Maejo Int. J. Sci. Technol. **2018**, *12*(03), 206-220

Maejo International Journal of Science and Technology

ISSN 1905-7873

Communication

Dynamic compaction experience in alluvial soils of Carsamba (Turkey)

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Received: 3 May 2017 / Accepted: 18 September 2018 / Published: 3 October 2018

Abstract: A case study using dynamic compaction to improve alluvial soils of a collective housing project area in Carsamba, Turkey, is presented. In-situ field pilot tests were employed to determine the optimum number of tamping, the grid spacing, the effective depth of improvement and the degree of densification of compacted soil. Optimal compaction conditions were found to be: grid spacing – 6.0 m and tamping number under a weight of 15-ton mass falling freely from a height of 18 m - 6.0. Based on the measurements of Standard Penetration Test Numbers (SPT-N) values before and after the dynamic compaction, it was found that SPT-N values were increased by more than 100% near ground surface (including the effect of filling and levelling) at some points, and the depth of ground improvement was experimentally determined to be 9.5 m. Dynamic compaction had an unfavourable effect on thin clay layers of about 0.5 m thick found at varying depths in two different locations since SPT-N values for the clay layers were reduced by as much as 50% after dynamic compaction. However, soil layers beneath the clay layers could be improved to some extent.

Keywords: dynamic compaction, alluvial soil, thin clay layer, ground improvement, Carsamba (Turkey)

INTRODUCTION

Dynamic compaction (DC) is a widely used method of increasing the bearing capacity and reducing the compressibility characteristics of a wide variety of soils [1-3]. It consists of using a heavy tamper that is repeatedly raised and dropped with a single cable from varying heights to compact the ground. The mass of the tamper generally ranges from 5.4 to 27.2 ton and drop heights range from 12.2 to 30.5 m. The energy is generally applied in phases in a grid pattern over the entire area using either single or multiple passes. Following each pass, the craters are either levelled with a dozer or filled with granular fill material before the next pass of energy is applied [4]. This method was first introduced and used in France in 1970 and can be defined as the densification process of soil mass brought about by sudden impact loading, including shear deformation, temporarily high pore pressure increase and subsequent consolidation [5]. It was reported that one of the most important considerations regarding the applicability of DC is the type of soil being densified [4-6]. Because of its simplicity, fastness, practicability, and cost-effectiveness, it has become a well-established ground improvement technique all over the world [7].

DC has been utilised in different types of civil engineering projects including building structures, highways, airports, coal facilities and dockyards, and to reduce the liquefaction potential of loose soils in seismically active regions [8]. In practice, the design and application of dynamic compaction are still largely empirical in nature, relying heavily on the designer's experience and judgement.

One of the main problems facing design engineers and contractors is the assessment of the parameters required to attain satisfactory ground improvement depth. The related firms have been using a simple relation to estimate the depth of ground improvement for a long time [9]. The estimates obtained from that relation may vary considerably and in many cases may be rendered useless for dynamic compaction design [9]. A pilot test is often carried out at the site to ascertain the operational parameters so as to minimise the operational costs [10].

The main contribution of this paper is the evaluation of the effect of thin clay layers present within alluvial soil on the crater depth, ground surface heave, and the depth of ground improvement, which depends on the distance of the thin clay layer from the ground surface. We also aim to provide such compaction parameters as the number of tamping, the grid spacing, and the degree and effective depth of densification, which are needed for satisfactory improvement of the foundation soil of the collective housing project site.

For the evaluation of performance, standard penetration tests and topographic surveys were performed before and after DC and the findings were evaluated.

SITE SUBSURFACE CONDITIONS

The project site was on the delta formed by Yesilirmak River running through Carsamba district of Samsun (Turkey). The delta was composed of alluvial materials of silt, clay and larger particles of sand and gravel (Figure 1). The subsurface ground conditions and geotechnical engineering properties were evaluated based on field and laboratory data. The borehole explorations up to 18 m in depth, grain size distribution (Figure 2) and consistency limit tests revealed that the site typically consisted of 0.50 m thick organic matter at the top, followed by about 2.50 m thick poorly graded sand layer underlain by poorly graded silty gravel layer in zone I and poorly graded gravel in zone II. Zone I (pattern I) and zone II (pattern II) were two different locations in the field for the pilot compaction tests before the finalisation of the DC process. A layer about 0.50 m thick of low-plasticity clay was located at a depth of 10.50-11.00 m in zone I and a high-plasticity clay layer was located at a depth of 7.00-7.50 m in zone II (Figure 3). The moisture contents of the high- and low-plasticity clay layers were 22% and 23% respectively. In addition, the plastic limits of the high- and low-plasticity layers were 23% and 22% respectively, which were almost the same as their moisture contents. Moreover, their plasticity indexes were 41 and 28 respectively. Furthermore, the undrained shear strength of clay layers varied from 165 to 223 kPa. All these data indicated that the clay layers might be over-consolidated. A static water level was encountered at a depth of 5.5 m below ground surface. The moisture content of the ground except the clay layers varied from 11% to 17%. Prior to DC, the upper 3 m of soil layer was removed in order to allow impact to occur at the foundation level of the project.

