Vibrations due to Dynamic Compaction

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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

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ABSTRACT

Ground vibrations due to dynamic compaction (DC), at a harbor site were measured to investigate whether the magnitude of vibrations were unacceptably strong to cause any damage to or increase existing damage on the surrounding structures experienced August 17, 1999 Kocaeli, Gölcük earthquake (M~7.4). The site is located in city of Kocaeli at the seaside of The Port of Derince. It was a reclaimed land formed with Hereke limestones. The measured data included vibrations induced by the tamping energy of DC both at the vicinity of nearby structures and within the DC site. The site was surrounded by a shallow isolation trench all along the land sides. This paper presents the case and the characteristics of vibration such as peak values of the records, Fourier spectrum and amplitude attenuations over distance. The predominant frequencies and amplitude of vibrations were compared to the related code limits to estimate the effect of vibrations on the existing structures.

INTRODUCTION

Dynamic compaction (DC) is one of the methods to improve problematic loose soils. Problematic soils are tamped systematically by a heavy weight dropped from a predetermined height. Energy created in this way results a densification in the underlying soil layers, Mena'rd *et al.* (1975), Thevanayagam *et al.* (2009). The selection of heavy weight, drop height and grid spacing is made individually at each case and can typically vary between 5-35 Mg and 10 m to 40 m and 4-15 m, respectively, Thevanayagam *et al.* (2009). The weight and drop height are not independent variables in practice and a linear relationship exists between them, Mayne, (1984).

Densification of saturated, loose, granular soils increase their bearing capacity, and sand deposits modified by DC are more resistant to liquefaction, and perform well during earthquakes, Mayne (1985). DC is among the most practical and favorite ground improvement techniques. Obviously, it also increases lateral effective stresses. Silty sand deposits appear to densify well during DC when supplemented with closely spaced, pre-installed wick drains, Thevanayagam, (2009), Nashed *et al.* (2009a), Nashed *et al.* (2009b).

On the other hand, ground vibration which emerged during the procedure may cause damage to nearby buildings and other structures such as pipelines and sensitive equipments, as well as upset people living nearby, hence this modification technique should be chosen wisely. It should be kept in mind; ground vibrations caused by dynamic compaction are unique from other types of construction vibrations, such as pile driving, blasting, and traffic as pointed by Hwang *et al.* (2005). They give the results of a field measurement and analysis of ground vibrations during dynamic compaction.

The Port of Derince is located on NAFZ (North Anatolian Fault Zone) (Figure 1) at very end of Izmit Bay, in The Marmara Region of Turkey of which NAFZ passes through roughly along the centerline (Figure 2). The epicenter of August, 17, 1999 Kocaeli- Gölcük Earthquake (M~7.4) was 3 km away from the port, Table 1, Figure 3. The building structures as well as coastal structures experienced heavy damage during August, 17, 1999 Kocaeli Earthquake of which surface wave magnitude 7.8 (UGS) and moment magnitude 7.4 (USGS, Kandilli), Yüksel *et al.* (2005). Table 1 shows five of the important coastal facility with their type of foundation, degree of damage after the earthquake, operation condition under the earthquake damage and the distance between the coastal structure and the epicenter of the earthquake as given by Yüksel *et al.* (2005).

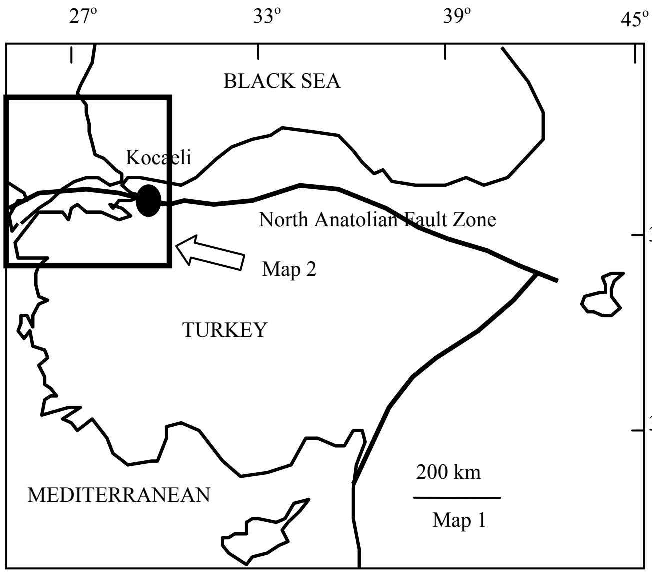

Figure 1. NAFZ (North Anatolian Fault Zone) and Marmara Region (denoted as Map 2).

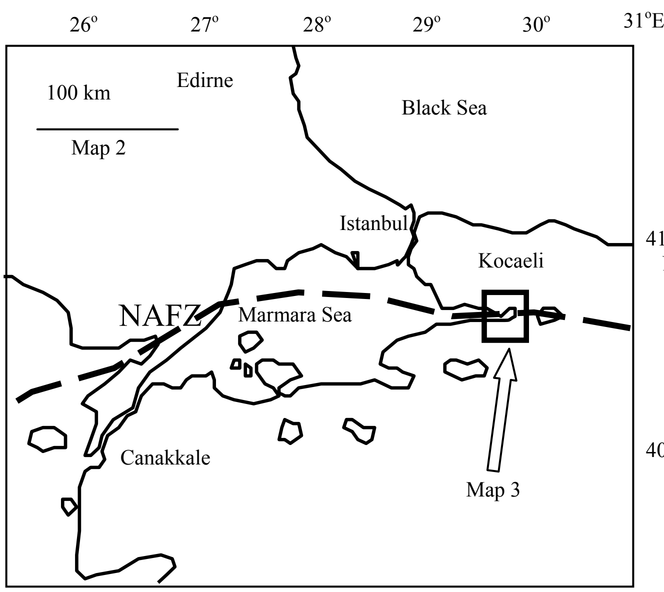

Figure 2. The Marmara Region and the location of Derince Harbor (denoted as Map 3).

