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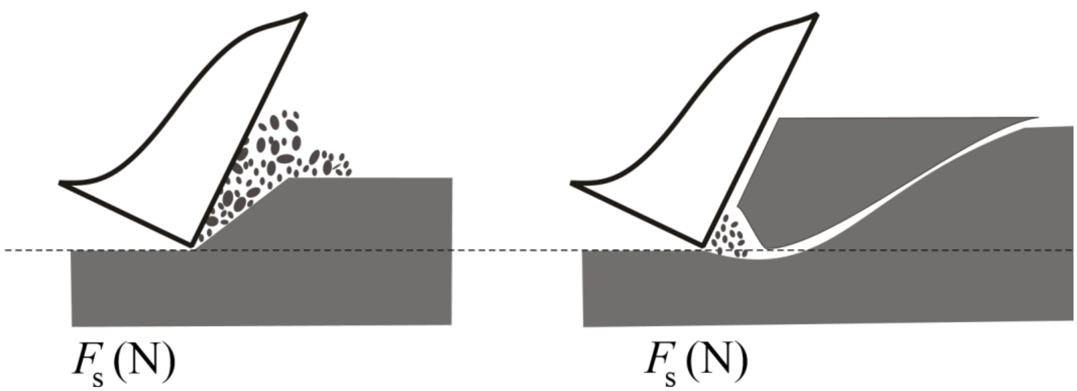

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Dynamic Cone Penetrometer and Relative Density Relationships for Uniformly-Graded Sands

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ABSTRACT: The dynamic cone penetrometer (DCP) has proven to be a useful tool in ascertaining relative density, assessing compaction quality, and evaluating variability of natural poorly-graded sand deposits and engineered fills of the same soil type. This paper presents several case studies where DCP test measurements are compared against SPT and/or nuclear density tests. DCP blow count to relative density and relative compaction relationships for sands are presented.

1 INTRODUCTION

The dynamic cone penetrometer (DCP) is a portable device used to give an indication of the relative density or compaction level for use in foundation (Sowers & Hedges 1966), agricultural (Herrick & Jones 2002), and pavement subgrade applications (Siekmeier et al. 1998, Ampadu & Arthur 2005, ASTM 2003). The ability of the DCP to evaluate soils at depth in a relatively non-destructive manner, along with its ruggedness, are advantages that the DCP enjoys over the nuclear density gauge/test.

The DCP has received much attention for pavement subgrade evaluations as mentioned, but less attention concerning foundation engineering applications. For the latter (the focus of this paper), the DCP is used mainly during site grading and/or foundation construction, but it may also replace a conventional soils exploration for certain lightly loaded buildings in areas where the general soil conditions are known (Sowers and Hedges 1966). And in these applications, the dynamic cone penetrometer test (DCPT) is widely considered more economical than the standard penetration test/testing (SPT). The DCPT is similar to the SPT in that it is a *dynamic* test involving dropping a known mass (hammer) a given distance, while an end element is advanced into the ground. In the case of the DCP, the hammer strikes an anvil which in turn drives a conical point/tip into the ground. As the cone penetrates the ground, energy applied by the hammer is dissipated. Blow counts in dynamic tests (e.g. SPT) have been correlated to relative density, friction angle, and other parameters (Terzaghi et al. 1996, Kulhawy & Mayne 1990). Burham & Johnson (1993) have provided a summary of similar correlations possible with use of DCPT.

This paper describes four example cases where the author used a DCPT approach to evaluate relative density/compaction of natural soils and an engineered fill to add to the body of knowledge of this instrument. Where appropriate, the results of the four case studies (along with others not presented) and other related testing, were used to develop relationships of the DCPT results to relative density and relative compaction.

1.1 *Sowers DCP Specifications*

Portable dynamic cone penetrometer test instruments and procedures are not currently standardized; numerous configurations of this type of instrument exist. Since the results of the dynamic cone penetration testing depend on the instrument configuration as well as the ground/soil conditions being considered, there is often little opportunity to use existing correlations for work in different areas. As a result, the practicing engineer is often in a position of using their dynamic cone penetrometer instrument of choice, relying on his or her own specific observations of results gained over time under specific ground conditions within a given area.

The DCP instrument considered in this paper is called the Sowers DCP after it developer, the late Professor George F. Sowers. The Sowers DCP (Sowers & Hedges 1966) uses a 6.8 kg (15lb) hammer falling a total distance of 50.8 cm (20 in), and it features a conical tip at the end — 3.8 cm (1.5 in) in diameter at the widest section (Figure 1). The instrument is driven in increments of 4.45 cm (1.75 in) with this configuration.

