity Kentledge Pile Loading Test.

Calculation of bearing capacity using two methods, namely Mazurkiewicz and Davisson MT methods.Mazurkiewicz method produce the ultimate bearing capacity Q ult = 850 tons and Davisson MT method produces Q ult = 730 tons. The calculation of the value of x can be seen in Table IV. Graph both methods are shown in Fig. 10.and Fig. 11.

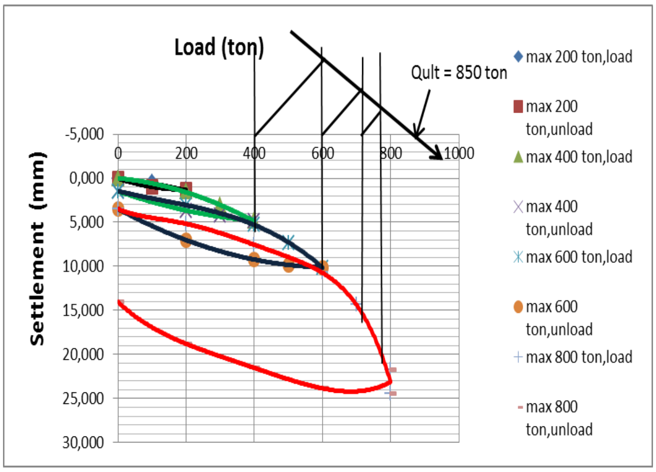

Fig. 10 Ultimate bearing capacity (Qult) of bored pile using Mazurkiewicz method

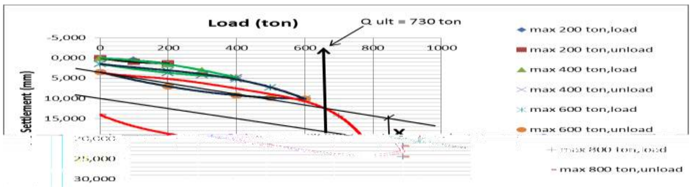

Fig. 11 Ultimate bearing capacity (Qult) of bored pile using Davisson MT method

TABEL 4
COMPUTATION OF X VALUE IN DAVISSON MT METHOD.

D (cm)	D (inch)	D/120 (inch)	X = 0,15 + D/120 (inch)	X = 0,15+ D/120 (mm)
80	31,496	0,262	0,412	10,477

4. Analysis and Conclusion.

4.1. Loading unloading with a maximum load of 200 tons.

The magnitude of decrease and increse of settlement in the loading test, it can be seen in Table III.
On the maximum load of 200 tons, the settlement that occurs when the maximum load of 200 tons, that is 1.30 mm is recovered when the load is reduced, so the pile return on the initial conditions. The settlement of last unloading is 0.01 mm = 0 mm. That's because the base resistance of the pile has not yet happened. So all of the burden borne by the pile (which retracts and extends back) and shaft resistance.
At first, pile fell by 1.30 mm, it cause the workings of friction between the soil and pile. Once the load is reduced, the shaft friction "raise" the pile back, so the magnitude of setting

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is 0 mm. Thus it can be said that based on this test, the shaft resistance that happened at that time is 200 tonnes.
The shaft resistance of pile happens if there is a movement of 0.5% to 1% pile diameter, while the base resistance occurs if there is a movement of 10% to 20% pile diameter. Pile diameter is 80 cm, shaft resistance of pile occurs if there is a movement of 4 mm to 8 mm, while the base resistance occurs if there is a movement of 8 cm to 16 cm.

4.2. Loading unloading with a maximum load of over 200 tons.

In the next load those are 400 tons, 600 tons and 800 tons, it is likely custody ends work. Settlement of the pile did not recover. The amount of setting occurred at 1.45 mm, 2.03 mm and 10.55 mm. While the recovered settlement of pile is equal to 3.41 mm, 6.63 mm and 10.44 mm.
At burdened loading - unloading, there was recover setting and unrecover setting. Recover setting can be said elastic deformation and unrecover setting is said plastic deformation. It turned out that an increase in the percentage ratio of plastic deformation to the elastic deformation increases very sharply when the load approached its ultimate. Of development, it can be concluded that the pile near the ultimate carrying capacity at the time weighed 800 tons (200% design working load) .See Table V and Fig. 12

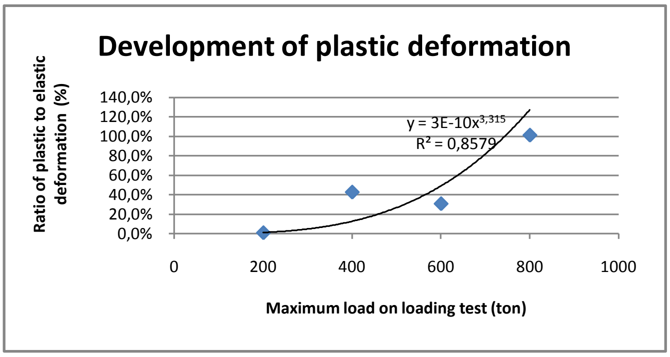

Fig. 12 The development of plastic deformation

TABLE V.
ELASTIC AND PLASTIC DEFORMATION DURING LOADING TEST.

No	Max Load	Settlement	Unrecover (Plastic deformation)	Recover (Elastic deformation)	Plastic Elastic deformation
	(ton)	(mm)	(mm)	(mm)	(%)
1	200	1,3	0,01	1,29	0,8%
2	400	4,87	1,45	3,41	42,5%
3	600	10,12	2,03	6,63	30,6%
4	800	24,48	10,55	10,44	101,1%

4.3. . The difference of ultimate bearing capacity between Kentledge loading test and Simplified Soil Mechanics method.

Calculation of bearing capacity of pile based on loading test is analysed using two methods, those are Mazurkiewicz and Davisson MT methods.Mazurkiewicz method produce the ultimate bearing capacity Q ult = 850 tons and Davisson MT method produces Q ult = 730 tons. While the calculations using the Simplified Soil Mechanics method using SPT data produces the ultimate bearing capacity = 998 tons. See Table VI. The difference between Kentledge loading test and Simplified Soil Mechanics method are 13 % and 25%, if the ultimate capacity of Kentledge loading test is computed using Mazurkiewicz and Davisson MT method.

TABEL VI
COMPARATION AMONG ULTIMATE BEARING CAPACITY OF PILE USING SIMPLIFIED SOIL MECHANICS METHOD , MAZURKIEWICZ DAN DAVISSON MT METHOD.

No	Method	Q ultimate (ton)
1	Simplified Soil Mechanics method & SPT	980
2	Mazurkiewicz	850
3	Davisson MT	730

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