Technical Paper

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PDA ASSISTED DYNAMIC LOAD TESTS PROVIDE A COST-EFFECTIVE FOUNDATION SOLUTION – A CASE HISTORY

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ABSTRACT: This paper presents a case history of design, prediction, probe pile driving, static and dynamic load tests, and installation monitoring of production piles for a 12-story beach-front hotel and convention center located in southwest Florida, USA. The subsoil conditions were explored by performing Standard Penetration Test (SPT) borings and Piezcone Penetration Test (PCPT) soundings to depths of 15 to 30 m. (50 to 100 ft) throughout the site. Deep foundation alternatives included pressure–injected footings, auger-cast, and driven piles. Following an evaluation of high structural loads, site-specific constraints, and costlier foundation alternatives, a prestressed concrete driven pile support system was selected as the most cost-effective foundation approach. The building foundations were designed for a revised pile capacity of 705 kN (80 tons) in compression and 310 kN (35 tons) in tension. As part of the design process, a probe pile-driving program was carried out to evaluate the pile driving equipment to determine and verify pile capacities, and to optimize the production pile lengths. In December 1994, a total of six 356 mm (14 in) square prestressed concrete probe piles were driven and two of them were static load tested. In mid 1997, when the project was modified and revised, a total of three 356 mm (14-in) square prestressed concrete piles were driven and dynamically load tested utilizing the proven new technology of Pile Driving Analyzer (PDA). Additionally, restrike of all the six original piles was also performed. Utilization of PDA during probe pile driving program helped in predicting pile freeze (i.e., increase in pile resistance with time) and thus optimized the production pile lengths. The predicted pile capacities were revised upward by 25 percent and resulted in cost savings for the owner. This information was effectively utilized as part of the overall quality control program to install a total of 430 prestressed concrete piles within the building footprint to depths ranging from 16.7 to 18.6 m (55 to 61 ft) below the pile cut-off elevation as part of the foundation support system. Prior to installation of probe or production piles, a termination criterion was established using 1-dimensional wave equation analyses in addition to the design pile embedment requirement.

This engineered, monitored, and tested foundation solution provided a case history where proven new technology has resulted in engineering a solution in a cost-effective manner.

RÉSUMÉ: Ce document est un dossier d'un projet, prévision, essai dynamique et statique de pilotage et le côut de l'installation des pieux pour un hôtel de douze (12) étages, qui se trouve devant la plage, et center convention, localisé dans le sud-ouest de la Florida, USA. Les conditions du sous-sol ont été explorées partout l'emplacement, en faisant l'épreuve Pénétration Standarde Teste (SPT) [des sondages] et aussi l'épreuve Piezcone Pénétration Teste (PCPT) résonnements à profondeurs de 15 à 30 m. (50 - 100 ft). Les alternatives pour la fondation profonde inclus pression-injecté des conditions, tarière-jet, et d'enforcement des pieux par pilotage. Après une évaluation de les hautes charges structurelles, contraintes emplacement spécifique et des alternatives plus chères pour la fondation, un système de béton prescontraint d'enforcement du pieu a été choisi comme la fondation le plus coût-efficace. Les fondations du bâtiment ont été conçues pour une pieu, dout la capacitée a été révisée de 705 [kN] (80 tonnes) de compression et 310 [kN] (35 tonnes) de tension. Comme partie du dévellopement du projet, un programme pilotage des pieux a été fait pour evaluer l'equipement du pilotage verifier les capacitées des pieux et optimiser la production des longueurs des pieux. Pendant le mois de décembre 1994 un total de six 356 [mm] (14-inch) et de béton précontraint étaient enforcer. Deux d'entre les six étaient éprouvé/testé de change statique. Mi 1997, quand le projet a été modifié et révisé un total de trios pieux, modifié béton précontraint, et de 356 [mm] (14-inch) carré ont été conduits et chargé du dynamique. Ils ont été mis a éprouves, en utilisant la nouvelle téchnologie (deja bien éprouver) du pieu Analyze (PDA). En outre, restrike de tous les six pieux originaux ont été exécutés aussi. Exploration du PDA pendant le programme pilotis a aid é a prédire le pieu gèle (cad, augmentation du resistance des pieux avec temps) et donc, optimize la production des longueurs des pieux. Les capacitées ides pieux prédit a été revu d'un montant de 25 pour cent et a eu comme resultat des économies des coûts pour le propriétaire. Cette information a été utilisée efficacement comme partie de la qualité total commande du programme pour installer dans l'empreinte de pas du bâtiment, un total de 430 pieux, béton précontraint, a profondeurs de 16.7 - 18,6 m (55 - 61 pieds) en dessous l'élévation coupe-éteinte du pieu comme partie de la systeme supporte de la fondation. Antérieur à installation d'enquête ou production des pilotis/pieux un critère de la terminaison a été établi utilisant 1 (un) dimensional analyses de l'équation de la vague dans l'addition de l'exigence du l'enterrement du projet pieu.