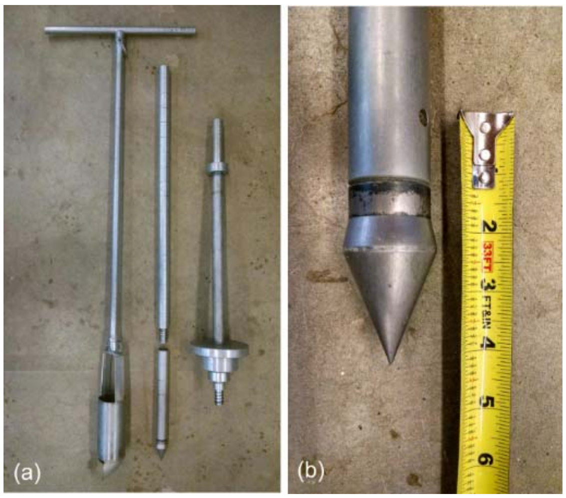

Figure 1. (a) Sowers DCP instrument components including hammer and anvil assembly, shaft, and conical tip, with hand auger shown; (b) close up of conical tip.

1.2 *DCP Operation*

Completion of a DCPT typically includes advancing a hand auger (a 7.6 cm (3 in) diameter auger was used in this work) to a given depth of interest. In the process, the operator recovers the cuttings to classify the soil(s). Once the borehole is prepared to the desired depth, the DCP is lowered into the hole. The required number of shaft extensions is installed on the DCP depending on the depth of the borehole. A taut string line, or other means of establishing a reference point, is placed across the borehole and DCP shaft to provide an accurate view of the movement of the incremental marks on the DCP shaft during testing, as the instrument descends. The hammer is lifted in a controlled manner and dropped a given number of times. The soil is penetrated a distance equal to the first increment. This is regarded as instrument seating. The weight is then dropped the required number of times to penetrate the soil at least one increment, but sometimes more depending on the ground conditions. If the blow count required to penetrate each incremental distance is not judged to be excessive (~ <12 per 4.45 cm (1.75 in)), then the DCP may easily be advanced continuously into the ground as a means of recording the trend of blow counts with depth. As the cone advances, soil is pushed to the side, in a manner that is analogous to a bearing capacity failure in relatively dense soils. In looser soils, the cone likely compacts the surrounding soil and pushes it aside as the cone penetrates the ground. The density of the soil and stress level determine how easily the cone is able to penetrate the material under consideration with the energy used. The deeper and denser the soil, the more energy required to penetrate the ground (greater number of blows).

2 OVERVIEW OF THE DEVELOPMENT OF DCPT TO RELATIVE DENSITY RELATIONSHIPS

The author evaluated a number of sites when considering the DCPT to relative density (D_r) (Eqn. 1) and relative compaction (RC) (Eqn. 2) relationships shown later. Of prime importance when developing the relationships was the quality and accuracy of the data and, by extension, the general homogeneity of the site under consideration.

$$D_r(\%) = \left(\frac{e_{\max} - e}{e_{\max} - e_{\min}} \right) x100 = \left(\frac{\frac{1}{(\gamma_d)_{\min}} - \frac{1}{(\gamma_d)}}{\frac{1}{(\gamma_d)_{\min}} - \frac{1}{(\gamma_d)_{\max}}} \right) x100 \tag{1}$$

$$RC(\%) = \left(\frac{(\gamma_d)}{(\gamma_d)_{\max}} \right) x100 \tag{2}$$

where e_{max} = maximum void ratio; e_{min} = minimum void ratio; e = *in situ* void ratio under consideration, $(\gamma_d)_{max}$ = maximum dry unit weight; $(\gamma_d)_{min}$ = minimum dry unit weight; and (γ_d) = *in situ* dry unit weight under consideration.

Data was gathered in a number of ways. Generally, sites including homogeneous sands of the type described below were considered the target soil type for the purposes of developing the types of correlations of interest. The ground conditions were often initially evaluated using SPT as part of a preliminary soils exploration. Sand samples collected from the spilt barrel sampler were often subjected to limit density testing using the simplified method by Muszynski (2006), which is tailored to clean sands, with relatively small available specimen volumes (e.g. split barrel sampler bulk specimens). In some cases, where maximum density tests were performed only, the minimum density was estimated based on an acceptable ratio of the minimum density to that of the maximum density.

On those sites of interest where SPT borings had been performed, some sites were further evaluated during construction. In some cases, as excavation commenced (e.g. basement, ramp, lower floor area, etc.), when and where possible, the author had a unique opportunity to perform density tests (nuclear density gauge) as excavation proceeded. These density observations were compared against the original estimates for D_r as determined by correlations with the SPT using the Gibbs & Holtz (1957) approach. Anecdotally, the author has found a good comparison between the D_r measured/calculated and those estimated using the Gibbs & Holtz (1957) approach.

