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Axial Pile Capacity from CPT Data in Difficult Soil

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ABSTRACT
Dynamic pile load testing was performed during the driving of 610 mm displacement prestressed concrete piles into sandy soils, and actual pile capacity was determined during end-of-drive (EOID) and at beginning of restrike (BOR) using CAPWAP procedures. The load test provided an opportunity to compare pile design techniques to measured pile performance. The soils at this site prevent the pile driving process from being completed and the required pile length and capacity were not achieved due to early refusal. An evaluation was carried out to evaluate nine Cone Penetration Test (CPT) methods based on their ability, to which predictive method would be best suited for estimating the pile capacity at a site where such soils may encountered. The study also compared the CPT methods to the results of the bearing capacity obtained from Standard Penetration Test (SPT) based methods presented in the literature for the same pile. The ratio of predicted total capacity, Q_p, to measured total capacity, Q_m, is presented, along with the absolute percent difference between the predicted and measured capacities. Four methods included Philipponnat (1980), De Ruiter and Beringen (1979), Price & Wardle (1982), Zhou Etal (1982) had slightly over predicted the capacities for test pile within 50% to 63% of the capacities determined by the 1-day BOR dynamic loading test. The Q_p and Q_m ratio was between 1.5 to 1.9 which showed good agreement between predicted and measured capacities.

1. INTRODUCTION AND BACKGROUND

Historically, the prediction of pile capacity was based on the Standard Penetration Test (SPT). The piezocone provides an alternate field test to characterize the subsurface condition Almeida et al. (1996). The cone penetration test (CPT) is considered one of the most cost-effective and reliable method for soil classification. In 1917, the Swedish railways introduced the CPT. Ten years later, Danish railways started to use CPT. The first apparatus was simply a cone and a string of outer rods. In 1936, the Dutch Mantle cone was introduced. This cone has an area of 10 cm ^{2} and an apex angle of 60°, which is similar to ones in use. But the cone was pushed by hand and there was a limitation on the capacity and penetration depth. In addition, it could not penetrate very dense sand or cemented soils (Schmertmann, 1978).

The prediction of pile capacity is complicated by the large variety of soil types and installation procedures. In engineering practice, design and analysis of friction piles is carried out based on empirical formulas and depends to large extent on personal experience and judgment of the engineer. Because of many uncertainties associated with pile foundation analysis and design, full- scale pile load tests and dynamic load test are usually carried out at the site for important projects (Meyerhof, 1976).

Driving a pile has different effects on the soil surrounding on the relative density of the soil, loose soils and sand soil is compacted. In dense soil, any further compaction is small, and the soil is displaced up ward causing ground have. In loose soils, pile driving is preferable to boring since compaction increases the end bearing capacity. In non-cohesive soils, skin friction is low because a low friction around the pile. The presence and movement of ground water the processes of construction and sometimes the durability of piles in service, the pile rebounds in these soils generally tends to increase as driving progresses due to increased pore water pressure. The incompressible water in the soil forces the pile rebound to increase. The ultimate load must then be divided by a factor of safety depending on the maximum tolerable settlement (Jarushi et al., 2013&2015).

Certain soils exhibit large elastic behavior, practically at the pile toe in end bearing, causing unfavorable high rebound during pile driving. High rebound results when a pile/soil system that's highly compressed during a hammer blow springs back to near its original condition. This situation adversely affects pile drivability and complicates assessment of its load bearing capacity (Jarushi et al., 2013).

High rebound typically occurs when driving large displacement-type piles into saturated soils (e.g., dense silty sand, hard silty clay). The pile rebounds in these soils generally tends to increase as driving progresses due to increased pore water pressure. The incompressible water in the soil forces the pile rebound to increase (Jarushi et al., 2013&2015).

2. OBJECTIVES

The main objective of this research is to evaluate and compare prediction of axial pile capacity based on nine different theoretical approaches using CPT data with data from dynamic load test

3. SITE INVESTIGATION PROGRAM

The Anderson Street Overpass is located in downtown Orlando, Florida and is part of the I-4, SR 408 interchange the intersection of I-4 and SR-408 in Central Florida. Figure1 shows the location of the pier where the pile was installed and SPT borehole (AS-103). The site investigation program at this site included a large number of in-situ tests, including CPTu, Seismic CPT (SCPT), DMT, and SPT. The CPT results and soil stratigraphy are shown in Figure 2. The ground water table was located 3 m below the ground surface. The average unit weight of the soil is 18 KN/m³.