Table 1. Degree of damage to the Derince Harbor and some coastal neighboring structures during the 1999 Kocaeli-Gölcük Earthquake, after Yuksel et al. (2005).

	Title	Foundation type of port and docks	Degree of damage	Operation Condition	Epicentral Distance (km)
1	Tüpraş Coastal Facilities	RC and steel piles	Medium	Partial Operation	5.5
2	Derince Port	RC piles	Medium	Partial Operation	3.0
3	Petrol Ofisi Port	RC and steel piles	Heavy	Partial Operation	4.5
4	Shell Derince Port	Steel piles	Heavy	Out of Operation	5.0
5	Koruma Tarım Port	RC piles	Heavy	Out of Operation	5.5

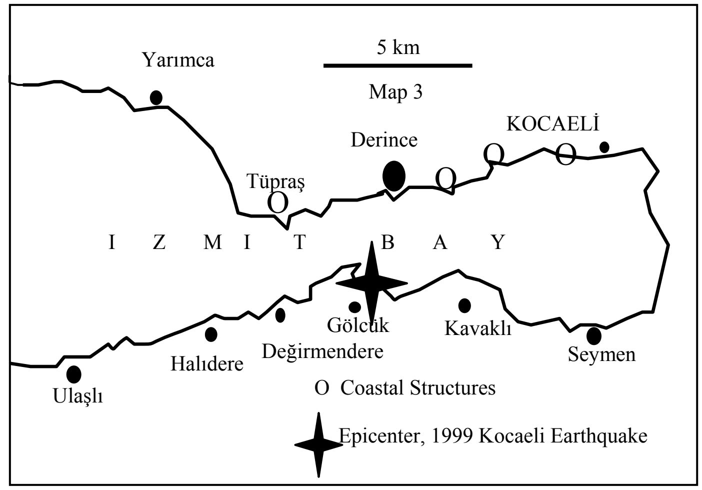

Figure 3. Derince Harbor and other neighboring coastal structures experienced medium to heavy damage during August 17, 1999 Kocaeli-Gölcük Earthquake.

The port of Derince experienced medium damage during the earthquake, as given in Table 1. After the earthquake the facility resumed its operation partly. The distance between the epicenter and the port is 3 km. It is located within the epicentral region during the earthquake. The type of foundations of the ports' coastal structures is reinforced concrete (RC) pile.

As ground vibration induced by DC may affect the structures located around the construction site and may increase the existing earthquake damage, a detailed damage survey has been conducted and reported by Özer *et al.* (2002) at the onset of DC.

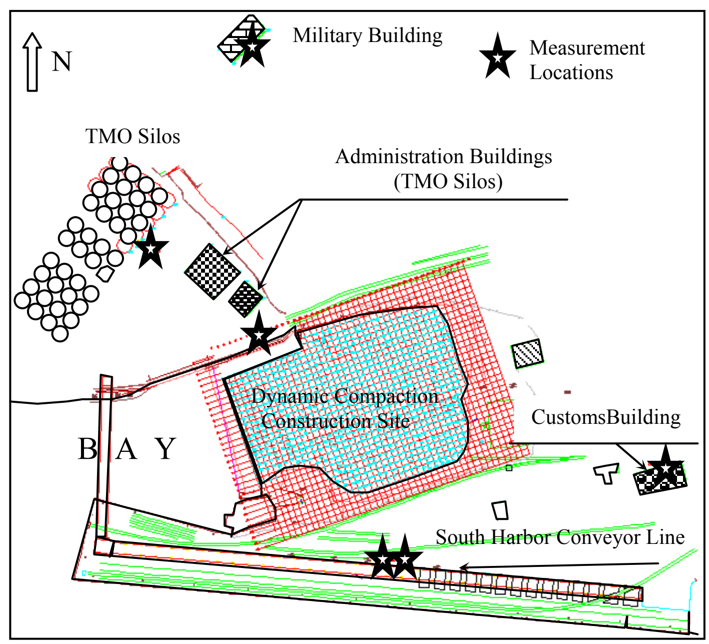

Figure 4. The locations of the neighboring buildings with respect to DC site and measurement places.

The ground vibration induced by DC of the construction site are carefully monitored, and analyzed. The DC site was located within the Derince port (Figure 4). The Structures monitored were silos and administration buildings of Turkish Grain Board, Military Building, Customs Building and South Port Conveyor Line. The locations of these buildings with respect to the DC site as along with measurement locations are given in Figure 4.

The followings are information on the surrounding structures, a summary of site condition, characteristics of construction, measurement and the characteristics of ground vibration.

SURROUNDING STRUCTURES

Structures around the DC site are shown in Figure 4. Turkish Grain Board silos and their administration building were located at the north-west of the DC site. Turkish Grain Board silos, constructed on friction piles, are made of reinforced concrete and have a cylindrical cross section and a height of 42.5 m.

As can be seen from the silo details in Figure 4, middle part of the silos were constructed separately from the sea and land sections; however they are all constructed on the same pile cap. The measurements at the silos were taken at the middle section on the pile cap and on the roof.