Figure 1. The soil at project site

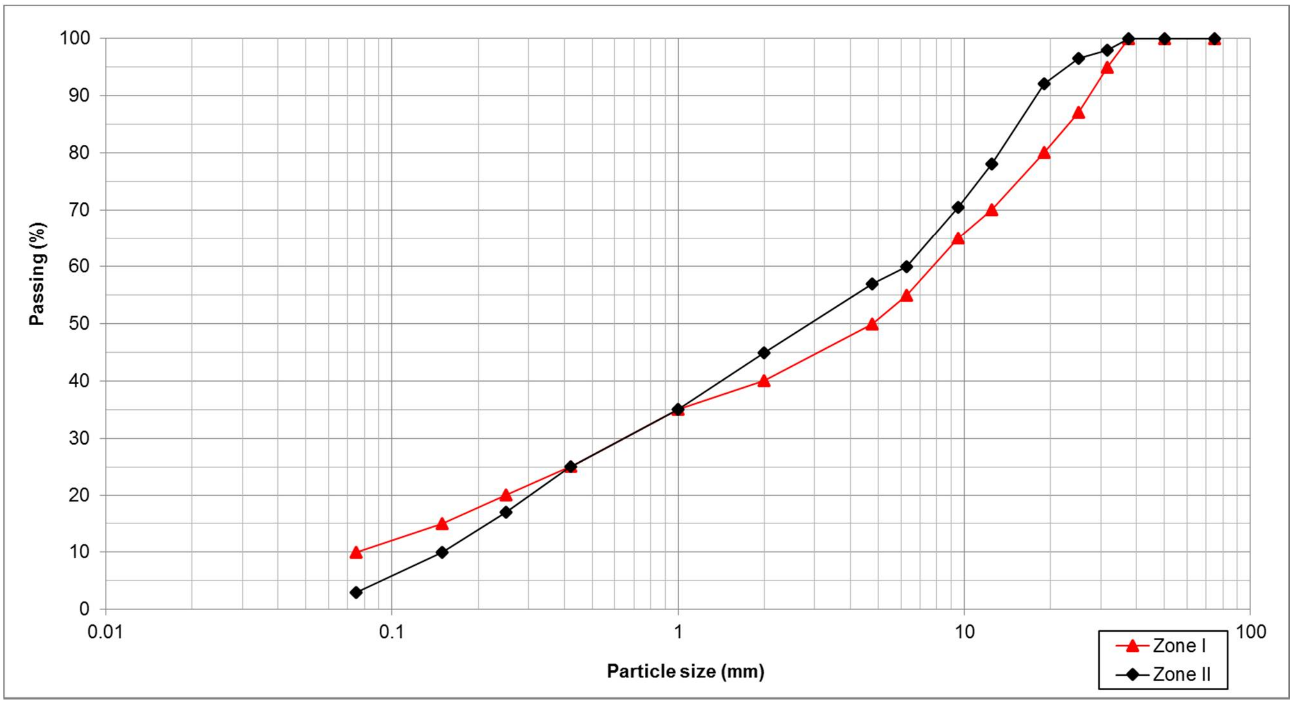

Figure 2. Particle size distribution of soil in zones I and II

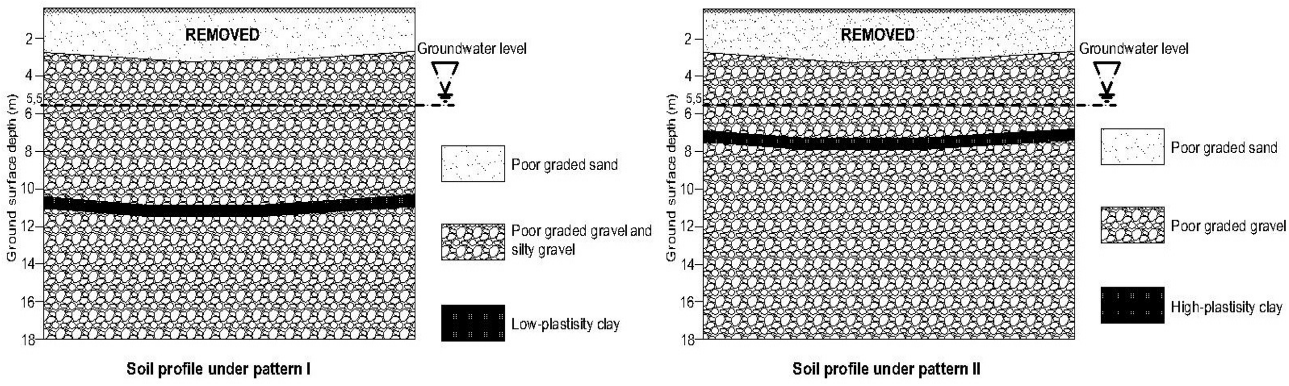

Figure 3. Subsurface soil profiles

PROJECT REQUIREMENTS

The collective housing project was located in a seismically active region. It consisted of eight-story buildings with a size of 20 m x 20 m of mat foundation and 3-m foundation depth. The net foundation pressure was 165 kPa. Moreover, the average allowable bearing capacity and the settlement for the project site soil, considering an 8 m depth below foundation base based on Burland and Burbridge [11] approach, were estimated to be 163 kPa and 25 mm respectively.

Two crucial points in this project site were the allowable bearing capacity and possible differential settlements of subsoil at some locations due to heterogeneity. In addition, since the project site was located in a seismically active region, the bearing capacity loss and differential settlement of the foundation soil due to earthquake were of main concern. Therefore, the main objective of subsoil treatment in the project site was to achieve a more homogeneous subsoil condition below the foundation base.