As the SPT test progresses, Split-barrel and Shelby tube samples were obtained in order to establish the soil profile. The soil samples were classified in accordance with Unified Soil Classification System (USCS). Sand was the predominate soil at this site consistently representing over 50 percent of the soil. The soil strata were classified as one of the following groups: SC, SM-SC, SM, CL, SP-SM, and SP-SC. These soils displayed an olive green to light green color with visual descriptions ranging from clayey and silty fine sands, to highly plastic clays with low permeability

Figure 1. Site and CPTu locations at the site of the test pile program (Jarushi, 2013)

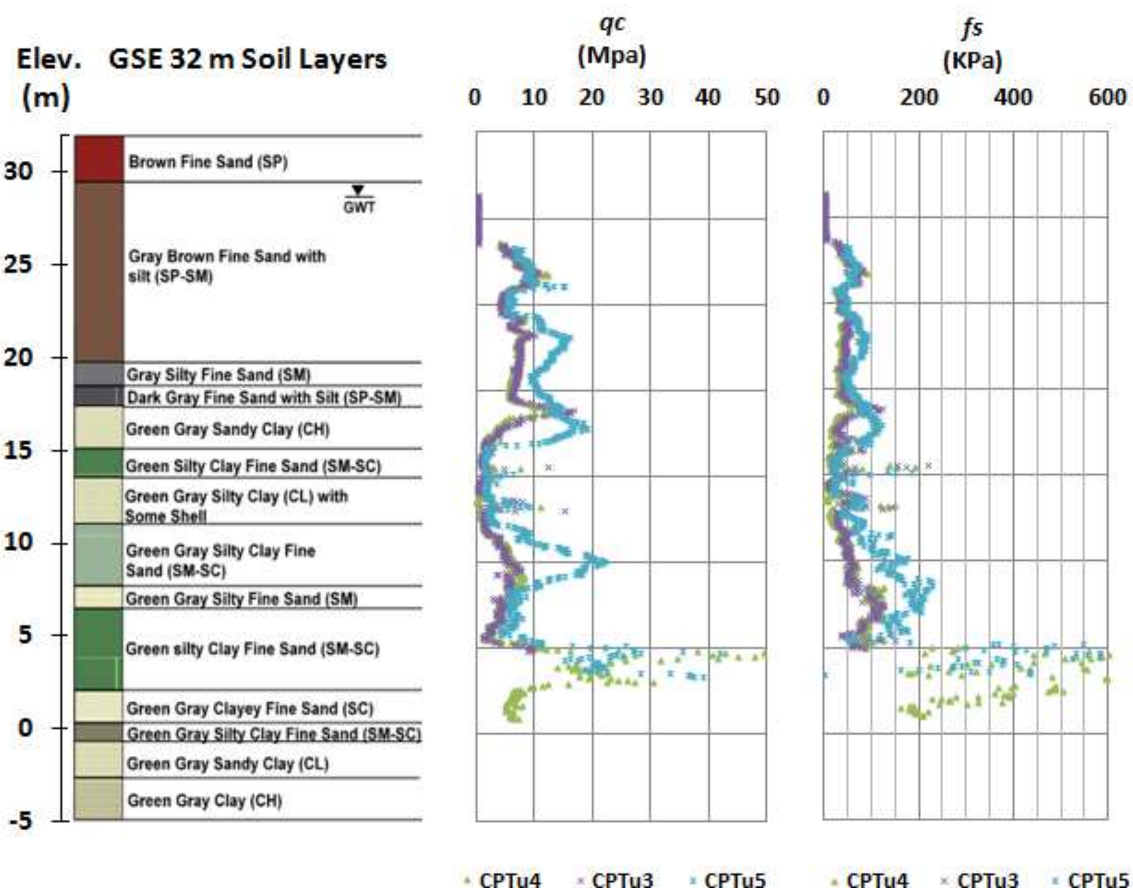


Figure 2. CPTu profile with USCS soil classification

4. PILES AND TESTING PROGRAM

The piles were 610 mm square prestressed concrete piles (PCPs). A Delmag D62 diesel hammer with a rated energy of 122 kJ, was used for driving. When the test pile was driven, plywood cushions of either 300 mm or 410 mm were used. However, when installing the remaining production piles, a 300 mm plywood cushion was used. At the end of initial driving, the contractors and engineers encountered problems with pile rebound at Pier 2 on the west end of the bridge, but after allowing the piles to "set-up," the required capacities were achieved. Severe driving problems occurred during installation of the displacement piles at Pier 6 located on the east end of the overpass, causing the foundations to be redesigned using low displacement steel H-piles (HP 14 x 89).