The vibration measurements at Turkish Grain Board administration building were taken both at the foundation level and at top of ground floor. The administration building had pile foundations and located in the north-west of the silos yet closer to the DC site. The administration building is a two storey RC building. It has raised floors build in order to manage computerized control systems.

The construction of Turkish Grain Board silos and administration building were completed between 1994 and 1996. Both structures have a high quality standard in their design and construction and survived the earthquake with minor damage.

Located south of the DC site, South Port Conveyor Line is a two storey steel structure and only the second floor is covered by brittle walls.

On the far north side of the DC site, a three storey conventional type of RC army buildings with brittle partitioning walls were located.

All the buildings survived 1999 Kocaeli-Gölcük Earthquake with minor damage. However, the customs building with a half basement, one ground floor and two storey located east of the DC site, was experienced some damage and strengthened afterwards.

FEATURES OF GEOLOGY AND CONSTRUCTION

In this section, information about the site conditions, reclaimed landfill and DC are given, yet detailed information on the site conditions may be found elsewhere (Zemin Etüd ve Tasarım A.Ş., 2001)

Local Geology and Soil Condition

Quaterner aged sediments underlay the engineering structures all along the shores of The Izmit Bay. Geotechnical characteristics of the sediments along the coasts of the bay was examined by dividing the coast in three regions, even though each show slight differences due to the local morphology, Yüksel et al 2005. One of these regions summarizes the properties of local soil conditions all along the coastal line from Yarimca to Değirmendere. The characteristic soil profile consists of Hologene aged, actual sea bottom sediments at the top which is a very soft to soft (N-SPT blow counts = 0-4) clay layer of which thickness vary between 5 to 25 m. and increase with an increase in sea depth. The thickness of the Holocene aged sediments under the coastal structures is between 20-30 m. This soft layer is underlined by Piliocene-Pleistocene aged, old medium stiff to hard clay and/or medium dense to very dense sand layers which are capable of carrying structural loads with foundation systems in the form of concrete or steel friction piles. During 1999 earthquake almost all of the end bearing pile foundations were collapsed because of the tip soil failure. However, friction piles survived with much less damage, Yüksel et al 2005. The shear wave velocity of the very soft to soft clay on the top is much lower than 180 m/sec. The shear wave velocity of the Piliocene-Pleistocene aged layers are generally between 300-700 m/sec. These values are approximately converted from soil descriptions. The topography around the test site is rather flat and surface elevations varying around +5.50m.

Reclaimed Landfill

Hereke limestones were used in the construction of reclaimed landfill. The sea fill effort was started at depths between 4.0-6.0 m with 0-40 cm Hereke limestones and continued up to an elevation corresponding to the sea level. A second layer of 0-25 cm size Hereke limestones was overlaid in 1.20 m thickness. Filling was completed with a final layer of crushed stone having a 0.5 cm thickness. A mesh with a size of 4 m x 4 m was marked on the surface of 130 m x 160 m reclaimed landfill to provide systematic conduct of DC (Figure 5).

Dynamic Compaction

The job at the test site consisted of tamping and then ironing the surface. The tamping was carried out with a 12.5-ton weight falling from a height of 22 m. The tamping was carried out in three phases. The impact number of each tamping point was five with ironing after each phase.

Vibration measurement was performed only for tamping of the grid point close to each structure because its tamping energy is higher than the tamping of the other points in the grid, thus having a greater influence to each of the structures surrounding the DC site.

Trench

Before the tampings started, three sides of the DC site of 130 m x160 m were separated by trenches to prevent Rayleigh wave propagation to the surrounding area (Figure 5). The tamping points were selected far enough from the trench line to prevent probable debris generation on trench sidewalls.

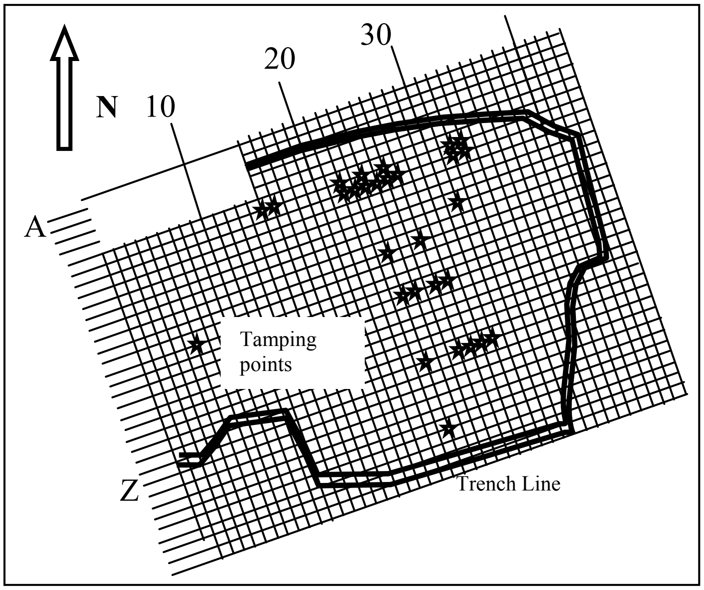

Figure 5. Location of trench line and tamping points used in this study marked on 4m x 4m mesh within DC construction site.

VIBRATION MEASUREMENT

The vibration measurements were carried out in three phases denoted with their dates. Phase 1 measurements were carried out at the onset of first 5 tampings on Oct.5, 2002. Then, Phase 2 measurements were taken on the completion of first 5 tampings on Oct.9,2002. Finally, Phase 3 measured on Oct 23, 2002 when the second pass of the 5 tampings and ironing were completed. It should be noted that, some measurements were taken within the DC site.