Celui-ci a construit, a dirigé, et a testé la solution de la fondation a fourni une histoire du cas prouve où la nouvelle technologie a eu pour résultat de génie une solution dans une manière coût-efficace.

1. INTRODUCTION

The Diamondhead Convention center is bounded by the Gulf of Mexico on the west, Estero Boulevard on the east, Palm Avenue to the north, and is located in Fort Myers Beach, Lee County, Florida, USA. The project consists of a 12-story tower with approximately 120 units and serves as a beachfront hotel and convention center. The building is concrete and masonry construction with post-tensioned floor slabs supported by reinforced concrete columns and shear walls. The geotechnical design for the original 16-story tower and an adjoining garage parkade was completed in late 1993. The building was later scaled down to a 12-story structure. Construction commenced in July 1997, and the building was released for occupancy in August 1998. The project building layout and boring location plan are illustrated in Figure 1.

In September 1994, some probe piles were driven, and two static load tests (compression and tension) were performed by another consultant. The scope of the value engineering services provided by ASC for the project included: (i) performing a complete geotechnical exploration, testing, and engineering evaluation program; (ii) dynamically load testing probe piles utilizing PDA; (iii)) assisting the owner in the proper selection of an economical, effective, and satisfactory foundation system; and, (iv) providing pile installation logging and quality control inspection/testing services during the foundation construction phase of the overall construction.