On other sites, where SPT had been performed, with very soft, unfavorable soils present, the author may have also had the chance to observe removal of those soils, followed by replacement with engineered fill to arrive at the proper grade for the project. Projects calling for engineered fill to raise site grade were also of interest. As fill was being placed in either case, parallel nuclear density gauge and DCPT were performed at a variety of elevations, with the density of each lift documented. Samples of the sand were collected for additional limit density, water content, and other testing in the laboratory, where necessary.

The author would also often perform nuclear density tests and DCPT within borrow sand piles on site when time permitted. This allowed for data points at or slightly above $D_r = 0\%$. On borrow piles containing sand with water contents and/or a gentle dumping method leading to bulking (negative D_r), the results were disregarded.

Groundwater is a consideration with DCPT. With an uncased borehole, sandy soils quickly cave below the groundwater table and testing is often terminated at that time. However, when evaluating an engineered fill during or after placement, there are seldom groundwater problems within the fill itself (provided the fill is constructed above the groundwater level or the dewatering system remains operational during the time of testing).

2.1 *Sand description and suitable conditions for DCPT*

To date, the author's study has been limited to unsaturated (above the groundwater), clean (typically less than 3% silt), fine to medium, poorly-graded sands without appreciable coarse sand or gravel content. These soils were predominately poorly-graded sands (SP) according to the unified soil classification system (USCS). The soils were primarily quartz sands (specific gravity (G_s) of about 2.68) and often associated with dune, alluvial, or outwash deposits. The case studies described in subsequent sections were situated within Northern Michigan in the lower peninsula, generally just inland of the Lake Michigan shoreline. These soil conditions are prevalent throughout this particular area.

Figure 2a shows representative size distribution curves depicting the general type of sand considered. Figure 2b shows a sample of the sand particles typically observed within these specimens, where the particle roundness (R) is often interpreted to be between 0.3 and 0.4; subrounded to subangular (using Krumbein & Sloss 1963, Powers 1953). The limit densities of these sands most commonly ranged from 17.12 kN/m^3 (109 pcf) to 17.60 kN/m^3 (112 pcf) for the maximum dry unit weight (by modified Proctor, vibratory table methods, or the approach proposed by Muszynski (2006)) and 14.45 kN/m^3 (92 pcf) to 14.92 kN/m^3 (95 pcf) for the minimum dry unit weight. It is noteworthy to mention that the author has observed the ratio of the minimum to the maximum unit weight to be 0.85 (within approximately 0.01 to 0.02) in nearly every instance. That is, the range of the ratio of minimum dry density to maximum dry density is narrow for these soils.

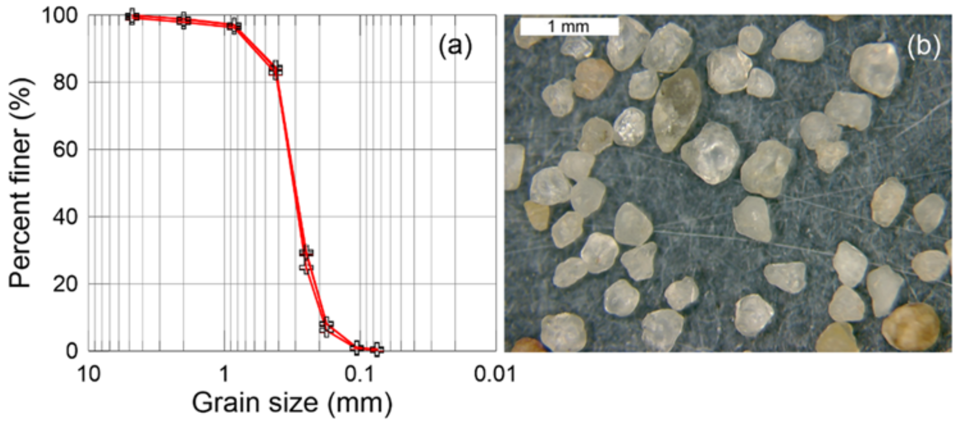

Figure 2. (a) Grain size distribution of typical sands encountered, and (b) image showing typical particle shapes observed.

