International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

2010 - Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

27 May 2010, 5:05 pm - 5:15 pm

# Ground Improvement to Reduce Liquefaction Potential Using Vibrocompaction and Stone Columns

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Fifth International Conference on
Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
*and Symposium in Honor of Professor I.M. Idriss*
May 24-29, 2010 • San Diego, California

# GROUND IMPROVEMENT TO REDUCE LIQUEFACTION POTENTIAL USING VIBROCOMPACTION AND STONE COLUMNS

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ABSTRACT

With the rapid pace of industrialization, structures are being designed and constructed in the flood plains of major rivers. In earthquake prone areas, a fundamental issue in the design and construction of structures on saturated sandy soils is weather or not the design earthquake could initiate liquefaction in the form of lateral spreading, sand boils, settlement, or cracking. Many different methods, including vibrocompaction, deep dynamic compaction, compaction piles, geopiers, deep mixing, vibratory probes, displacement/compaction grout, etc., have been used to reduce the liquefaction potential at various sites. Use of vibrocompaction to densify cohesionless soil is becoming more common and cost effective. For projects in the New Madrid seismic zone (NMSZ) another challenge to perform site specific analysis is the lack of recorded ground motions. Therefore, synthetic time histories need to be generated using the attenuation models applicable to the region. This paper provides details about a site specific study performed for a site in the bootheel area of Missouri, and results of liquefactions analysis and ground modification achieved using vibrocompaction.

## INTRODUCTION

Liquefaction of saturated sands has been the topic of extensive research over the past four decades. A number of publications and special presentation papers have discussed the expanded interest in liquefaction and its effects (e.g., Arulanandan et al. 1995; Dobry et al. 1995; Finn 1991, Kumar, 2000, 2001; Kutter 1995; O'Rourke and Pease 1995; and Youd 1993, 1995). Laboratory experimentation and field testing on soil liquefaction has provided valuable insight into the mechanism of excessive pore-pressure buildup (National Research Council 1985).

Damaging earthquakes occur infrequently in the Central United States (CUS). The earthquakes of 1811-1812 caused damage in the St. Louis area, at least 175 miles from the main-shock epicenters. However, because of the sparse population and simple, log cabin structures in the region during this era, a relatively small number of deaths and minimum property loss was observed. The earthquakes of 1811-1812 caused liquefaction and landslides in an area of 6,000 square miles in southeast Missouri, western Tennessee, and northeastern Arkansas. Although, surface indications of liquefaction during these earthquakes are rare in the St. Louis metropolitan area, any liquefaction below the ground surface today is likely to cause significant loss of life and property (Kumar 2001).

Paleo-seismic studies suggest that the region has experienced several major prehistoric earthquakes with an approximate recurrence interval of 500+ years. However, it is important to note that three of the largest earthquakes in the Central United States during the 20$^{th}$ century were not on the New Madrid fault. Two were on the Wabash Valley fault, which runs approximately north-south from the Ohio River along the Illinois-Indiana state line and the third occurred on the Cincinnati Arch near Sharpsburg, Kentucky. The largest earthquake from the New Madrid fault in the 20$^{th}$ century was in 1976 near Marked Tree, Arkansas (CUSIES 1994).

Bootheel area of Missouri lies near the northern edge of Mississippi embayment. The Mississippi embayment is a physiographic feature in the south-central United States which is essentially a northward continuation of the Mississippi River delta. The embayment is a topographically low lying basin that is filled with tertiary to recent sediments. The NMSZ, also known as the Reelfoot Rift or New Madrid Fault Line, lies at the northern end of the embayment. The NMSZ extends southward from Southern Illinois, through the Missouri boot heel and western Kentucky, into northwestern

Arkansas. The fault zone in this area is predominantly characterized by high-angle normal faults. Figure 1 shows the epicenters of various earthquakes recorded in the vicinity of the site. The size of the circle is related to the magnitude of the earthquake as shown in the legend for Fig. 1.