Equipment

Simultaneous recordings achieved using two different measurement systems for the purpose of validation of both records. One of the equipments consists of small vibration measurement systems and recorders owned and run by İstanbul Technical University, Civil Engineering Faculty Earthquake Engineering Laboratory. The other recording equipment was B&K4376 measurement system owned and run by İstanbul Technical University, Mechanical Engineering Faculty.

CHARACTERISTICS OF DATA

Velocity and /or acceleration measurements in horizontal and /or vertical directions were taken by both of the recording systems, simultaneously. Vibrations were recorded both at the ground floor and other floors of the buildings.

The recordings were kept in memory disks. Points of energy application and measurement, direction of recordings, name of data files, type of records, number of points in each record as well as the maximum amplitudes determined are collected in tables and given in the report, Karadoğan *et al.* (2002).

The peak value of particle velocity is generally a measure in the evaluation of the levels of vibration emanating from an impact source to the surrounding area. The peak values of particle velocities are given in Table 2.

Table 2. Peak values of particle velocity

Measurement Location	Peak Particle Velocity (mm/s)		
	Date of measurement		
	Oct 5, 2002	Oct 9, 2002	Oct 23, 2002
Basement of Silos	0.80	0.90	0.90
Harbour South Conveyor Line	1.92	1.80	--
Military buildings	--	0.64	2.32 vertical
Customs building	--	0.57	--
Silo, landfill site	--	2.55	--
DC site	3.47	--	--
Silo administration building	--	--	2.20 vertical

Typical velocity records

Fig. 6 and Fig. 7 show a sample of the recorded radial and vertical velocity-time histories, respectively, at the sea side of the construction site along the line of conveyor footings at various distances as 66 m, 85 m, and 124 m from the tamping points AC24, W24 and M24, respectively, under standard DC energy (22 m x 12,5 tonnes).

Fig. 8 and Fig. 9 show the recorded maximum, median and minimum radial and vertical velocity-time histories, all around the DC construction site at distances 66 m, 124 m, and 189 m from the tamping points AC24, M24 and K31, respectively, under standard DC energy.

The DC load impact produce these wave forms. These are typical forms under an impact load as can be seen in Hwang and Tu (2006). It is possible to separate high and low frequency wave forms and to follow their attenuation with distance. All the vibration durations are within 0.5 to 1.0 sec. however, durations are reported shorter as 0.1–0.3 s for each direction by Hwang and Tu, 2006.

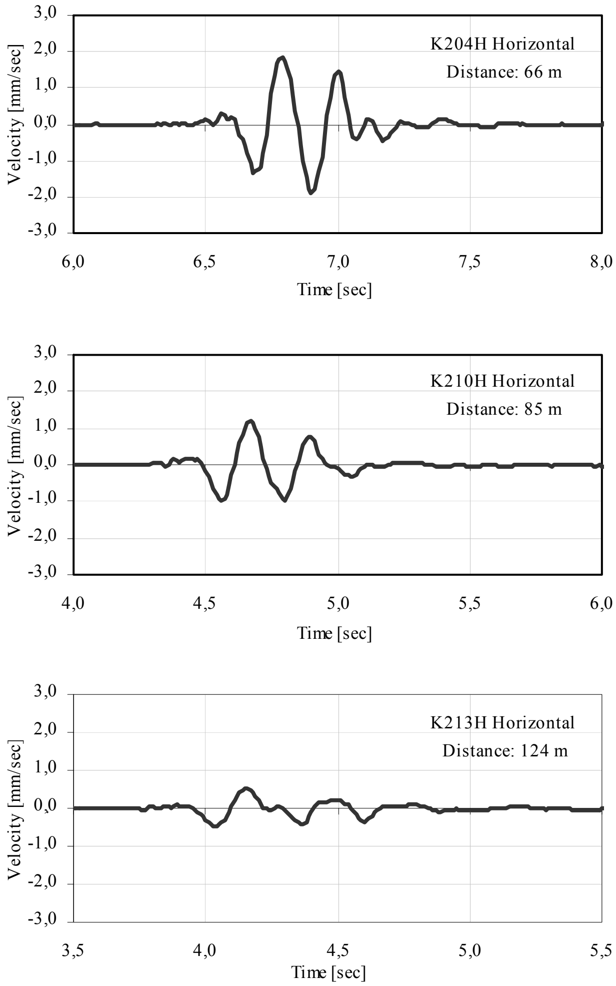

Fig. 6. The radial velocity histories at different distances under standard tamping energy.