2.2 *Case Study 1: Water tower tank foundation subgrade evaluation*

A new elevated water tank was to be constructed. A review of the geotechnical report indicated that very dense fine to medium sands would be encountered at the elevation of the proposed water tower foundation subgrade (4.6 m (15 ft) below natural grade). The SPT blow counts (N) were between 40 and 60 from the proposed subgrade level to boring termination at a depth of about 50 feet. Near the elevation of interest – 15 to 20 feet below original grade (the proposed tower footing subgrade level) – the blow counts were between 50 and 59. Note that no coarse sand or gravel was encountered during the SPT work, and that the SPT results represented unusually dense natural ground conditions. Also, N_{ave} and the N corrected for testing conditions (N_{60}) were taken as constant in the narrow depth range of interest (about 5 feet below proposed subgrade level).

The relative densities shown in Table 1 were estimated based on the Gibbs & Holtz (1957) relationships with SPT. As a side note, the author also performed work on a site directly adjacent to the water tower site (for a different project) and encountered similar, very dense sands. This observation suggests this particular area was relatively uniform in terms of its ground conditions.

Initially, the main concern was evaluating the exposed subgrade ground conditions to see that the conditions were consistent with those of the original geotechnical report. Once the contractor arrived at the proposed foundation level (using a smooth, non-toothed bucket excavator), the author used a combination of a nuclear density gauge tests and DCPT with hand auger. The nuclear density gauge, at the subgrade level, indicated undisturbed RC values on the order of 98 to 99% of the maximum density by modified Proctor. This was consistent with the original relative densities estimated using the soil borings completed (85 to 100%) using Gibbs & Holtz (1957) correlations. And because of the consistency observed in the SPT of the original soil boring program, the RC and D_r were both assumed to be constant to the depths of interest below subgrade level. Water contents of the natural sand averaged 1.9%. The DCP blow count values obtained were also relatively great, appearing to indicate very dense or compact ground conditions (Table 1), and therefore indicating consistency with the geotechnical report. Of special interest was the notion that the ground, considering the combination of the nuclear test results and the original SPT results, was a suitable site for evaluating the DCPT in very dense, natural sand conditions. Note that the DCP blow counts shown in Table 1 present the initial seating increment, followed by the subsequent increment used in the development of the plot shown later. The difference, in these very dense soils, between the seating increment and the subsequent DCPT blow count is substantial. However, as third and fourth increments were completed in some cases, the author observed that those blow counts were closer to that of the second increment than to the seating increment. This suggests that disregarding the seating increment (particularly in dense soils) is important.

Table 1. Water Tower SPT, DCP, and nuclear density gauge results.

Location	Depth below subgrade level		SPT blow counts		Relative density	Relative compaction	DCP blow counts
	(m)	(ft)	N_{ave}	N_{60}	D_r (%)	RC (%)	
15.5' s/o	0.30	1	54	65	85-95	98.8	13, 22
tower	0.95	3.1	54	65	85-95	98.8	21, 58
center	1.58	5.2	54	65	85-95	98.8	31, 81
13' n/o	0.85	2.8	54	65	85-95	98.8	18, 33
center	1.68	5.5	54	65	85-95	98.8	22, 70

2.3 *Case Study 2: New school engineered fill placement evaluation*

This case study involved using DCPT as a means of quality control testing during compaction of an engineered fill for construction of a new school. Testing performed on backfill specimens indicated limit densities as shown in Table 2, using the modified Proctor and minimum density by loose pour in a standard Proctor mold. The engineered fill was to be approximately 1.8 m (6 ft) above grade, and large in areal extent. The author originally performed a soils exploration for this site, and there were no special concerns of the underlying natural soils as it related to placement of the fill or support of the proposed buildings. During fill placement, a large vibratory roller compactor was used to compact 30 cm (12 in)-thick lifts of borrow sand from a nearby site. As fill was being placed and compacted, the level of compaction (using relative compaction as a metric) was checked using a nuclear density gauge. When the fill height arrived at approximately 1.5 m (5 ft), the author began DCPT. The water content of the borrow sand, as-conditioned, ranged between 5.5% and 8.3%. Table 2 shows the comparison between the DCPT blow counts and nuclear density gauge results. The author used these DCPT results to: a) see that the penetration values were consistent with a properly compacted engineered fill, and b) to gain additional information for development of DCP to relative density correlations.

Table 2. New school engineered fill DCPT and nuclear density gauge results.