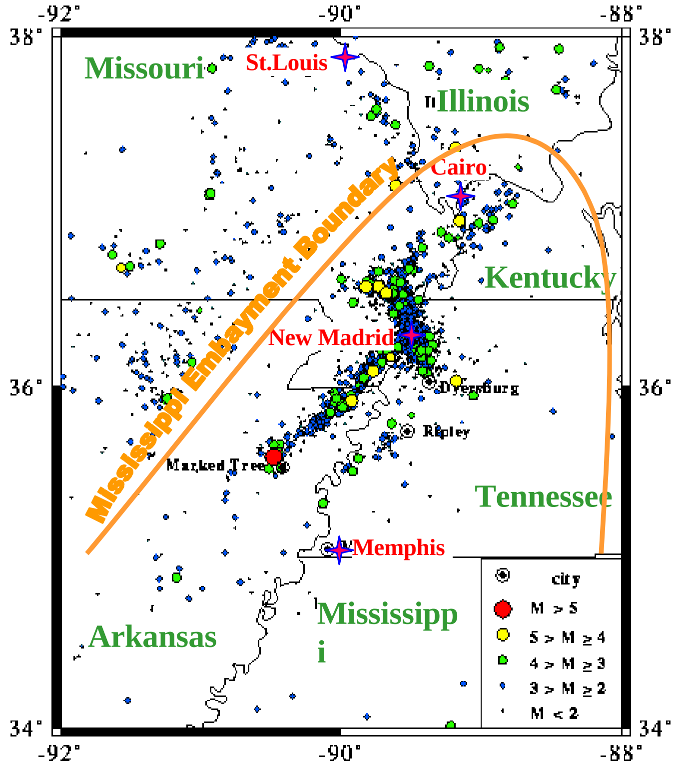


Fig. 1. Seismicity in General Vicinity of the Site

The project consisted of construction of a single story structure founded on shallow foundations. A site specific seismic study along with liquefaction analysis was performed to develop seismic design parameters as per 2003 International Building Code (IBC). Since strong ground motion data are not available for CUS, synthetic earthquake time histories were used to perform ground response analyses. Based on the liquefaction analysis performed, it was concluded that the existing soils have potential for initiation of liquefaction. Vibrocompaction along with construction of stone columns was used to remediate the site. A smooth, uniform hazard, response spectrum based on the seismic parameters used in the International Building Code (IBC, 2003) for 2 percent probability of exceedance in 50 years (i.e., 2500-year return period) and 5 percent damping was developed. Analysis procedures used and results of site remediation are presented.

SUBSURFACE CONIDITONS

A total of eight borings were drilled at the site as a part of original subsurface exploration. Four borings were drilled to depths of 20 feet, three borings were drilled to depths of 10 feet, and one boring was drilled to a depth of 100 feet below the existing ground surface using CME 750 drill rig mounted on an all terrain vehicle (ATV). Standard penetration tests were performed using an automatic hammer. Grain-size distribution tests and amount of fines tests were performed on selected samples obtained from the 100 foot deep boring.

In general, the soil stratigraphy at the site consists of intervening layers of brown and gray, silty clay, sandy clay, sandy silt, and silty sand to depths of 11 to 17 feet. Below this stratum, the soil layer consists of gray, loose to medium dense, fine to medium sand down to the maximum depth explored, i.e. 100 feet. The fine content (material with grain size less than 0.075 mm) in the sand stratum was generally less than 3 percent. The groundwater was encountered at depths between 9 and 11.5 feet during drilling. Groundwater level at the site depends on the water levels in the nearby Mississippi River and varies significantly over time due to the effects of seasonal variation in precipitation, recharge, or other factors not evident at the time of exploration.