DESIGN CONSIDERATIONS

Before the implementation of full-scale DC over the whole area of the collective housing project, two pilot test areas of about 20 by 20-m at different locations within the project site were selected. Then the upper 3-m soil layer was removed since the collective housing project entailed basements for the buildings to be constructed. Thereafter, a series of standard penetration tests (SPTs) were performed within the pilot test areas at an interval of 1.5 m to a depth of 9.5 m as the 8-m depth of improvement below the base of excavation was considered to be sufficient for meeting the requirements of foundation design. The SPT is the most widely used *in situ* test throughout the world, as an indicator of the density and compressibility of granular soils. It is also commonly used to check the consistency of stiff or stony cohesive soils and weak rocks. There are many variations in international SPT practice, which leads to differences in the penetration resistance determined in similar soil types. In this study the test procedure and their evaluation were conducted according to ASTM D 1586 [12].

The maximum depth of ground improvement ($D_{max.}$) in metres for granular soil was initially estimated by the following equation (1) [13-14]:

$$D_{max} = n \times (W \times H)^{0.5}$$

where W is the weight of tamper in metric tons, H is the height of tamper drop in metres, and n is soil constant (0.5 for relatively coarse, predominantly granular soil). Based on the above empirical relation, the maximum depth of ground improvement with a weight of 15 ton dropped from 18-m height would be 8.2 m, sufficient for the project site under consideration.

Pilot test programmes were conducted to establish such criteria as spacing of impact points, number of tamping and phasing. Two different square grid spacing patterns (Figure 4) were investigated: (1) 5x5 m^2 and (2) 6x6 m^2. In general the weight is dropped on the square grid patterns of 5–10 m. Five to ten blows are applied to each imprint in each pass of the weight [15]. Initially, impacts were made with seven drops at each point in both patterns by repeatedly lifting and dropping a 15-ton 2-m^2 tamper from a height of 18 m (Figure 5). Settlement in the form of crater formation following each series of drops and ground heaves between impact points were monitored during the compaction process. Craters up to 1.07 m deep were observed at tamping points as shown in Figure 6. In addition, Figure 7 shows heaves up to 80 mm occurring between tamping points during the compaction process.