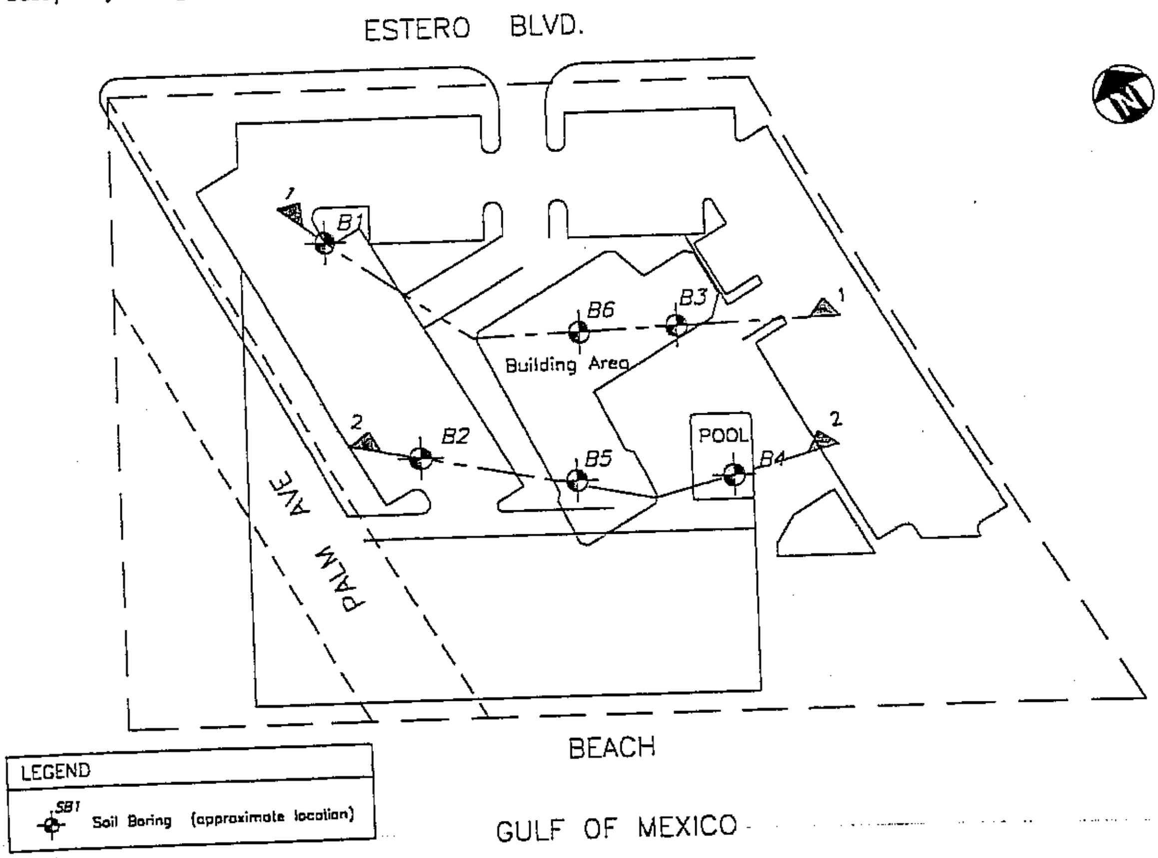


Figure 1. Project Layout and Boring Location Plan

2. SUBSURFACE CONDITIONS AND PROJECT DATA

Pre-construction project site features included a beachfront tract with generally level topography. A detailed geotechnical exploration program was undertaken, consisting of Standard Penetration Test (SPT) borings. The work was carried out using conventional drilling and sampling techniques. Subsurface soil conditions observed from 6 test borings ranging from 16 to 30 m (50 to 100 ft) below the existing ground surface are illustrated in figures 2 and 3.

In general, project site stratigraphy consisted of loose to medium dense, poorly graded sands to sand-shell mixtures and silty sands to approximately 7.0 to 8.8 m (23 to 29 ft). This section is underlain by medium dense to very loose sands, with weathered and fractured limestone to depths of 13.1 to 14.3 m (43 to 47 ft). Silty sands were encountered to 22.3 m (73 ft) and stiff to very stiff silty marine clays and clayey silts extended, thereafter, to the depth of termination of 30.5 m (100 ft).