ANALYSES FOR EXISTING SUBSURFACE CONDITIONS

Figure 2 presents the measured N-values ($N_{msd}$) and corrected N-values [N-corr or $N_{1(60)}$] from the 100 foot deep hole. The N-values were corrected for the overburden and hammer energy, assuming the efficiency of the automatic hammer used to be 75 percent. The average N-value for this site ( $\bar{N}$ ) as per the recommendations of IBC 2003 was calculated to be 13.1.

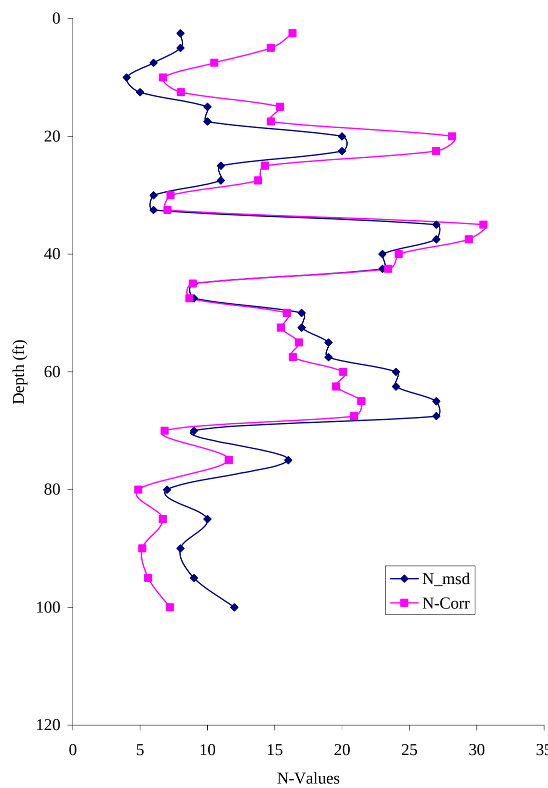


Fig. 2. Measured (N_msd) and Corrected (N-Corr) N-Values Observed at the Site

Based on the average $\overline{N}$-value in the top 100 feet, the site was classified as Site Class E. However, due to potential for liquefaction, the site was classified as "F" and the site specific ground response analysis was performed.

## DYNAMIC SOIL PROPERTIES

For seismic ground response analysis, low strain soil shear modulus and damping are the required dynamic soil properties. Brief discussion on these properties is given below.

Low Strain Dynamic Modulus. A key parameter necessary to evaluate dynamic response of soils is the dynamic shear modulus, $G_s$ or shear wave velocity which is also related to dynamic shear modulus. Shear modulus is not a constant property of soil but decreases nonlinearly with increasing strain. For initial design purposes, shear modulus measured at small shear strain amplitudes (less than $10^{-4}$ percent), referred to as $G_{max}$, is a desired design parameter.

The shear wave velocities for the upper 100 feet of soil strata were estimated from the N-values using the correlations developed by Wei, et al. (1996), and the shear wave velocities for the remaining depth of soil/rock (from the B-C Boundary to 100 feet) were estimated based on the shear wave velocity profile discussed in Pezeshk et al. (1998 and 2004). The shear modulus, $G_{max}$, corresponding to small shear strain was estimated based on the estimated shear wave velocities.

Damping. The inelastic behavior of soil also gives rise to energy absorption characteristics of soil which is known as material damping. Damping is generally expressed as percentage of the critical damping. Low strain damping of approximately 5 to 10 percent of the critical damping is commonly used for soils. Damping of 5 percent of critical was used for the analysis. However, this damping was modified in the analysis based on the strain levels in the soil.

Effect of Strain on Dynamic Soil Properties. It is well understood that the stress-strain relationship of soils is nonlinear. This means that the soil shear modulus and damping are not constant values but degrade nonlinearly with increasing strain in the soil. Dynamic analyses considering true nonlinear behavior of soil are very complicated and therefore, equivalent nonlinear analysis is most commonly used in practice. Equivalent nonlinear analyses consists of performing a series of linear analyses, in an iterative way, using, for each analysis, soil properties consistent with the strains resulting from the previous one. Equivalent nonlinear analysis was used in the present study. Many studies have been performed in the past to establish a relationship between modulus degradation with strain. The shear modulus degradation curves and damping ratio curves used were taken from Pezeshk et al. (1996) and Chang et al. (1989).