Pile 6 at pier 6 was driven as an instrumented test pile. Pile information is summarized in Table 1. In order to further investigate pile driveability, a set-check were performed one day after the initial drive was completed. A set-check typically consists of performing at least 10 hammer blows or 10 or more inches of driving, after the pile has set for at least 15 minutes after driving. Results of the loading tests and dynamic CAPWAP measured pile capacity are presented in Table 1. The total pile capacity obtained from the dynamic loading testing at EOD and BOR are presented in Table 1.

At EOD, based on dynamic measurements, pile had a predicted resistance of 1775 kN. Comparing the results from end of initial driving found that the capacity increased by a factor of about 1.9 (3323 kN) over the approximately 1-day wait period through side shear set-up.

Table 1. Summary of test pile

Pile Location and GSE	
Station No.	4013+22.27
Ground Elev.	+32 m
Hammer Information	
Type	Delmag D62
Rated Energy	122 kJ
Pile Information	
Size	610 mm square
Type	Pre-stressed Concrete
Required Length	38 m
Achieved Length	32 m
Pile Capacity Information	
End of Driving (EOD)	0 day
Shaft Resistance	525 kN
Toe Resistance	1250 kN
Total EOD Capacity	1775
Beginning of re-strike (BOR)	1 day
Shaft Resistance	2344 kN
Toe Resistance	978 kN
Total BOR Capacity	3223

1. PILE CAPACITY PREDICTION

The ultimate axial pile load capacity, Qu, is calculated as the summation of two components: end bearing resistance, Qt, and friction resistance, Qs. The end bearing resistance is calculated as the product of the unit end bearing stress, qt, and the pile end area, Ap while a friction resistance is calculate as the summation of the unit skin friction, fs, multiplied by the outer area of the pile shaft, Asi at every layer,i. Nine CPT based methods were used to evaluate the pile capacity. A brief description of each method used in the current investigation is presented in Table 2.

2. FINDINGS

The total pile capacity, obtained from the dynamic load test and the predicted total pile capacity by the various pile capacity prediction methods are presented in Table 3. The ratio of predicted total capacity, Qp, to measured total capacity, Qm, is also presented, along with the absolute percent difference between the predicted and measured capacities.

Each axial pile capacity predictive method was compared to the measured pile response during the dynamic load test EOD and one-day BOR capacities.

3. RELIABILITY OF THE PREDICTION METHODS

The comparison between measured and predicted ultimate resistances is shown in Figure 3. The ultimate shaft resistance measured during the dynamic load testing when compared to the predictive methods varied with the methods, depth and soil type.

Four which included Philipponnat (1980), De Ruiter and Beringen (19982), Price & Wardle (1982), Zhou et al. (1982) (see Figure 3) had slightly over predicted capacities for test pile within 50% to 63% of the capacities determined by the 1-day dynamic loading tests.

The Penpile method underestimated capacities within -70% of the capacities determined by the 1-day dynamic loading tests.

The Schmertmann (1978), Bustamante and Gianeselli (1982) methods excessively overestimated the ultimate bearing capacities within 200% of the BOR capacity.

Eslami and Fellenius (1997) appear to provide a more reasonable estimate of the ultimate capacities for these soils with estimation of 100%.

Jarushi et al. (2015) presented the same history where ten SPT bearing capacity based methods used to evaluate the pile. They found only three methods among the ten methods estimated the bearing capacity with good agreement with dynamic load test results (BOR). The best methods were Briaud & Tucker (1984), Decourt (1995) and Shioi & Fukui (1982) methods appear to provide a reasonable estimate of the ultimate bearing capacity with predicted/measured ratios around 2.0. Results of these methods are presented in Figure 4 and 5b.