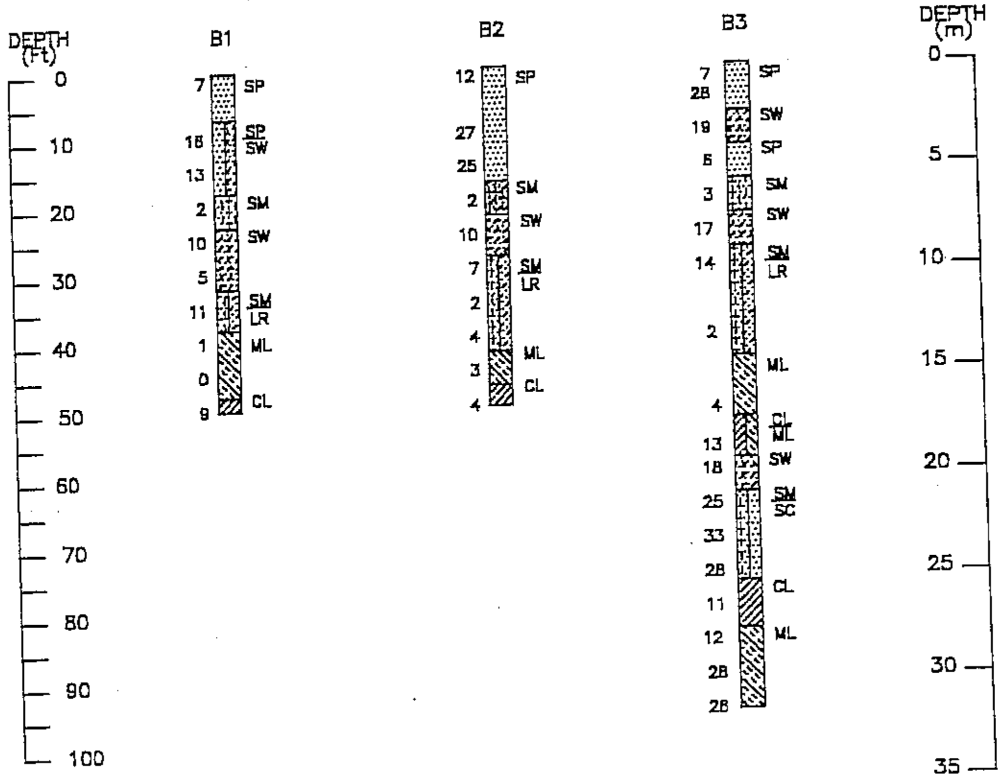


Figure 2. Generalized Subsurface Profile—Section 1-1

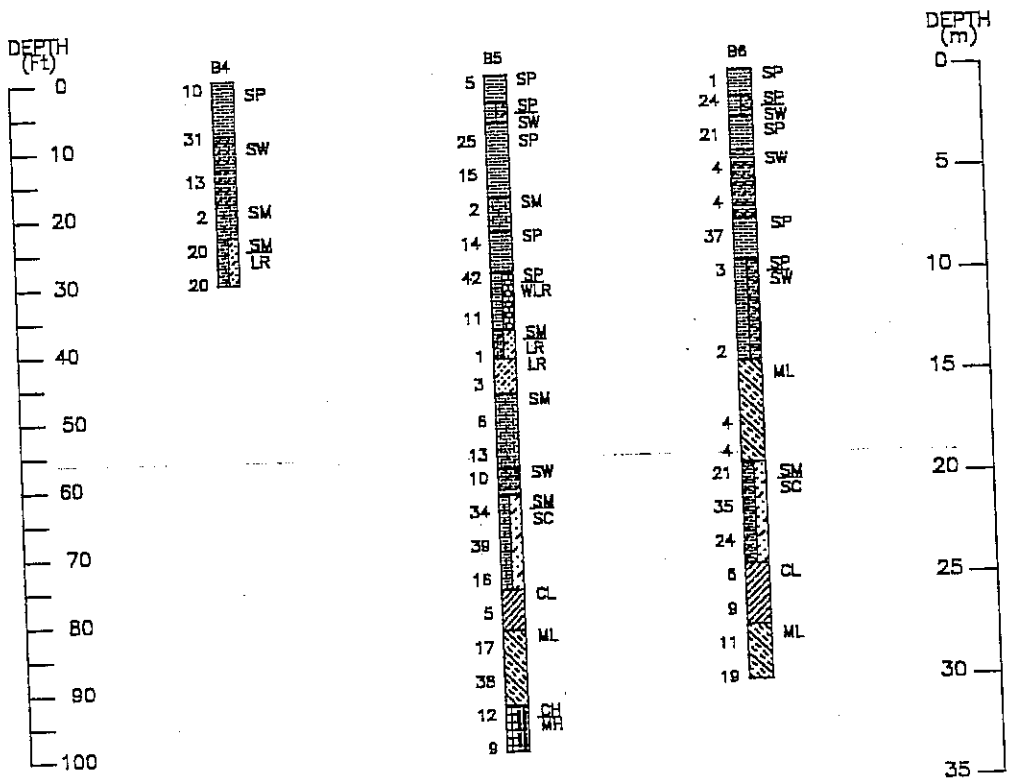


Figure 3. Generalized Subsurface Profile—Section 2-2

3. FOUNDATION DESIGN AND ALTERNATIVES

The first step of the design process was to evaluate various foundation systems and associated costs. Several foundation alternatives were considered for the proposed tower structure including: (i) driven precast prestressed concrete (PPC) displacement piling (straight and taper); (ii) cast-in-place auger piles; (iii) monotube steel pipe with tapered fluted sections. In view of the owner's accelerated construction schedule decision not to seek any more revisions in project foundation drawings, a driven prestressed concrete piling system was finally selected for the tower. Piles were designed to penetrate through the weathered limestone layer as composite action piles.

The allowable capacity of piles was estimated using SPT91 software. This program, which was developed and used by the Florida Department of Transportation, estimates the axial capacity of a pile based on SPT "N" values. The design method used in SPT91 has been found to be very reliable for driven piles in predominantly cohesionless soils (McVay, et al., 1989). Results indicated that prestressed concrete piles driven into silty sand, clayey sand, weathered limestone and sandy lean marine clays to depths ranging from 18 to 20 m (50 to 75 ft) below the original ground surface would provide a satisfactory and economical foundation alternative.