## GROUND RESPONSE ANALYSIS

Ground response analysis was performed to obtain representative response spectra at the ground surface based on the time histories at B-C boundary propagated through the site soils. According to the United States Geological Survey (USGS) Hazard Maps, the project location has a mapped 0.2 second spectral response acceleration ($S_s$) of approximately 2.38g, a mapped 1.0 spectral response acceleration ($S_1$) of approximately 0.76g, and peak ground acceleration (PGA) of 1.31g. Site specific, synthetic earthquake ground motions of uniform hazard for 0.2 second and 1.0 second response spectral values and PGA were then developed using the following procedure.

Horizontal bedrock time histories were generated at the site from a seismologically-based model mainly due to shear waves generated from a seismic source. The seismologically-based model used included effects of attenuation, characteristics of the source zone, recurrence interval, and the seismotectonic setting of the New Madrid seismic zone, Wabash zone, and other potential seismic sources in the region. To accomplish this task the following steps were taken:

1. Seismic source zones were identified that could significantly contribute to the seismic hazard at the site,
2. Ground motion attenuation relationships of response spectral values of 0.2 second and 1.0 second and PGA developed and discussed in Pezeshk (2004) were used,
3. Maximum earthquake magnitudes and earthquake recurrence rates of each identified seismic source zone were determined based on published data,
4. Probabilistic seismic hazard analyses were performed to determine probabilistic consistent magnitudes and epicentral distances using the attenuation relationships for spectral accelerations of 0.2 and 1.0 seconds and PGA, and
5. Boore's earthquake generation program (SMSIM) was used to generate horizontal bedrock time histories. These time histories were then propagated from the focal depth to the NEHRP B-C boundary using the quarter-wavelength approximation method and values suggested in Boore and Joyner (1997) for hard rock in eastern and central North America.

According to the results of the probabilistic study, the design spectral accelerations, $S_{DS}$ and $S_{D1}$, were determined to be 0.54g and 0.98g, respectively. However, according to IBC 2003, the site specific acceleration coefficients cannot be lower than 80% of the code acceleration coefficients. Therefore, the site specific acceleration coefficient at short periods, $S_{DS}$, was adjusted to 1.141g. The peak ground acceleration at the ground surface was estimated to be 0.74g. The design response spectrum using these values and the design response spectrum for Site Class "E", developed as per IBC 2003 are shown in Fig. 3.

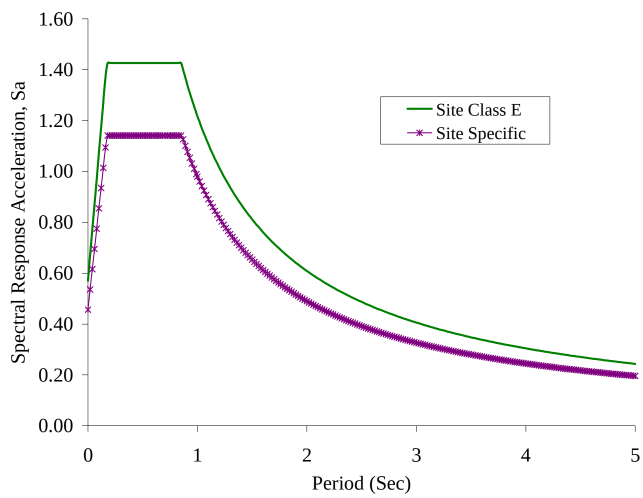


Fig. 3. Design Response Spectra for Existing Soil Conditions

## LIQUEFACTION ANALYSIS

Liquefaction is a phenomenon of loss of shear strength of saturated soils due to the sudden increase in pore pressures. Generally, loose cohesionless soils are susceptible to liquefaction. However, studies have shown that certain low plastic clayey soils may also suffer strength loss during and immediately after an earthquake.