Depth below grade		Limit densities (pcf)		Relative density	Relative compaction	DCP blow counts
(m)	(ft)	γ_{min}	γ_{max}	D_r (%)	RC (%)	
0.30	1	95.0	111.6	78	96.6	8
0.46	1.5	95.0	111.6	79	96.9	13
0.76	2.5	95.0	111.6	80	96.6	20
1.52	5	95.0	111.6	87	98.0	44

2.4 *Case Study 3: Commercial building #1 natural sand subgrade evaluation*

Excavation for a new commercial building foundation had already commenced when it came to the attention of the owner and contractor, by a building code representative, that a geotechnical exploration would be required. The site had been stripped of all topsoil and the site had been excavated partially for the foundation to subgrade level (conventional shallow footings extending to 1.4 m (4.5 ft) below original grade; below frost depth). The author led a geotechnical exploration including hollow stem auger (HSA) drilling around the perimeter of the building with interior hand auger borings with DCPT. The truck-mounted drill rig could not access the interior areas of the building because of the foundation excavation that had already begun. Instead, the hand auger and DCPT combination was used to gain information about the type and density of the soils at these interior locations.

Figure 3a shows all soil boring/SPT and hand auger/DCPT completed on the site. Soils consisted of fine to medium sand with a trace silt (SP); likely outwash deposits. The numbers within the boring symbols in Figure 3b represent the magnitude of the uncorrected SPT blow counts (on the SPT borings side), while the numbers on the hand boring (DCP) side represent the magnitude of the DCPT blow counts. The depths/elevations for each are indicated in Figure 3b as well. When corrected to N_{60}, the SPT values indicated natural relative densities between 25 and 65%. Of particular interest is the combination of soil boring SB-1 and hand boring HB-3, where HB-3 was conducted in relatively close proximity to the soil boring. Based on the results shown in Figure 3b, it appeared that with the reasonably close blow counts (blows per 30 cm (12 in) for the SPT, and blows per 4.45 cm (1.75 in) for the DCPT), and with the other soil borings/SPT conducted elsewhere on the site considered, an approximate 1:1 relationship exists between the SPT and DCPT results in these types of clean sandy soils.

Note that these DCPT and SPT comparisons were not considered in the development of the relationships shown later because nuclear density testing was not undertaken at the time of this site visit, and the D_r of the soils could not be substantiated in an acceptable manner. Instead, this case study serves to provide information about the relationship between the SPT and DCPT.

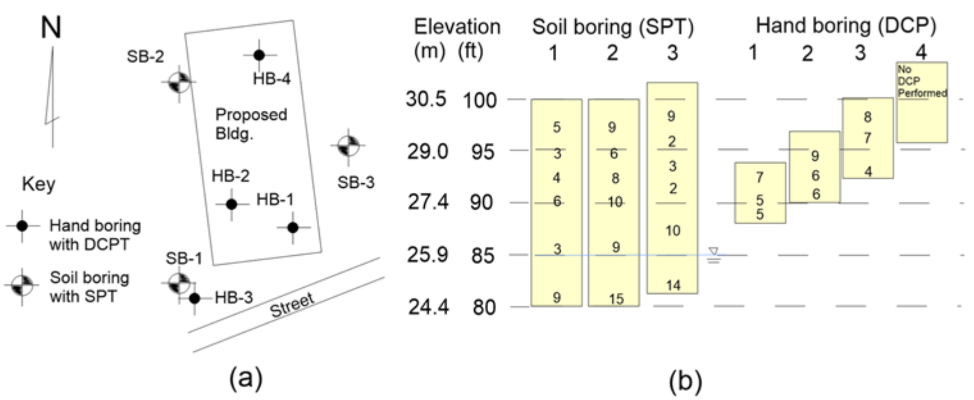

Figure 3. Commercial building #1 site map (a) and representative soil boring (b).

2.5 *Case Study 4: Commercial building #2 improved natural soils evaluation*

This case study involved natural (undisturbed) sands with one exception: the *in situ* sands needed to be improved to a depth of about a meter below foundation subgrade level to properly support the proposed structure. A variety of borings was undertaken (Figure 4a). The loose fine to medium sands (SP) on this site were likely ancient dune sand deposits. SPT values between 2 to 9 (corresponding to N_{60} values between 3 and 11) indicated D_r on the order of 25 to 50% (Figure 4b). The contractor elected to use a hoe-pack type compactor in an attempt to densify the relatively loose sands well below the foundation subgrade level without removal and replacement with engineered fill. With this decision, the author had the opportunity to use DCPT as a means to check the improvement in density of the soils below the subgrade level without disturbing the area with large test pits. Before and after DCPT were performed within the foundation subgrade. The contractor first excavated the subgrade to the proposed footing level. At this depth, the author completed several hand borings with DCPT to various depths below the subgrade level prior to compaction. It is noteworthy to mention, once again, that the DCPT results in the natural (undisturbed) sands were consistent with the average of the N counts from the soil borings completed on the site (Table 3). The contractor then applied vibratory compaction to the subgrade with the hoe pack compactor. The subgrade responded to the compaction by locally contracting in volume without any surrounding ground heave observed. This indicated that the settlement was generally due to densification of the loose sand rather than by displacement of the sand through a bearing capacity (shear) failure.