4. PROBE PILE AND STATIC LOADING TEST PROGRAM

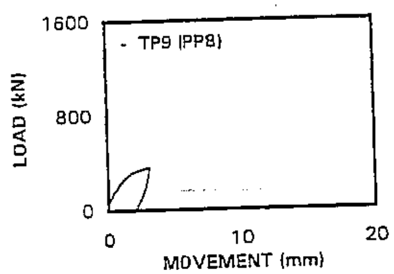
In an effort to obtain penetration and capacity data for piles for the old layout, a static loading test program was performed by another consultant in 1994. A total of five 356 mm (14 in.) square prestressed precast concrete piles were driven within the tower area. Driving was accomplished with an ICE 640 double acting diesel hammer with a maximum rated energy of 5,300 m-kg (40,000 ft-lbs). For a design capacity of 580 kN (65 tons) a guideline pile driving termination criterion using a 1-dimensional wave equation of 2 blows/cm (65 blows per ft), for the final, 61 cm (2 ft) was used. Two static load tests (1 in compression and 1 in tension) were performed for the originally required allowable design capacities of 580- kN (65 tons) in compression and 180 kN (20 tons) in tension. The procedures described in ASTM D 1143 and ASTM D 3689 for loading tests on piles in compression and tension were used. Figure 4 illustrates the load-movement curves attained for the loading tests as well as the predicted and allowable capacities from SPT 91 and loading test.

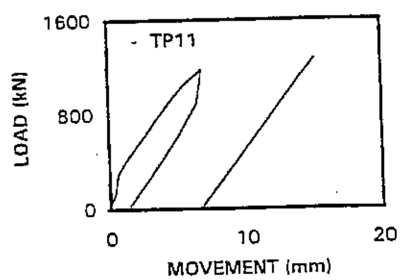
5. PROBE PILE AND DYNAMIC LOAD TESTING (DLT) PROGRAM

As an integral part of the value engineering process and in an effort to optimize penetration and capacity requirements for piles, a DLT program utilizing PDA instrumentation was recommended and performed.

5.1 Dynamic Load Testing

Three 19.8 m (65 ft) long and 365 mm (14 in) square probe piles were dynamically load tested during the probe pile driving and testing program within the revised tower footprint. Additionally, five probe piles installed previously in December 1994, without the use of PDA, were instrumented and retapped. The purpose of the probe pile program was to evaluate the suitability of the contractor's pile driving equipment, determine and verify pile capacities, provide production pile lengths, and establish driving criteria for use during the production pile driving phase of the project.





LOADING TEST NO.	PILE		CAPACITY[2]	
	WIDTH (mm)[1]	LENGTH (mm)[1]	PREDICTED (kN)[1][2]	DAVISSON (kN)[1][3]
TP9 (PP8)	356	21.3	364	---
TP11	356	21.3	1,182	---

NOTES:
(1) 1 mm = 0.04 in.; 1m = 3.28 ft; 1 kN = 0.11 tons
(2) Maximum load applied during static load test
(3) Not attained

Figure 4. Load Movement Curves from Loading Tests and Summary of Test Results

The PDA was utilized to better evaluate and predict the increase in pile resistance with time (i.e., set-up or freeze). This benefit was incorporated in the determination of production pile lengths and increase in capacity.

Two probe piles were instrumented after 2.1 m (7 ft) of driving while one pile was instrumented to 9.1m (30 ft), and then first restrike was performed after one day and subsequently instrumented and driven to 18.6 m (61 ft). Additionally, restrikes of the five probe piles, installed without PDA in December 1994, were instrumented in 1997. The next day three piles were retapped either the first or second time. The same ICE 640 hammer was used in 1997, and pile embedment depths ranged from 18.6 to 19.2 m (61 to 63 ft).

Dynamic load tests on probe piles confirmed that the subsoil conditions at the project site were generally uniform, thus making the site ideally suited for dynamic load testing utilizing a PDA. Because of it being relatively inexpensive and quick, many tests were performed to provide an interpretation of a larger portion of the site. Alternately, 8 static-load tests would have been required to obtain the quality and quantity of useful information as obtained during dynamic load tests utilizing the PDA.