Subsurface exploration at the site indicated that the existing soils are primarily loose to medium dense sands except the surface stratum which consists of intervening layers of brown and gray, silty clay, sandy clay, sandy silt, and silty sand. Groundwater was encountered at depths between 9 and 11.5 feet at the time of exploration which fluctuates depending on the water levels in the Mississippi River. Because of the presence of low density, saturated sands having relatively uniform grain size distribution, and the level of ground shaking expected at the site from an earthquake, the site was identified to have significant potential for liquefaction. Analysis was performed to determine the density of sands required to reduce the potential of liquefaction. These densities were then compared with the densities of the existing soil to determine the liquefaction potential of the site.

Liquefaction analysis was performed using the simplified method originally proposed by Seed and Idriss (1971, 1982) and Seed et al., (1983) which is based on in-place evaluation of resistance of soils. Simplifications and modifications proposed by Youd et al. (2001) were used to perform the liquefaction analysis. This method is based on the extensive analysis of field data from sites which liquefied or did not liquefy in various earthquakes in the past. The procedure consists of comparing the shear resistance of the soil (in terms of corrected blow count, $(N_1)_{60}$) to the cyclic shear stresses expected from the design level earthquake.

To determine the liquefaction at the site, the corrected number of blows $[(N_1)_{60}]$ required at any depth to reduce the liquefaction were estimated using the simplified procedure. Figure 4 shows the corrected $(N_1)_{60}$ measured during the subsurface exploration and corrected $(N_1)_{60}$ required to reduce the liquefaction potential at the site.

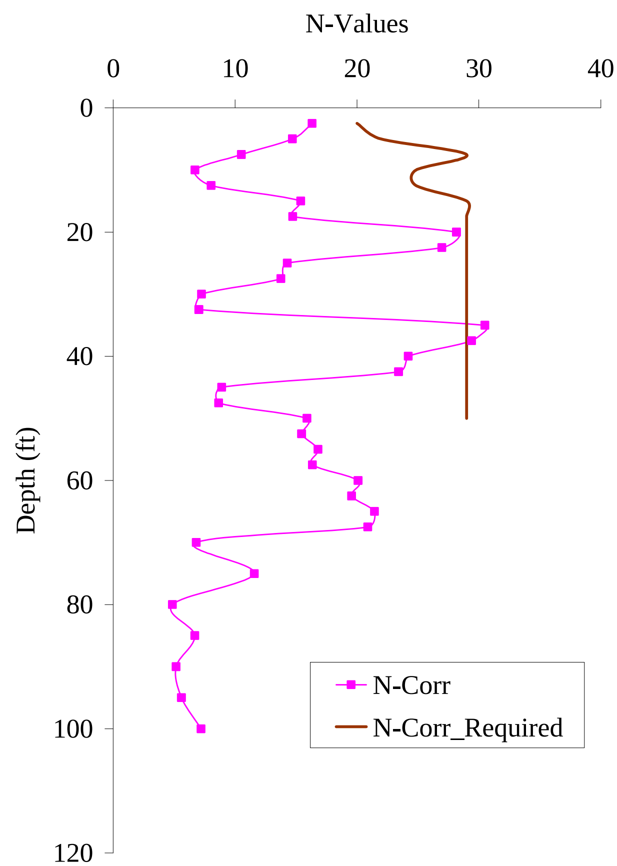


Fig. 4. Corrected N-Values Measured (N-Corrr) and those Required to Reduce Liquefaction Potential (N-Corr_Required)