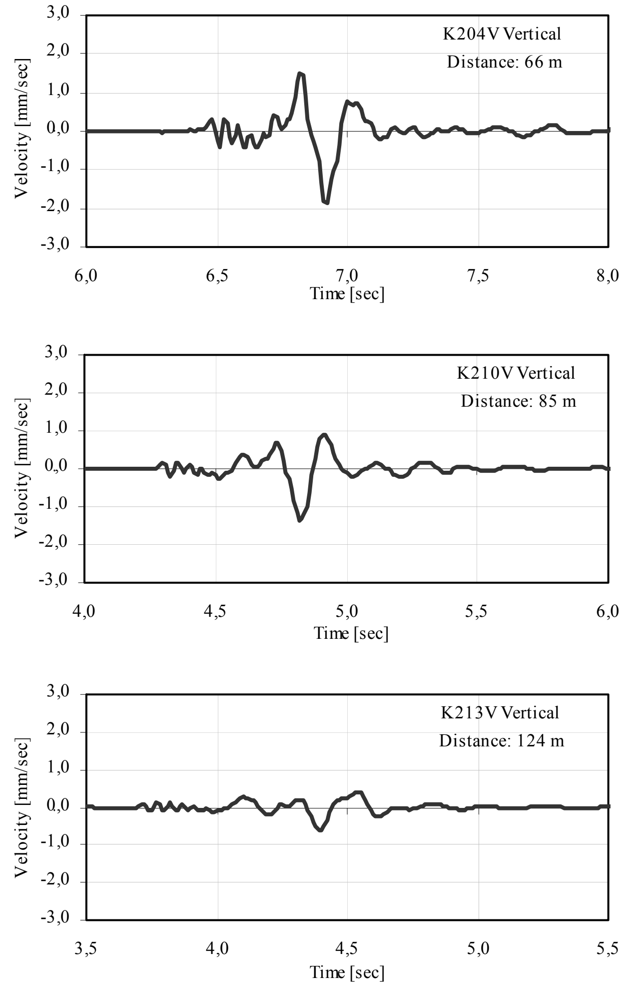

Fig. 7. The vertical velocity histories at different distances under standard tamping energy.

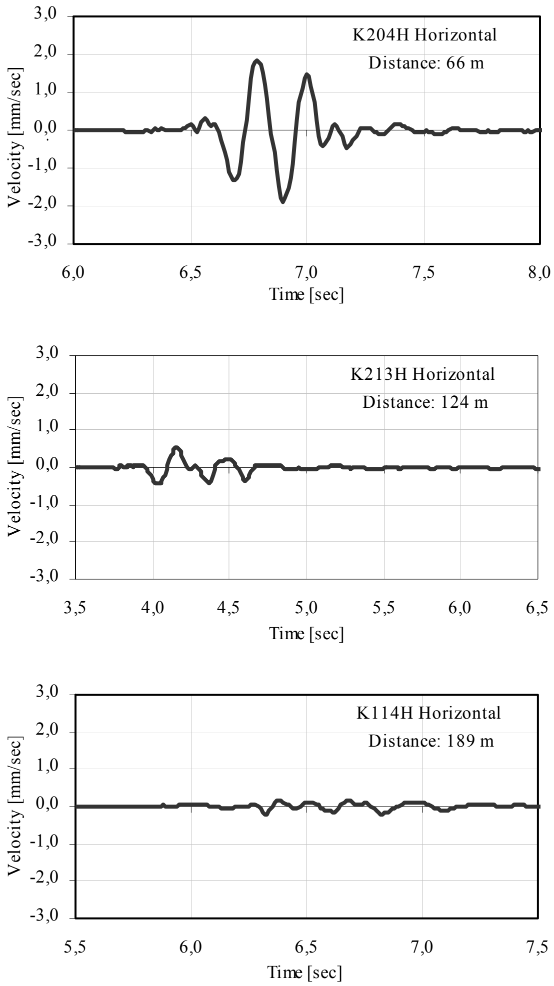

Fig. 8. The radial velocity histories at minimum, median and maximum distances under standard tamping energy.

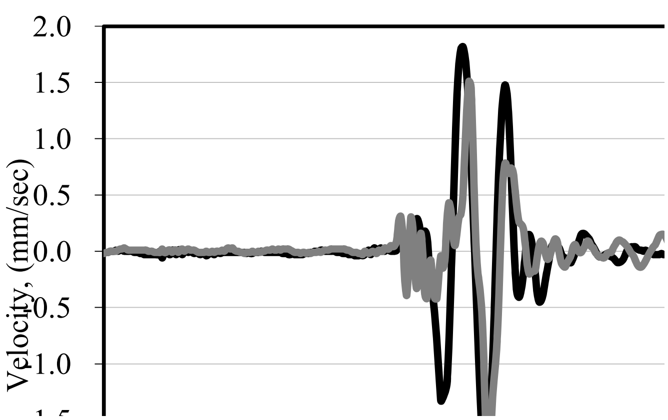

Fig. 10. The comparison of the velocity wave forms in radial and vertical directions at 66 m distance from tamping point (Record No: K204H and K204V).

Fig. 10 shows a comparison of the velocity wave forms in radial and vertical directions at 66 m distance from the tamping point. It can be observed from the figure that the vertical and radial velocity amplitudes are very close to each other. This similarity was also reported by Hwang and Tu (2006). They also reported that the velocity amplitudes in radial and vertical directions are far greater than that of tangential direction. Throughout this study, only radial and/or vertical velocity measurements were utilized. Despite a few velocity measurements in tangential direction were taken yet they were not used for the main purpose of this study.