Dynamic test date obtained in the field was further analyzed according to CAse Pile Wave Analysis Program (CAPWAP) for a more comprehensive understanding of the soil and pile behavior during pile driving (McVay, et al., 1989). CAPWAP analyses were performed on a total number of blows collected during restrikes of probe piles. These analyses provided a better evaluation of total ultimate pile capacity. It is a signal-matching process where a measured signal is matched with a simulated signal. This step provides a refinement of the pile capacity estimated in the field during driving. Additionally, CAPWAP provided a distribution of soil resistance along the embedded piled depth. The distribution of resistance along the pile depth enabled an estimate of resistance at other depths above the pile tip. Results from the CAPWAP analyses generally include static pile capacity, soil resistance distribution along pile shaft and under toe, soil damping and quake values, and forces along pile lengths at ultimate resistance.

PDA predicted ultimate pile capacity ranged from 1335 to 1600 kN (150 to 180 tons) at EOID and from 1500 to 1920 kN (170 to 215 tons) at the beginning of restrike (BOR). For pile PP3 at a depth of 9.1 m (30 ft) there was no documented increase in ultimate capacity at either EOID or BOR. Restrike was performed the next day and it was driven and instrumented to the full penetration depth of 18.0 m (60 ft). Time elapsed between BOR and EOID was one day. For the other 5 piles initially driven to the instrumented full length for a load tested ultimate capacity of more that 1155 kN (65 tons), the BOR capacity ranged from 1540 to 2490 kN (175 to 380 tons) for a time lapse of 900 days. Piles are this site exhibited no freeze increase for the 9.1 m (30 ft) long pile (Saxena, et al., 1998). However, substantial increase in pile resistance (set-up) ranging from 15 to 115 percent was noted in the other 7 piles, which were driven to the full embedment of 18.2 to 19.1 m (59 to 62 ft). The pile set-up is expressed as a ratio of pile capacity increase over initial pile capacity in percent. A summary of PDA test results is tabulated in Table 1, with the bar graph of pile capacity increase illustrated in Figure 5.

PROBE PILE NO.	LENGTH OF PENETRATION (m)	PDA CAPACITY AT EOID "a" [1] (kN)	PDA CAPACITY AT BOR "b" [2] (kN)	TIME ELAPSED BETWEEN EOID AND BOR (days)	INCREASE [(b-a)/a]*100 (%)
PP1	18.2	1335	1500	1	13
PP2	18.2	1600	1920	1	20
PP3[3]	9.1[3](18.0)	690 (1300)	665	1	-3
PP4	19.1	> 1155 [4]	2355	900	104
PP5	18.8	> 1155 [4]	1930	900	67
PP6	18.5	> 1155 [4]	2490	900	116
PP7	19.1	> 1155 [4]	2385	900	106
PP8	19.1	> 1155 [4]	1540	900	34

NOTES:
(1) EOID = end of initial driving
(2) BOR = beginning of restrike
(3) Pile penetration of 9.1 m (30 ft)
(4) Based on static load test I 5 days after initial driving in 1994)

Table 1. Pile Freeze or Set-Up

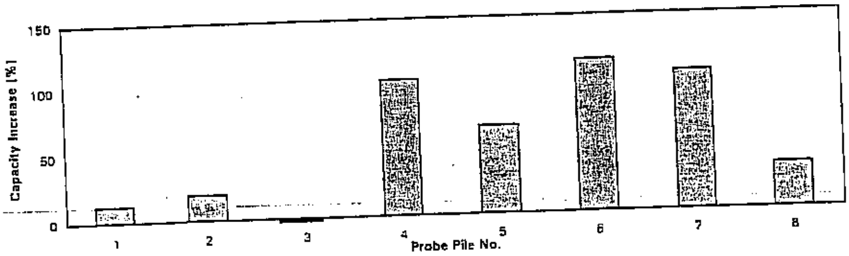


Figure 5. Bar Graph of Pile Capacity Increase

McVay, M.C., Townsend, F.C., Bloomquist, D. G., O'Brien, M., and Caliendo, J. A., (1989), *Numerical Analysis of Vertically Loaded Pile Groups*, Proceedings, Foundation Engineering, Current Principals and Practices, Vol. 1, Ed. F.H. Kulhawy, Evanston, Illinois, ASCE, 25-29 June 1989, pp. 675-690.