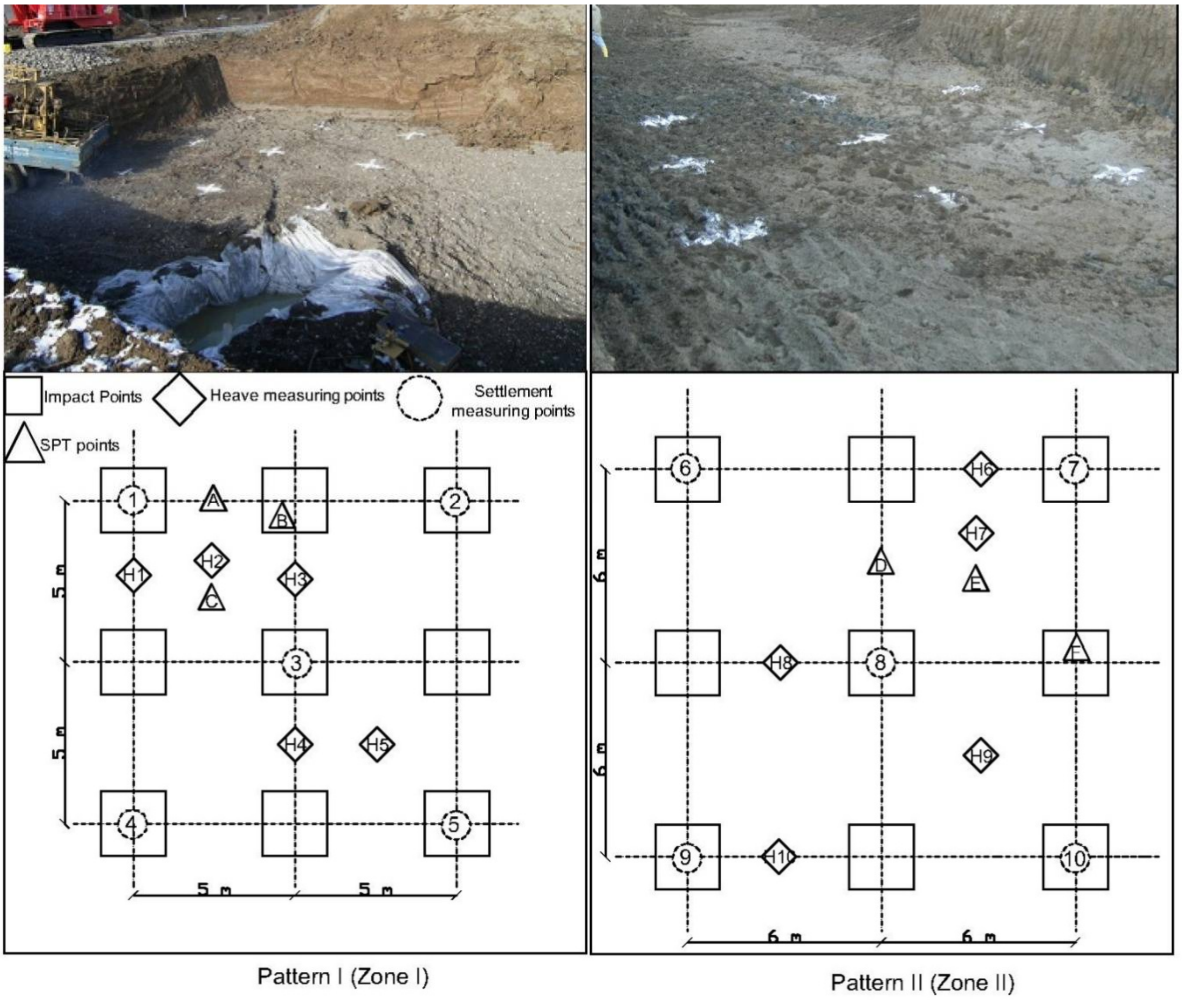

Figure 4. Layout of impact and test points (A, B, C, D, E and F are SPT points; H1, H2, H3, H4, H5, H6, H7, H8, H9 and H10 are heave measuring points; 1, 2, 3, 4, 5, 6, 7, 8, 9 and 10 are settlement measuring points.)

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Figure 5. DC equipment set-up

Figure 6. Measurement of crater depth

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Figure 7. Measurement of heave around tamping point

The craters were refilled with the surrounding soil using a dozer. Finally, a low-energy or 'ironing' phase was performed to densify the crater backfill and the disturbed soil between the craters by dropping a 15-ton tamper twice from a height of 3 m. After the ironing compaction, SPTs were implemented after a 24-hr waiting period to allow dissipation of excess pore pressure within the compacted granular mass below groundwater table. The results before and after DC are given in Figure 8. These are N^1_{60} values (corrected SPT-N values).

RESULTS AND DISCUSSION

Crater Depth

Crater depths as a function of number of blows at five different points under pattern I and pattern II are presented in Figure 9 and Figure 10 respectively. The figures indicate that crater depth increased as the number of drops increased in general. The first tamping produced the highest depth for craters and formed mainly more than 30% of the total depth. More than 90% of the total crater depth was achieved after the 7th drop for both conditions. Therefore, it was decided to limit the number of drops to seven to save time and money. It is important to note that the stratification and nature of soil has an effect on the depth of crater formation. Under pattern I, the thin clay layer was deeper and the prevailing soil type was poorly graded gravel and/or silty gravel compared with that under pattern II. Crater depths under pattern I was also much higher than those under pattern II. For example, while the maximum crater depth was as much as 1.07 m under pattern I (Figure 9), it was only 0.54 m under pattern II (Figure 10), where the thin clay layer was close to the ground surface. Mayne et al. [8], as part of his survey of 124 different sites, reported that the craters were 1-2 m deep, but in our pilot test under pattern II the maximum crater depth was only 0.54 m as mentioned earlier. An important point from this field study is that the thin clay layer seems to act as a damping zone to limit the depth of crater when it is close to the surface.