Table 2. Methods for predicting pile bearing capacity used in this study

No	Author, Year	Unit base resistance (KN)	Unit shaft resistance (KN)	Remarks
1	Schmertmann (1978)	qt =(q_{c1}+q_{c2})/2	f=αc f_s	qc1=is the average of cone tip resistances from 0.7 to 4D below the pile tip . qc2=is the average of minimum cone tip resistances over a distance 8D above the pile ;αc= reduction factor; fs=seelve friction
2	De Ruiter and Beringen (1982)	qt=(qc1+qc2)/2 <15 MPa	fs= min [fsa, qc(side)/300 qc(side)/400 tension 120 Kpa]	qc1=is the average of cone tip resistances from 0.7 to 4D below the pile tip . qc2=is the average of minimum cone tip resistances over a distance 8D f_{sa} is the average sleeve friction within the calculated layer along the pile q_c (side) is the average cone tip resistance within the calculated layer along the pile.
3	Bustam ante and Gianeselli (1982)	qt=k_b *eq (tip)	The pile unit skin friction (*f*) in each soil layer is estimated from the equivalent cone tip resistance (*qeq*(*side*)) of the soil layer, soil type, pile type, and installation procedure in Ref number 6.	
4	Aoki and De Alencar (1975)	qt=qca (tip)/F_b	fs=qc(side) αs / F_s <120 KPa	qca=is the average cone tip resistance around the pile tip Fb and Fs are an empirical factor that depends on the pile type
5	Price and Wardle (1982)	qt = Kb qc (tip) <15 MPa	fs = Ks $f$sa	Kb=0.35 for driven piles, Ks=0.53 for driven piles q_{ca}(tip) is the average CPT tip resistance within 4D below and 8D above
6	Pilipponnat (1980)	qt=Kb qca (tip)	fs=(αs/Fs) qcs	q_{ca} (tip) is the average tip resistance of 3D below and 3D above the pile k_b and F_S are functions of soil type, αs= 1.25 for precast concrete driven piles.
7	Penpile (1999)	qt=0.125qc	fs=$f$sa/(1.5+0 .1$f$sa)	qc=is the average of three cone tip resistances close to the pile tip. f_{sa}: the average sleeve friction within the calculated layer along the pile
8	Zhou et al. (1982)	qt= α q_{ca}	fs= β f_{sa}	α= is the function of soil type; β=is the function of soil type and f_{sa} is average CPT sleeve friction along the calculated soil layer.
9	Eslami and Fellenius (1997)	qt=Ct*q_{Eg}	fs=Cs q_E	Ct =is the end bearing coefficient; q_{Eg}=is the geometric average of q_E over the depth of influence above and below the pile base ; Cs=is the shaft friction coefficient; qE= is the effective cone resistance=qt-u2.