Based on the results of the liquefaction analysis performed, it was determined that the site soils, if not improved, have significant potential for liquefaction. However, improvement of soils below 35 feet from the existing ground surface was considered difficult and cost prohibitive because of the limitations of soil improvement equipment and the existence of a layer of medium dense sand at an approximate depth of 35 feet. Therefore, site soils were improved to depths of approximately 35 feet from the existing ground surface. Since the maximum width of the footing for the proposed building was likely to be less than 7 feet, the stress in the soil below 20 feet from the bearing elevation of the footing was likely to be less than 10 percent of the stress at the bearing elevation. Therefore, authors believed that liquefaction, if any, at depths below 35 feet may not significantly affect the structures as long as there is no flow of soil due to liquefaction and the liquefied soil layer does not significantly disturb the overlying soil layer. Ishihara (1985), based on the analysis of data from several case histories, showed that a layer of non-liquefiable surface layer is likely to prevent ground rupturing from liquefaction happening at depths.

## SITE IMPROVEMENT TO REDUCE LIQUEFACTION POTENTIAL

Based on the results of liquefaction analysis, it was decided to improve the site soils to depths of approximately 35 feet from the ground surface. The improvement was recommended to be least 10 feet beyond the footprint of the proposed structure. The remedial measures for reducing liquefaction potential depends on factors such as technical adequacy, long-term performance, environmental impacts, maintenance, economics, and many others. The remedial measure may consist of any one or a combination of the following techniques:

- Heavy tamping (deep dynamic compaction)
- Vibrocompaction
- Construction of Stone Columns
- Construction of Geopiers
- Injection and grouting

Based on subsurface conditions observed in the boring and existence of other structures in the area, vibrocompaction along with construction of stone columns was recommended to improve the subsurface conditions at the site.

Vibrocompaction, sometime also known as Vibroflotation, is generally used to densify clean, cohesionless soils. The action of the vibrator, commonly referred as float or probe, is usually accompanied by water jetting to reduce the inter-granular forces between the soil particles thus allowing them to move into a denser configuration. Relative densities of 70 to 85 percent could be achieved with vibrocompaction. Typically, densification causes the soils in the immediate vicinity of the probe to settle. Therefore, additional cohesionless soils are added during the vibration process. A variation of typical vibrocompaction is construction of stone columns. A stone column is constructed during compaction by pushing crushed stone into the hole created by probe and compacted by the vibratory action of the probe. Compaction can be achieved above and below the water table.

Before start of the site improvement work, several borings were drilled to establish the baseline N-values before compaction. Vibrocompaction was accomplished by penetrating the probe in a 7x7 ft grid pattern. Figure 5 shows vibrocompaction in progress. In order to verify level of site improvement, several borings were drilled after densifying the soils. Figure 6 shows N-values measured in the baseline boring (BB-6) compared to N-values measured in other borings drilled after first round of vibrocompaction. Targeted N-values are also shown as Required N. As evident from the data presented in Fig. 6, first round of vibrocompaction resulted in some improvement of site soils but improvement in most of the borings was still below the targeted improvement levels. Therefore, additional compaction was accomplished in the zones where insufficient densification was observed. Figure 7 shows N-values measured after second round of compaction. The results show that vibrocompaction was effective in compacting the soils to targeted N-values, except at a few isolated depths. Since the borings were drilled immediately after compaction, pore water pressures in the soils were likely to be high. Therefore, it was concluded that after the excess pore water pressures had opportunity to dissipate, N-values are likely to be higher than those measured.

Fig. 5. Vibrocompaction in progress

## CONCLUSIONS

A site-specific ground response and liquefaction analyses performed for a site in the boot heel area of Missouri are presented. The liquefaction analysis showed that the existing soils at the site had significant liquefaction potential. The site soils were densified using vibrocompaction and construction of stone columns. Results are presented to show that the procedure used to densify the site soils successfully improved the soils to targeted N-values. N-values in some isolated zones in sand layers were noted to be slightly lower than required, however, the low N-values recorded are likely to improve with time as the excess pore pressure dissipates.