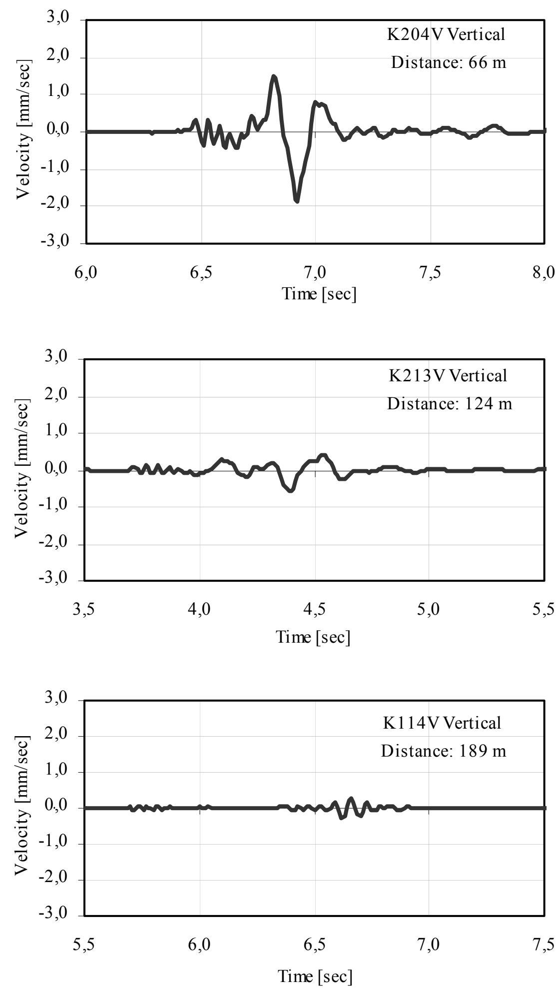

Fig. 9. The vertical velocity histories at minimum, median and maksimum distances under standard tamping energy.

Fourier spectra

Fig. 11 and Fig. 12 shows the Fourier amplitude spectra of velocity records in radial and vertical directions at various distances from the tamping point under standard tamping energy. As these records are almost in the same direction, their frequency contents are similar. Fig 13 and Fig 14 shows the Fourier amplitude spectra of velocity records in radial and vertical directions taken at minimum, median and maximum distances from the tamping point under standard tamping energy. The Fourier spectral shapes obtained different directions at the same distance are considerably different, Hwang and Tu, (2006). As the last figure in an entirely different direction its frequency content is obviously different. The spectrum at far distance hasn't got a clear predominant frequency band and a corresponding peak. The predominant frequency of radial and vertical vibrations are similar and in the range of 3-8 Hz. Fig. 15 is a comparison of Fourier spectra in radial and vertical directions. They show some differences but their predominant frequency ranges are similar. The spectrum in vertical direction has a low frequency peak.

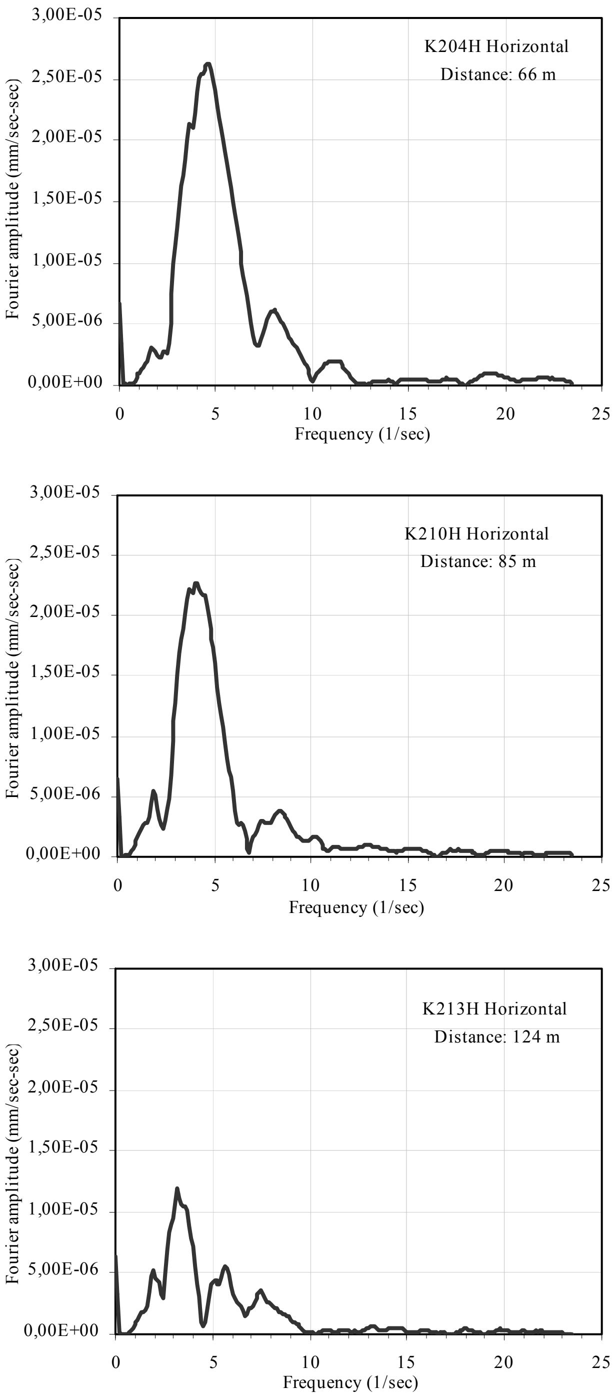

Fig. 11. The Fourier amplitude spectra in radial directions at various distances under standard tamping energy.