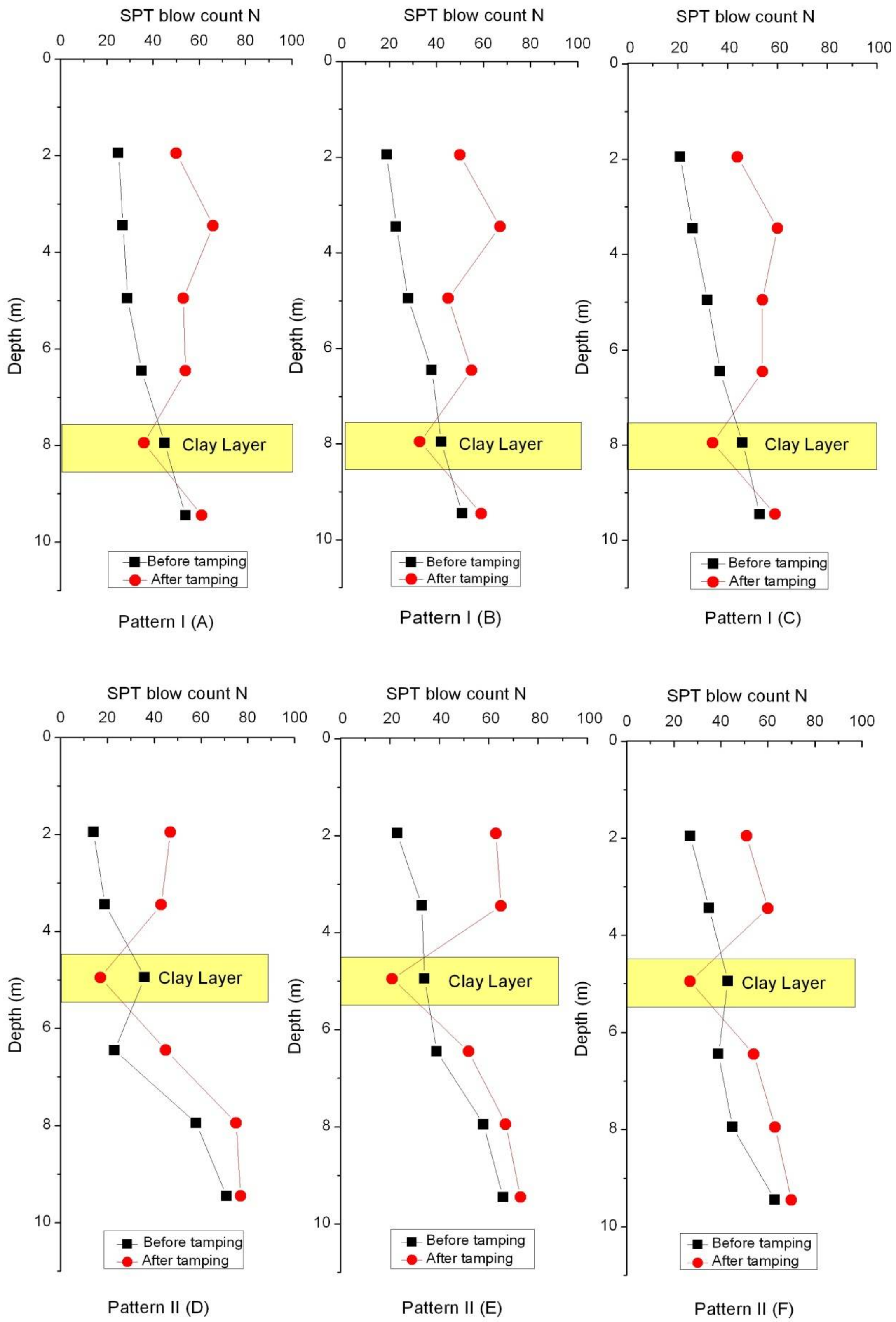

Figure 8. SPT results for pattern I and pattern II (A, B and C are SPT points on pattern I; D, E and F are SPT points on pattern II.)

The average induced settlement of ground surface for this project site was about 8%. For most projects, the induced ground settlement generally ranges between 6-10% of the thickness of the deposit being densified [4].

Ground Heave

The ground surface heave at the mid-point between two adjacent craters and at the mid-point of four craters at five different points (Figure 4) under pattern I and pattern II were measured in the course of tamping. As seen from Figure 11, more than 75% ground heave occurred after the 3rd tamping under pattern I. Similarly, generally more than 80% ground heave took place after the 3rd tamping under pattern II (Figure 12). Although the maximum ground heave (80 mm) under pattern I was slightly higher than that (70 mm) under pattern II, ground heaves generally ranged between 50-80 mm in both cases, which is deemed to be insignificant. These findings indicate that the selection of tamping spacing is reasonable.

Depth of Ground Improvement

The SPT is commonly used to evaluate the effectiveness of DC. In this project, SPTs were carried out under pattern I and pattern II at three different locations (Figure 4): (1) within the tamping point; (2) between two adjacent tamping points; and (3) at the centre of the area formed by four tamping points. For the purpose of comparison, SPT results before and after DC are shown in Figure 8. It can be seen that DC significantly increased SPT-N values in the range of 13-210% at different depths. Generally, the increase was higher near ground surface. DC reduced the SPT-N values of thin clay layers as much as 50%. This might be due to the fact that DC created zones of positive water-pressure gradient which induced water to drain rapidly from the soil matrix. This effect could be further accelerated by the formation of additional drainage paths by shear deformation and hydraulic fracture of stiff clay without volume change. However, it has been stated that clay soil continues to improve for a significant period after treatment [16]. Furthermore, the thin clay layer did not appear to limit the depth of ground improvement since the SPT-N values beneath the clay layer also increased under both cases but the magnitude of increase dissipated with depth. The depth of ground improvement extended up to 9.5 m below the excavation base. SPTs were terminated at this level since it was found in the foundation design of the project that an 8-m improvement depth below the foundation base was sufficient.

Based on the findings, the following conditions, viz. 6-m spacing, 18-m drop height, 15-ton mass and 6 drops at each point with a single pass of high energy phase, are considered to be suitable for improving the whole housing project site by DC because they provided an adequate level of ground improvement, time-saving and cost-effectiveness.

Bearing Capacity and Settlement

After the DC treatment, the allowable bearing capacity of underlying soil below the foundation base up to 8-m depth was increased from 163 to 372 kPa. Similarly, the settlements were reduced from 25 mm to 11 mm.