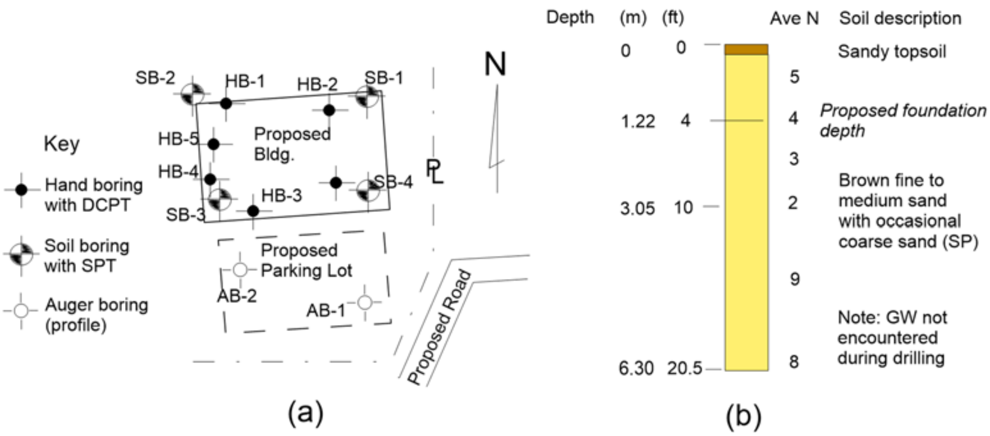

Figure 4. Commercial building #2 site map (a) and representative soil boring (b).

Table 3. Commercial building #2 SPT and DCP results.

Depth below subgrade level		SPT blow counts		Relative density[1]	DCP *before* compaction	DCP *after* compaction	% increase in DCP values*
(m)	(ft)	N	N_{60}	D_r (%)			
0.30	1	4	4.4	40	4	10	150
0.91	3	3	3.3	35	5	13	160
1.52	5	2	2.4	25	4	13	225

* [(Compacted DCP-Uncompacted DCP)/ Uncompacted DCP] x 100

3 ADDITIONAL DCPT

The author performed additional testing in a commercial sand pit near Petoskey, Michigan, using a series of test pits; each backfilled to a target compaction level, recorded using a nuclear testing gauge, with DCPT performed. The sands at the site were similar to the general sand type evaluated in the case studies previously described, with the grain size distribution for the sands shown in Figure 2.

Three pits (Pits P1, P2, and P3) were completed; each to approximately 1.2 m (4 ft) below the surface. Upon completion of each pit, it was backfilled in an attempt to arrive at a given compaction level, primarily by varying the number of passes with a vibratory plate compactor. Pit P1 was backfilled using a “moderate” compaction program consisting of approximate 20 to 30 cm (8 to 12 in) lifts and compacted with a single pass of the vibratory compactor. Pit P2 was backfilled by end dumping from the bucket of the excavator from a height of approximately 2.4 m to 3 m (8 to 10 ft) (in an effort to create a loose condition, yet one without appreciable bulking of the moist sand grains). Pit P3 was backfilled in approximate 15 to 30 cm (6 to 12 in) lifts and compacted with three passes of the compactor. Each lift, and all related pit depths, was measured with survey equipment with an on-site benchmark established.

DCPT were conducted within the completed backfilled test pit areas. In the case of this testing, the DCP was advanced multiple cone lengths into the ground in an effort to observe the changes with depth at each given borehole. It was also used to observe the continuity of DCP blow counts near the end of each run as compared with the beginning of the next run where the subsequent hand boring depth was reached. Figure 5a,b,c show cross sections of each pit (P1, P2, & P3, respectively) with relevant elevations of lifts, DCPT zones, and results. Note that the DCPT relationships developed considered each elevation of interest as an average of the DCPT results at the three test hole locations. As this figure shows, the DCP blow counts recorded in each test hole were relatively consistent for each elevation indicating a uniform pit preparation and compaction procedure, and suggesting good repeatability with the DCP instrument in the process.