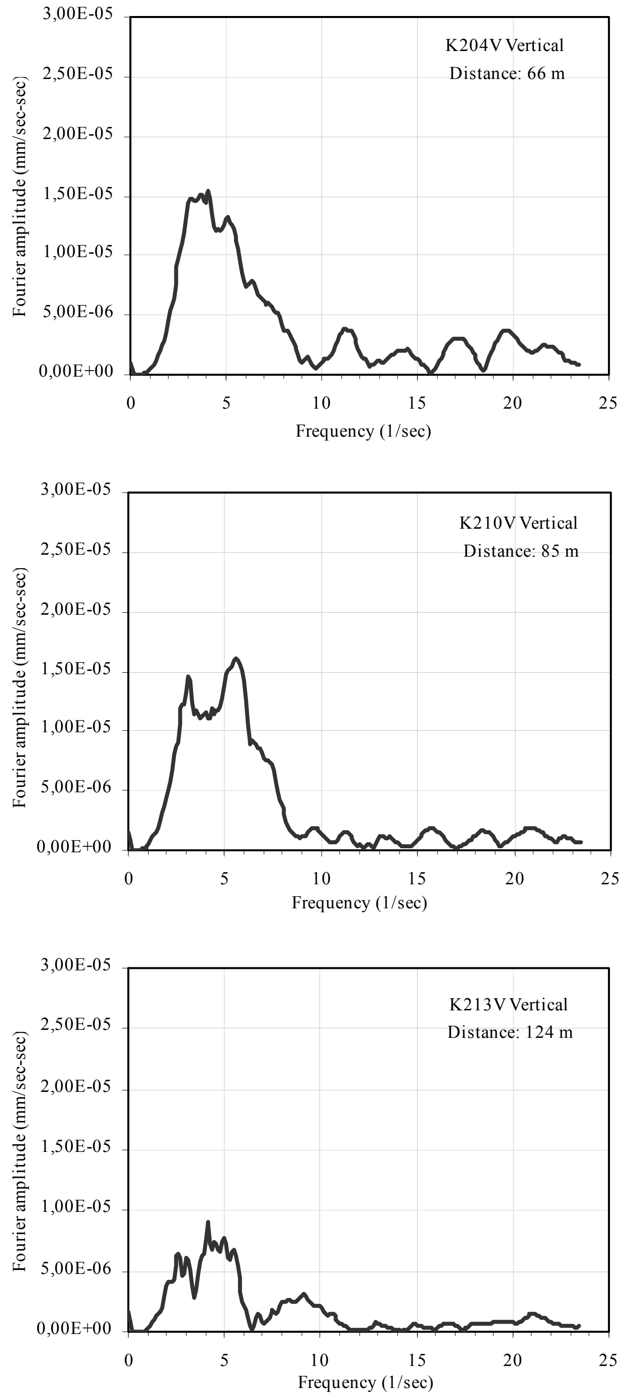

Fig. 12. The Fourier amplitude spectra in vertical directions at various distances under standard tamping energy.

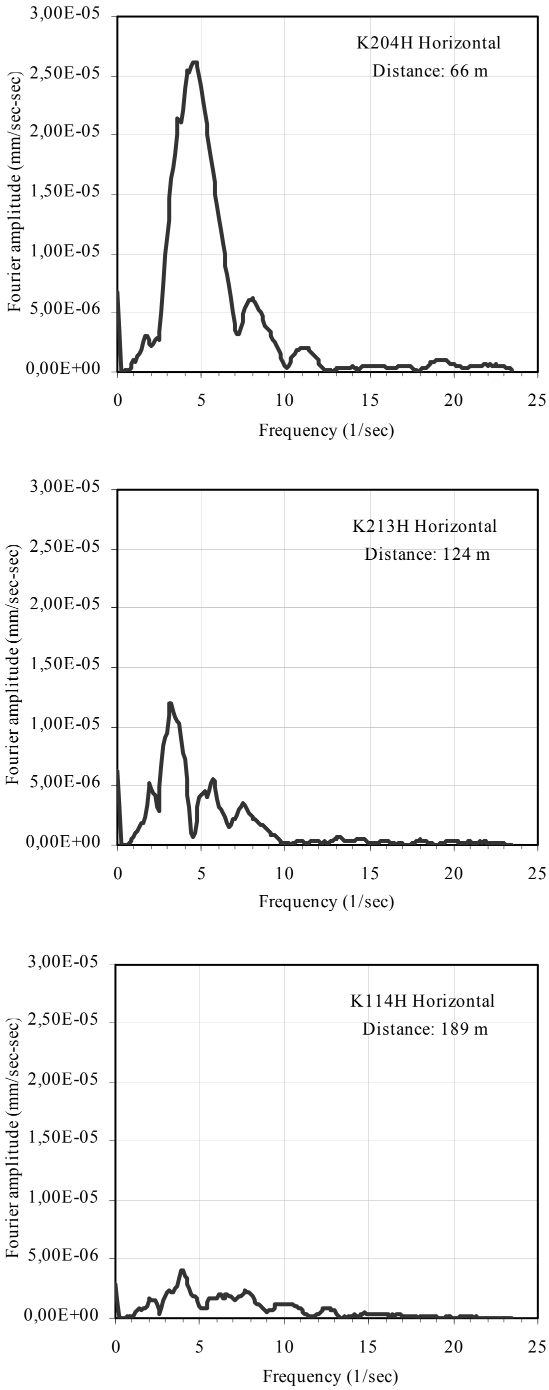

Fig. 13. The Fourier amplitude spectra in radial directions at various distances under standard tamping energy.