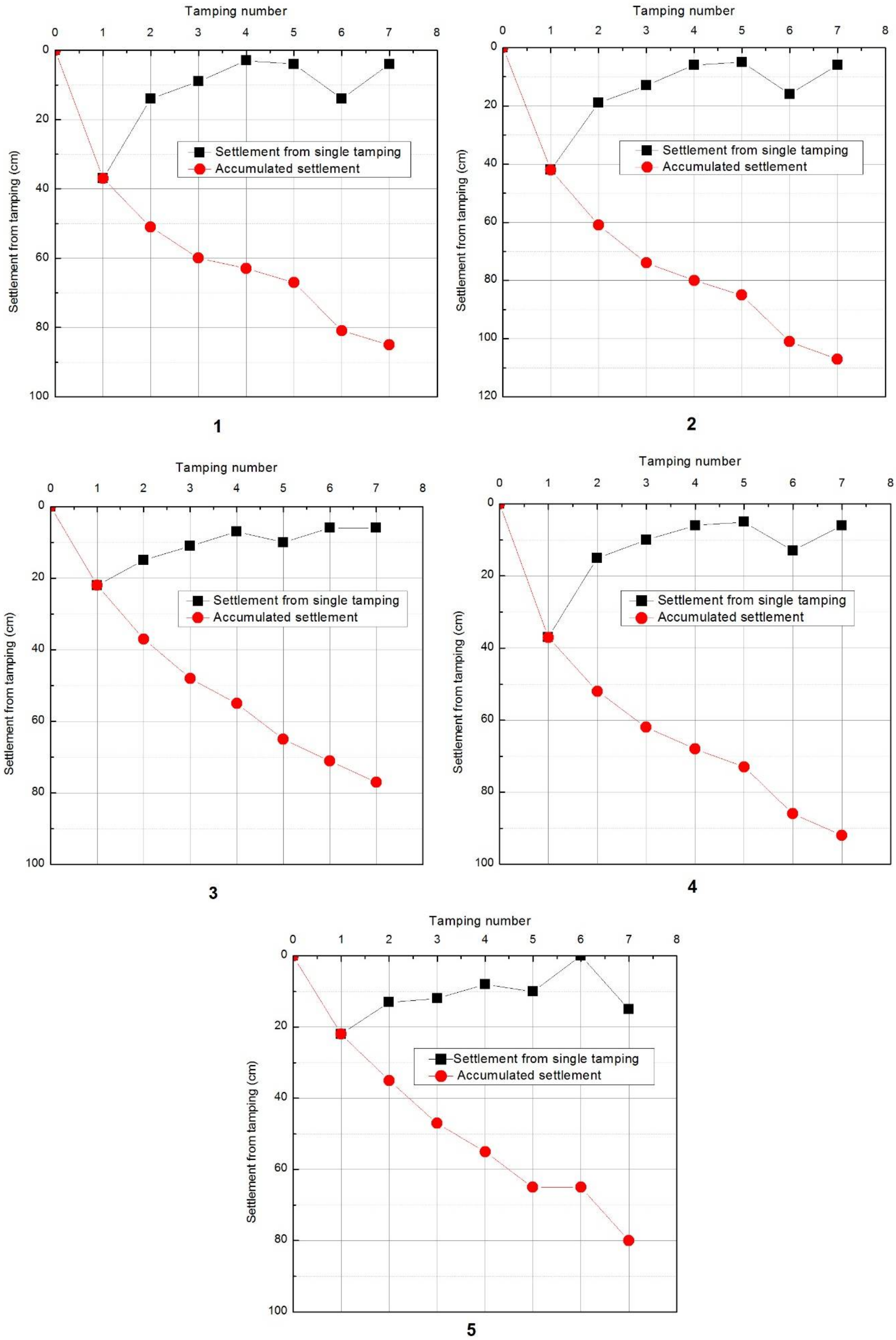

Figure 9. Settlement of craters for pattern I at settlement measuring points 1-5

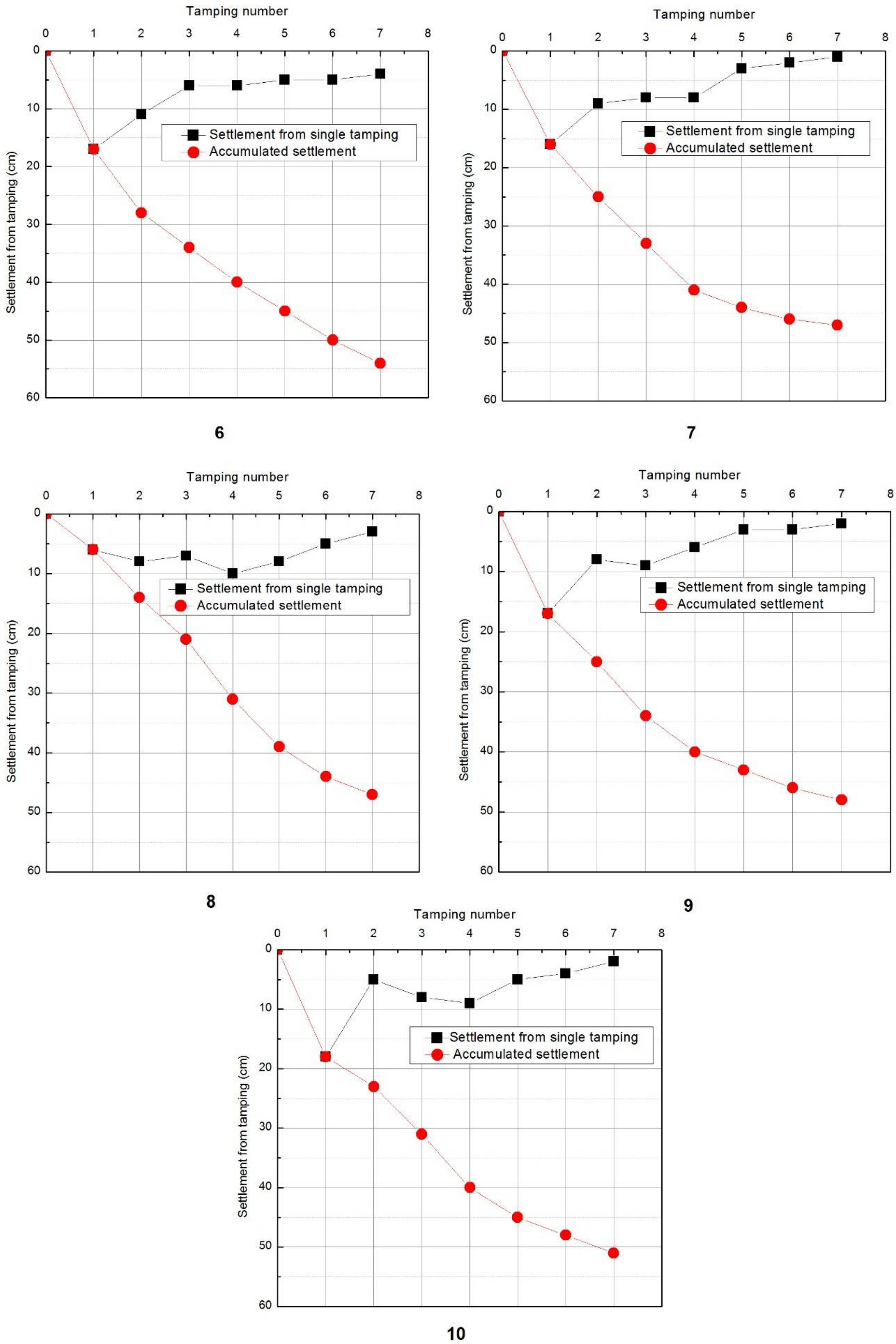