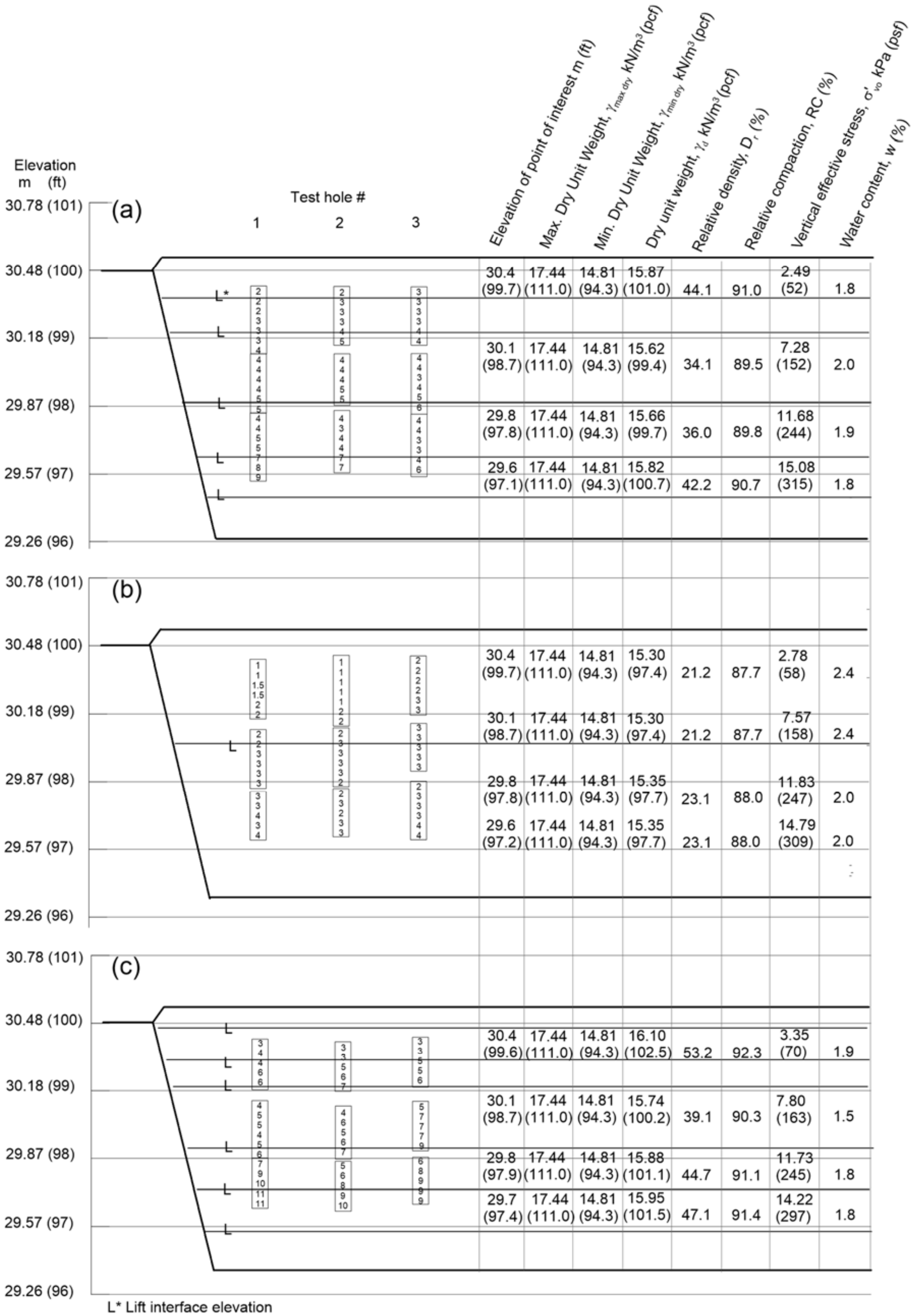

Figure 5. Cross sections of Test pits P1 (a), P2 (b), and P3 (c).

4 DCPT TO RELATIVE DENSITY AND RELATIVE COMPACTION RELATIONSHIPS

Developing DCPT to D_r and RC relationships has been an ongoing process, with the results used in practice as data is added, and supplemented by targeted testing specifically designed to obtain additional data. The current DCPT to D_r and RC relationships are shown in Figure 6a,b. The figure presents the data in terms of DCPT results with depth below subgrade (Fig. 6a) and vertical stress corresponding to the testing level (Fig. 6b). The engineer may elect to use one or the other to suit his or her preferences.

Figure 6 includes the author's interpretation of each depth (Fig. 6a) or stress level (Fig. 6b) in the form of trend lines. The solid lines, corresponding to low stress and shallow depths, are among those with the greatest number of data points available. The trend lines become increasingly tentative with increases in depth and stress because of the uncertainty/lack of data available.

Note that while the D_r ranges from 0% to 100%, the RC ranges from 85% to 100%. As mentioned earlier, this range of RC is specific to the clean, poorly-graded sands considered for these relationships and is based on the measured results of limit densities observed for these soils.