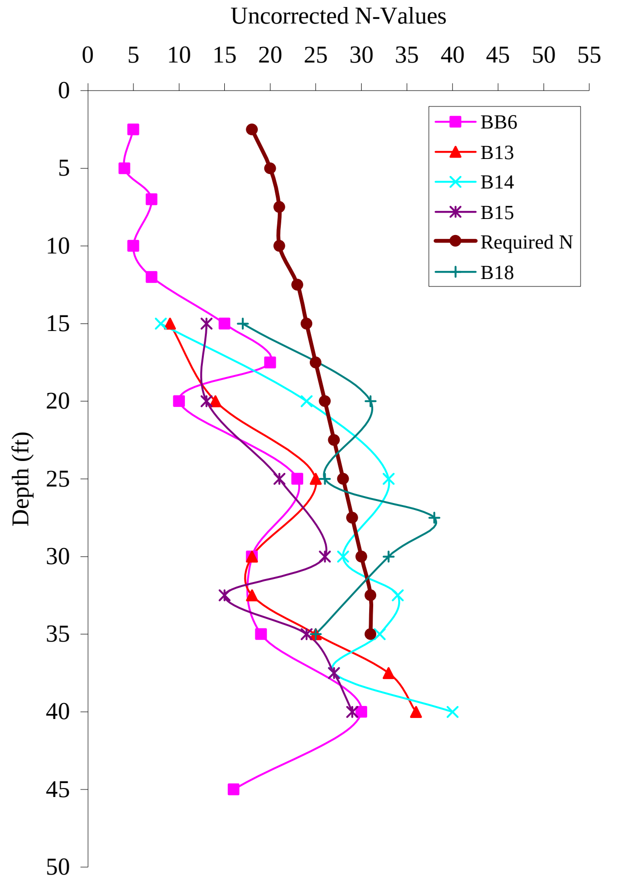


Fig. 6. Measured N-values compared to baseline N-values and Required N-values after first round of compaction

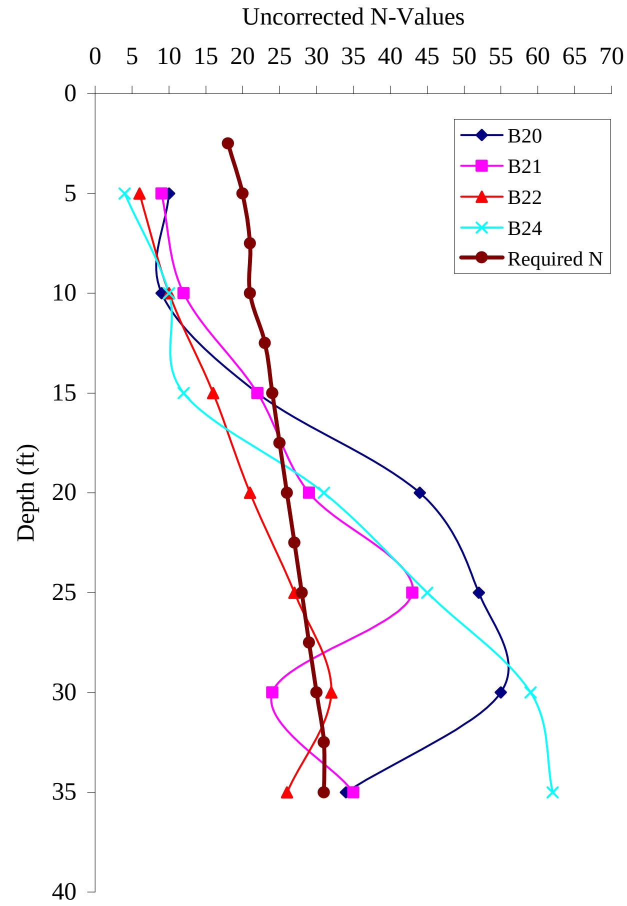


Fig. 7. Measured N-values compared to Required N-values after second round of compaction

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