Figure 10. Settlement of craters for pattern II at settlement measuring points 6-10

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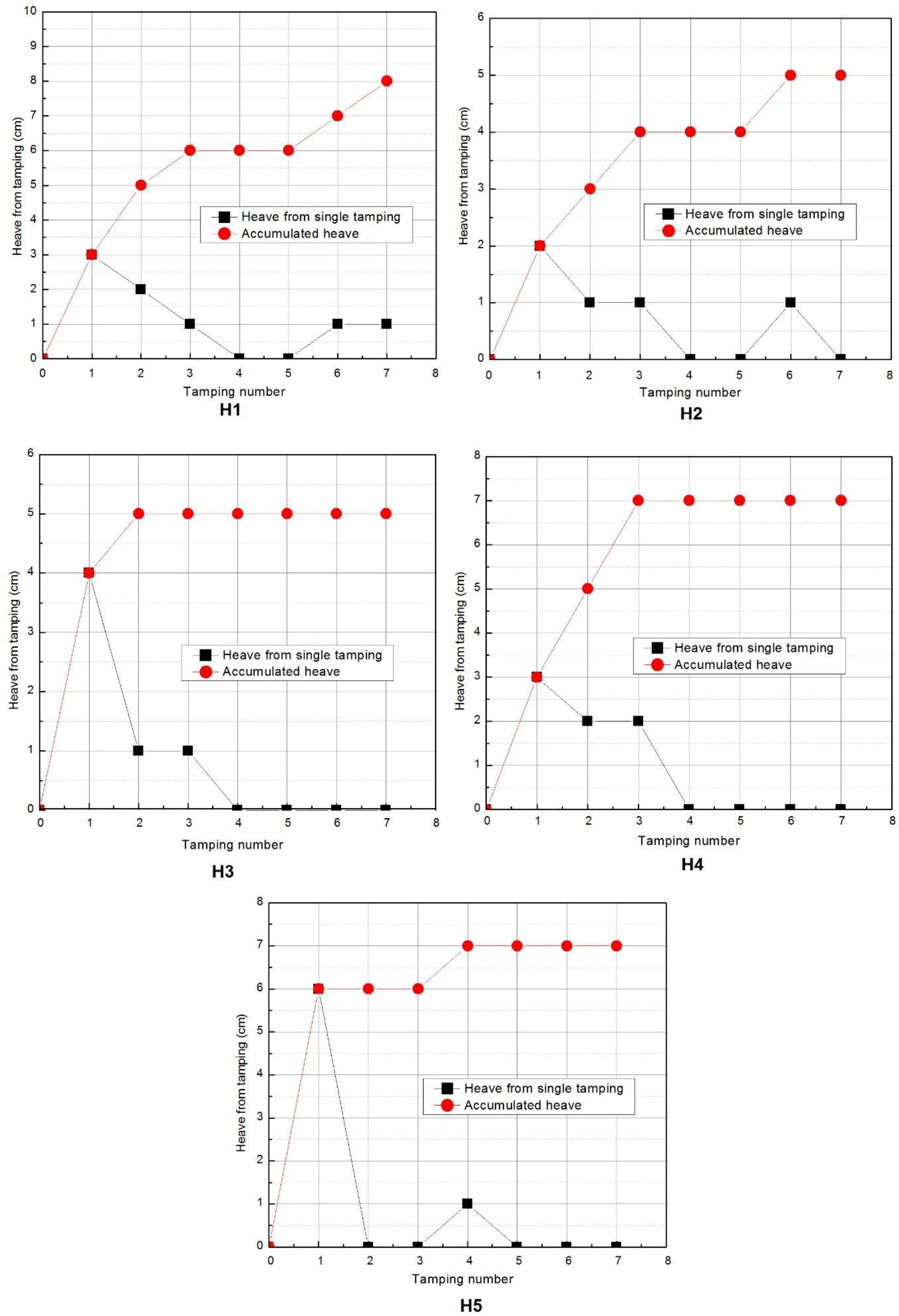