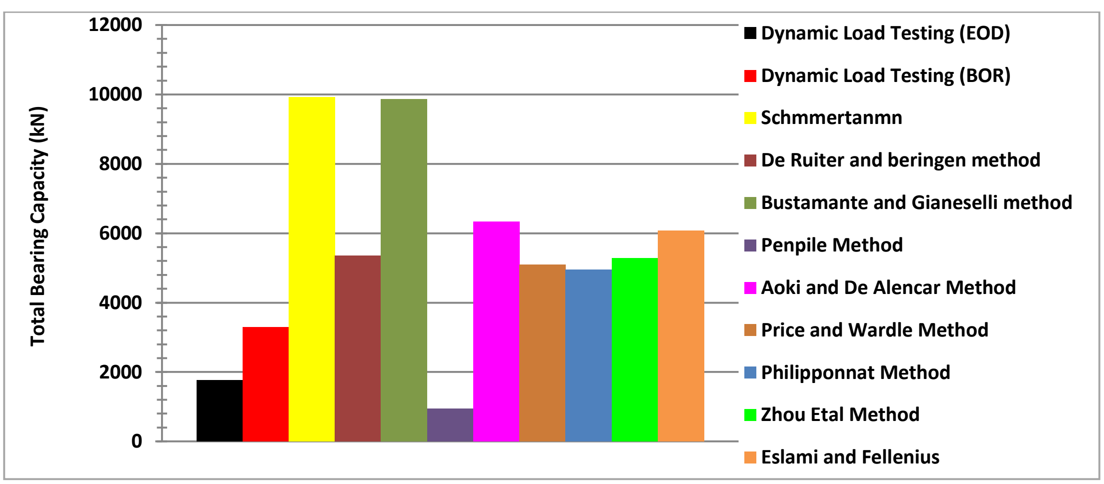


Figure 3. Measured and predicted total capacities using presented CPT methods

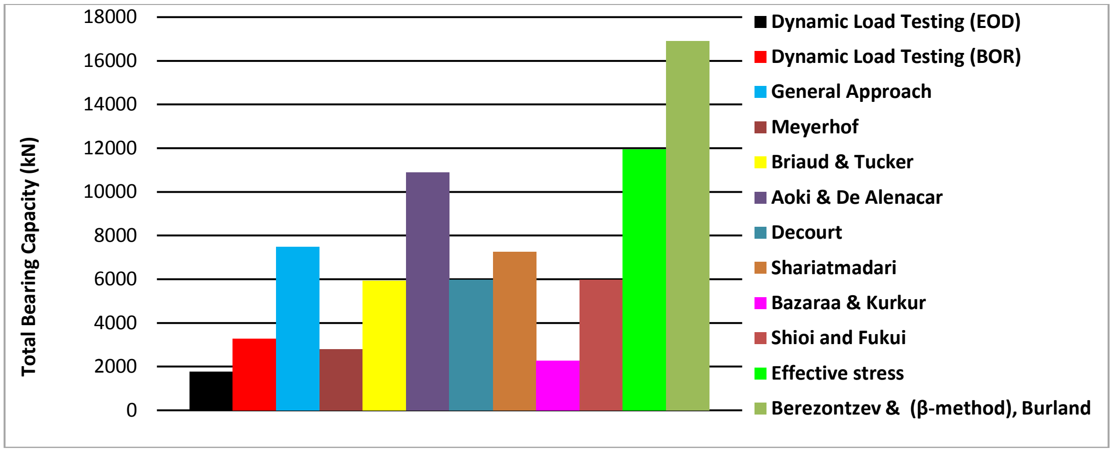


Figure 4. Measured and predicted total capacities using SPT methods (Jarushi et al. 2015)

4. PREDICTED CAPACITY/MEASURED RATIOS

Figure 5a shows the predicted/measured ratios (Qp/Qm) for the ultimate resistance from CPT methods. The ratio was between 0.29 which is underestimating the capacity and 3.55 which is excessively over estimated the capacity. None of the CPT based methods predicted within 1 to 20% of the measured capacity. The Philipponnat (1980) method had predicted capacities by 50% (4955 kN) which is provided very good agreement within 50%, of the results of the 1-day dynamic load test. The Qp/Qm ratio was 1.49. The ratio would be less if the test pile was after few days rather than 1-day restrike.

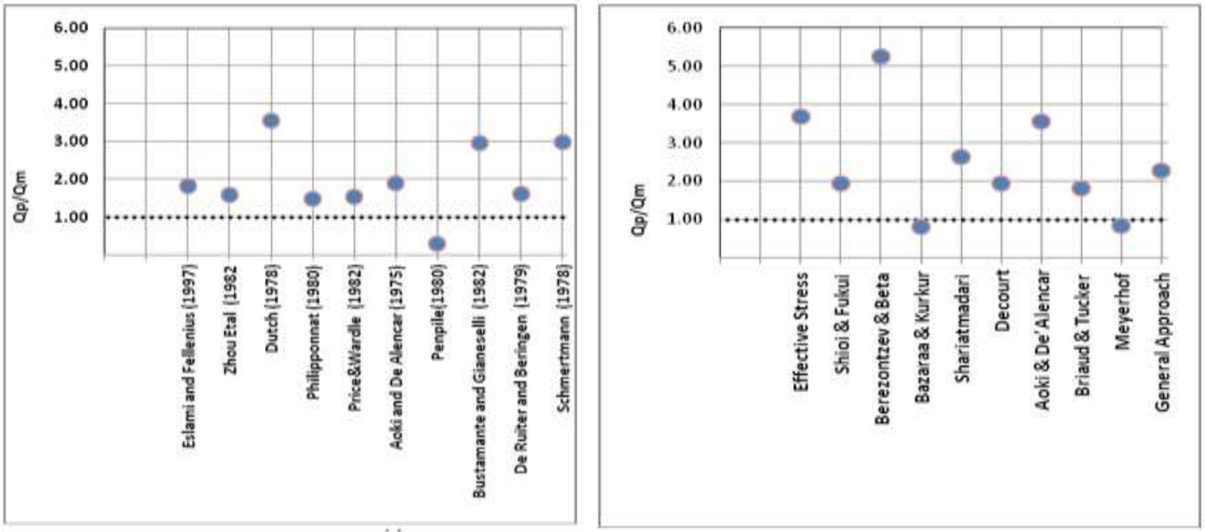
The CPT based methods tended to excessively over-predict the EOID pile capacity; however, none of the methods was found to provide an excellent prediction of the end of initial drive capacity. Figure 5b shows the Qp/Qm of the SPT methods.