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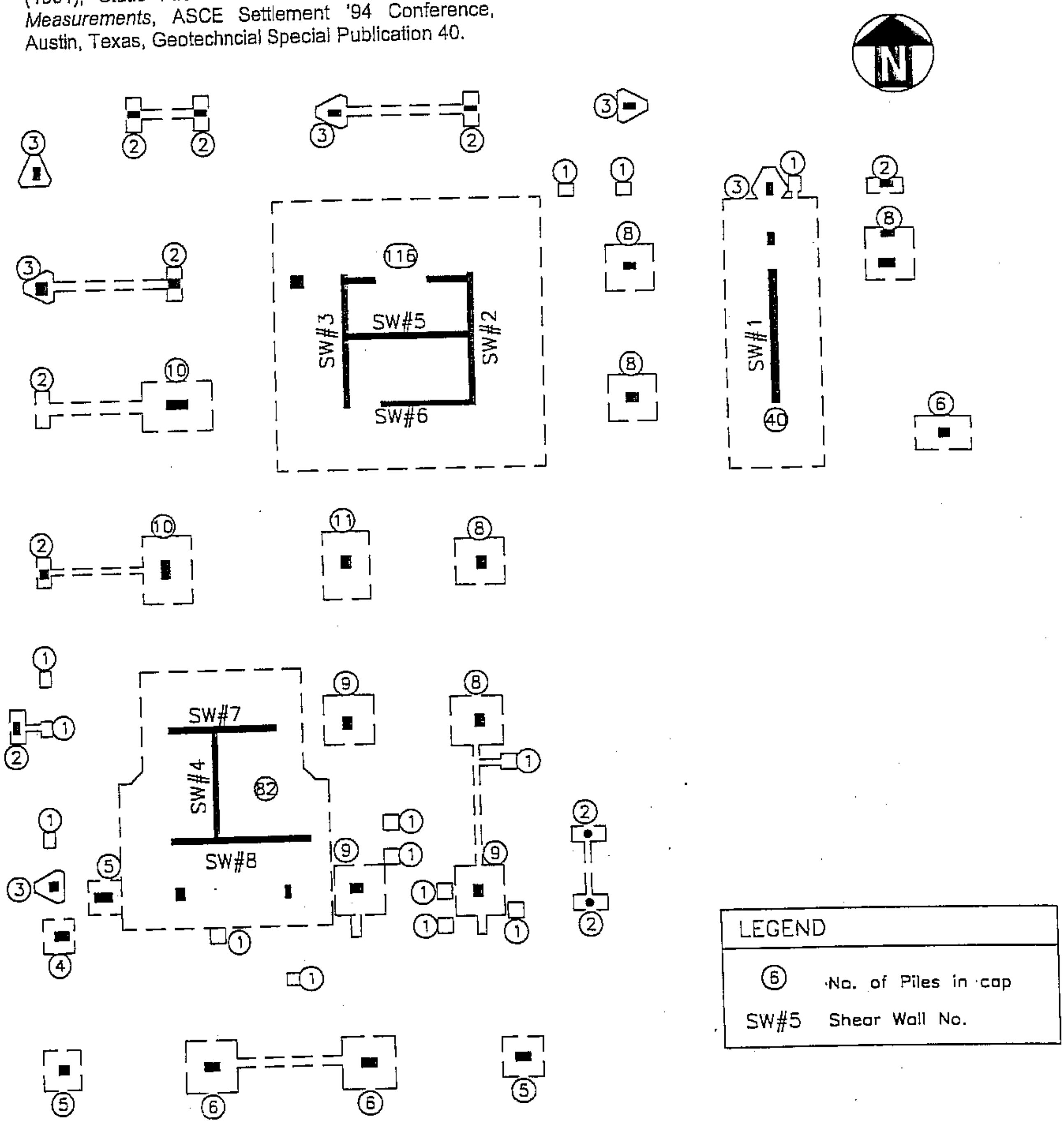

Figure 6. Pile Layout Plan and Loading Test Locations