Figure 11. Ground surface heaves for pattern I at heave measuring points H1-H5

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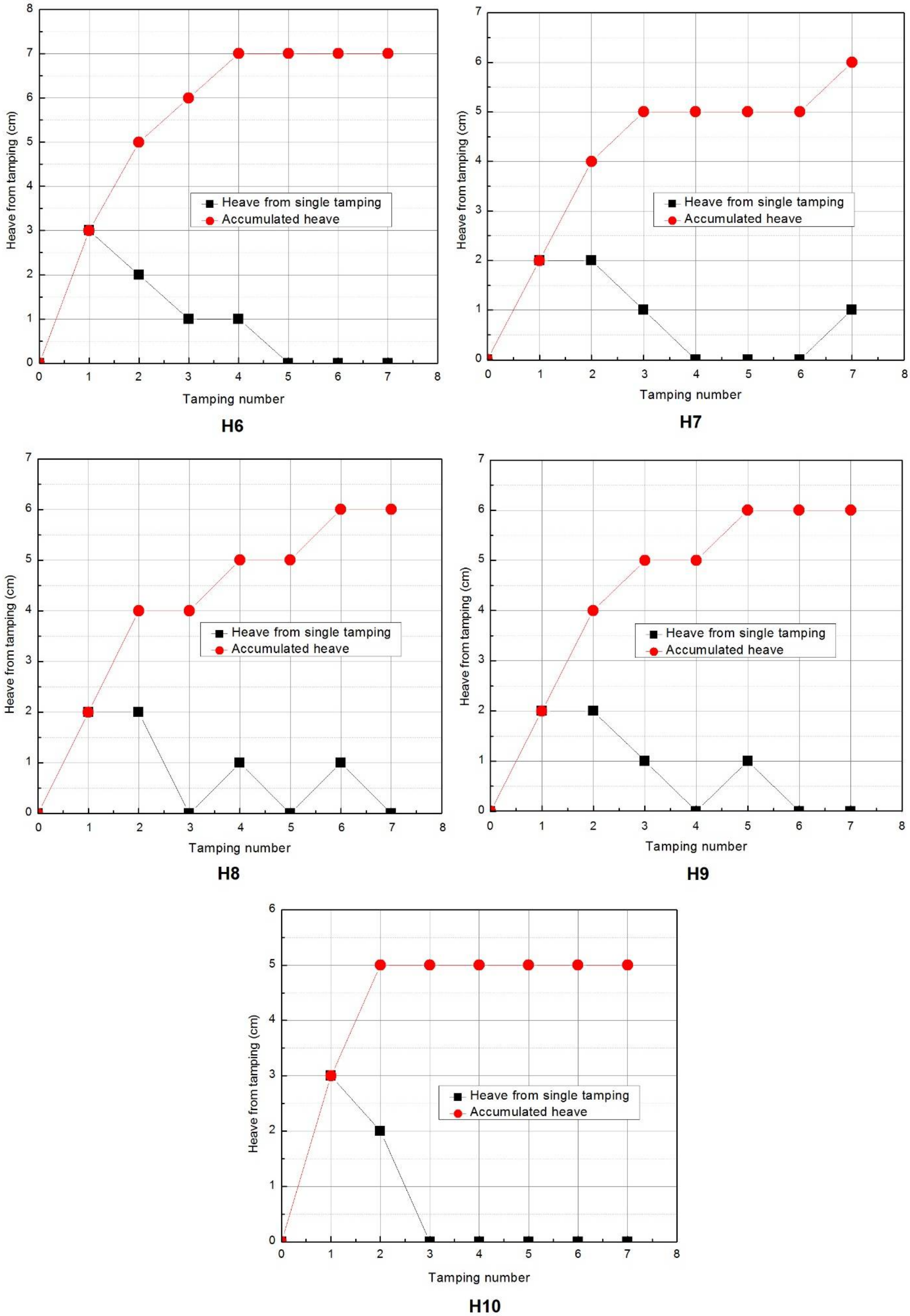

Figure 12. Ground surface heaves for pattern II at heave measuring points H6-H10

CONCLUSIONS

The main conclusions that can be drawn from this study are as follows.

1) DC treatment significantly increases SPT-N values in the range of 13-210% at various locations and depths, being higher near ground surface, but the magnitude of increase diminishes with depth.

2) Although the depth of ground improvement is estimated to be 8.2 m by empirical relation, evaluation tests show that the depth of ground improvement extends up to 9.5 m. This finding stresses that pilot tests should be carried out to determine the exact depth of ground improvement.

3) The bearing capacity of subsoil beneath the foundation base is increased by as much as 128% (from 163 to 372 kPa) and the settlements are reduced by 56% (from 25 to 11 mm). In this way the project requirements are met and the subsoil conditions are brought to a more homogeneous condition, especially from the viewpoint of differential settlements.

4) The thin clay layer present within the granular soil acts as a damping zone to limit the depth of crater formation as well as the ground surface heave when it is close to the ground surface. However, the presence of the thin clay layer does not hinder the depth of ground improvement whether it is close to or far away from the ground surface.

5) The DC for the improvement of predominantly coarse-grained cohesionless soil of Carsamba was experienced to be a fast, efficient and practical method.

ACKNOWLEDGEMENT

The authors acknowledge the financial support provided by Turkerler Holding for this pilot study.

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