Figure 6 indicates that for very loose sands (D_r ~ 0% to 15%), stress level has relatively little absolute effect on the DCPT results. This is likely due to the high compressibility of the loose sands, and the tendency for the advancing cone tip to push and compact the soil grains laterally during penetration. On the other hand, at very high relative density (D_r ~ 80% to 100%), where the sand is generally less compressible, penetration of the cone requires the development of a shear failure within the soil, requiring more energy input at the cone tip. This behavior is most pronounced for the specimens at greater stress levels. This general penetration behavior is similar to that observed for other dynamic tests (e.g. Gibbs & Holtz 1957).

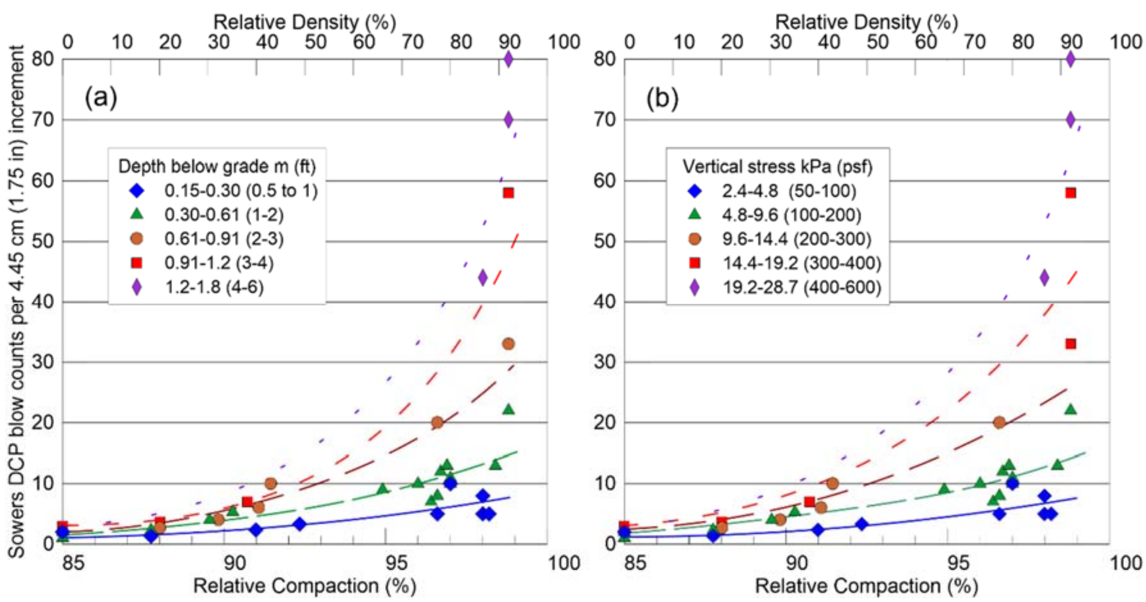

Figure 6. DCP to relative density and relative compaction relationship for Northern Michigan fine to medium quartz sands (natural and engineered fill); (a) depth-related, (b) vertical stress-related.

5 CONCLUSIONS

The case studies presented, in addition to the supplementary testing at the sand pit, represent examples of how the DCPT to D_r and RC relationships were developed. Other information was also gleaned from these case studies about how the DCP instrument may be used in practice. The following observations are made based on the case studies presented:

- a natural sand site may be evaluated during foundation excavation activities to see that the soils are consistent with the original geotechnical exploration (Case studies 1, 3, & 4);
- a newly placed engineered fill may be evaluated at depths below those accessible by the nuclear density gauge (Case study 2);

- Improvement in *in situ* density may be quantified using DCPT without the need for excessively disturbing the site by using a test pit-density test program (Case study 4);
- An approximate 1:1 (SPT:DCPT) relationship appears to be reasonable in practice for evaluation of sites formerly subjected to SPT (Case studies 1, 3, & 4).

Additional observations about the use and performance of the DCP instrument are as follows:

- At low relative densities ($D_r \sim 0\%$ to 15%), the absolute difference in DCPT performed at different stress levels is minimal as penetration of these soils is largely dependent on the ability of the instrument to compact soil laterally during advancement of the cone tip;
- At very high relative densities ($D_r \sim 80\%$ to 100%), interpretation of the DCPT results may be somewhat inaccurate, especially at greater overburden pressures, as the trend of the DCPT results begins climbing steeply;
- The DCP instrument appears to be well-suited to interpretation of the D_r or RC of sands near 65% or 95%, respectively; a common engineered fill specification by modified Proctor maximum density or maximum limit density by vibratory table methods.

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