The other three methods (De Ruiter and Beringen, 1982, Price & Wardle, 1982, Zhou Etal, 1982) had also provided a good estimation of the capacities determined by the 1-day dynamic loading tests. The Penpile method would result in a reduced ultimate resistance predicted value by ratio of Qp/Qm of 0.29. Figure 5a shows that Schmertmann, 1978, Bustamante and Gianeselli (1982) methods excessively overestimated the ultimate bearing capacities. It can note that the ratio was between 0.80 which underestimated the capacity and to 5.3 which is overestimated the capacity by factor of 5.

It was found at this site that direct CPT based methods predicted more accurately than SPT based methods, and provided very good agreement with dynamic load testing.

Table 3. Comparison between dynamic loading test results and presented capacity predictions

Author, Year	**Tip Resistance (kN)**	**Shaft Resistance (kN)**	**Total Qu (kN)**	**Absolute Differences (%)**	**Qp/Qm**
Schmertmann (1978)	4318	5605	9923	201	3.00
De Ruiter and Beringen (1982)	4318	1046	5364	63	1.61
Bustam ante and Gianeselli (1982)	875	8985	9860	200	2.97
Aoki and De Alencar (1975)	2467	3872	6339	93	1.91
Price and Wardle (1982)	1511	3583	5094	55	1.53
Pilipponnat (1980)	896	4059	4955	50	1.49
Penpile (1999)	339	615	954	-71	0.29
Zhou Etal (1982)	870	4416	5286	61	1.59
Eslami and Fellenius (1997)	1620	4456	6076	85	1.83
Dynamic Load Testing (EOD)	524	1249.8	1775	-	-
Dynamic Load Testing (BOR)	978	2313	3292	-	1.00



(a) (b)

Figure 5. Predicted/measured ratio recorded one day after driving using (a) the presented CPT methods (b) using SPT methods (Jarushi et al.2015)

5. CONCLUSIONS

Piezocone tests provide a considerable amount of information to characterize the geotechnical properties at a particular site needed for pile capacity. The time of the measured pile capacity from end of initial driving is an important consideration when attempting to predict pile capacity in high set-up soils, such as high pore pressure developing during driving of pile. An evaluation was carried out to evaluate nine CPT and CPTu methods based on their ability to which predictive method would be best suited for estimating the pile capacity at a site where refusal stage may encounter before required pile length achieved. The study also compared these methods to the results of the bearing capacity obtained from SPT based methods presented from a case history at the same pile.

The following methods were used in this evaluation: Schmertmann (1978), De Ruiter and Beringen (1982), Bustamante and Gianeselli (1982), Aoki and DeAlencar (1975), Price and Wardle (1982), Philipponnat (1980), Penpile (1999), Zhou Eetal (1982), Eslami and Fellenius (1997). All four methods (Philipponnat, De Ruiter and Beringen, Price & Wardle, Zhou Etal) had slightly over predicted capacities for test pile within 50% to 63% of the capacities determined by the 1-day dynamic loading test.

These methods provided satisfactory estimations. The Qp and Qm ratio was between 1.5 to 1.9 which showed good agreement between predicted and measured capacities.

Schmertmann (1978), Bustamante and Gianeselli (1982) methods over predict the ultimate bearing capacity by factor of 3 of the BOR pile capacity. It was found at this site that direct CPT based methods predicted more accurately than SPT based methods, and provided very good agreement with dynamic load testing.

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