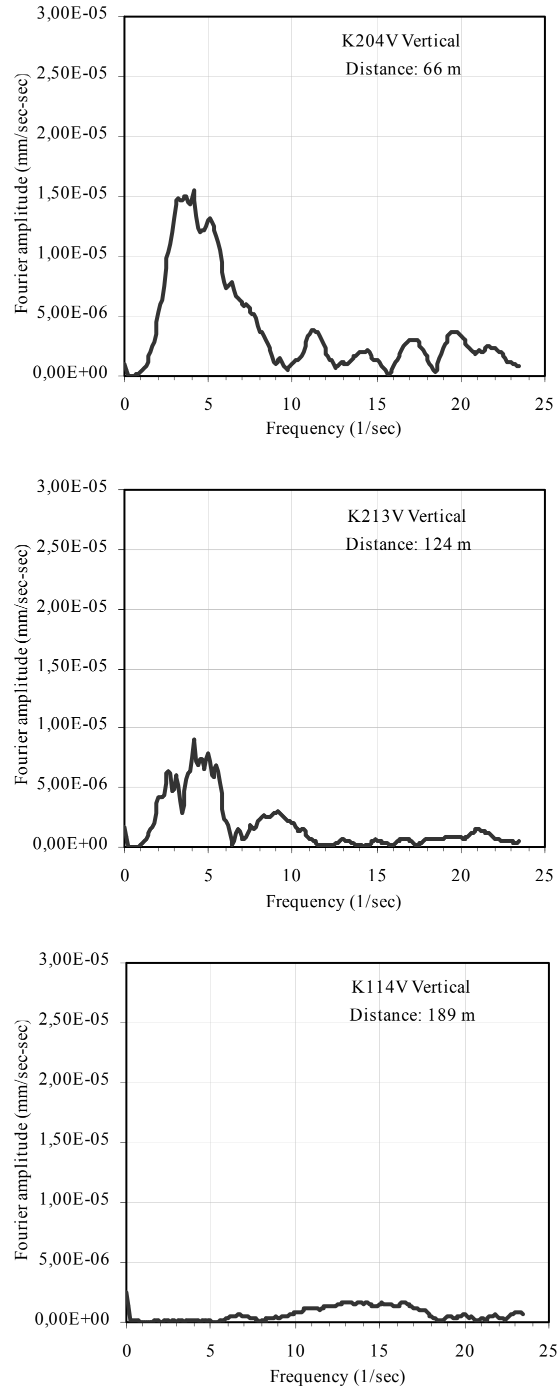

Fig. 14. The Fourier amplitude spectra in vertical directions at various distances under standard tamping energy.

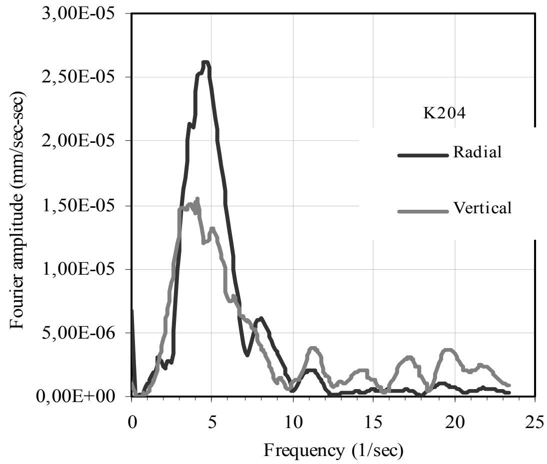

Fig. 15. A comparison of the Fourier spectra in radial and vertical directions.

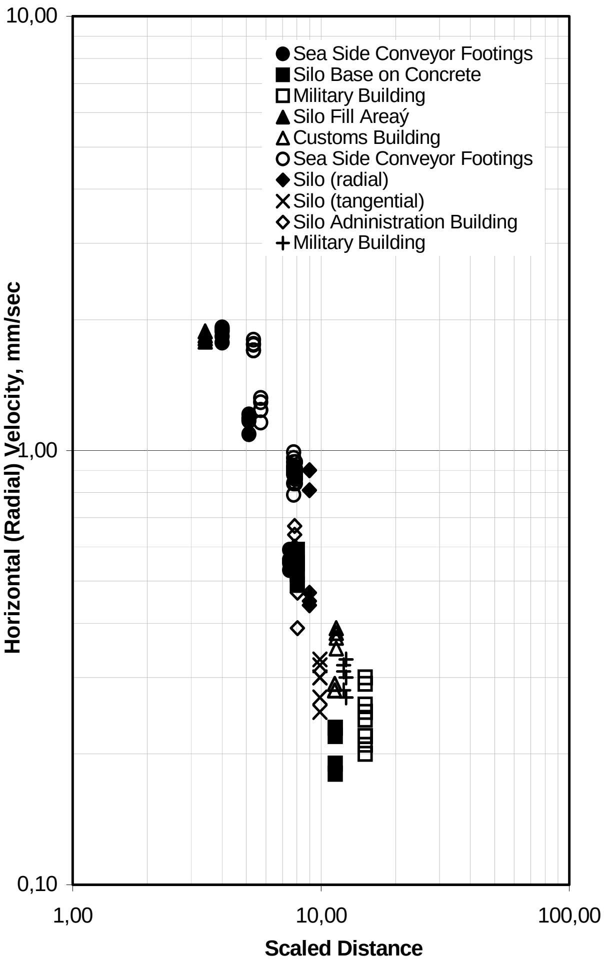

Fig. 16. The attenuation relationship of PGV with distance in horizontal (all radial but five-tangential) directions.

Attenuation of PGV with scaled distance

Fig. 16 shows the attenuation relationship of PGV on a scaled distance plot. A scaled distance is used since some points produced by using reduced tamping energy. The reduced tamping energy was produced using half of the falling height i.e. 11 m. It can be seen from Fig. 16, most of the data points are very close. However, some dispersion exists, which can be attributed to the change in soil characteristics and/or underground structure as groups of data points obtained different directions. In Fig.17 attenuation relation is given with other DC project's attenuation relations on a scaled distance plot. From this figure it can be seen that, PGV's produced in this study are rather low. The data points show a rather steep overall attenuation relation.

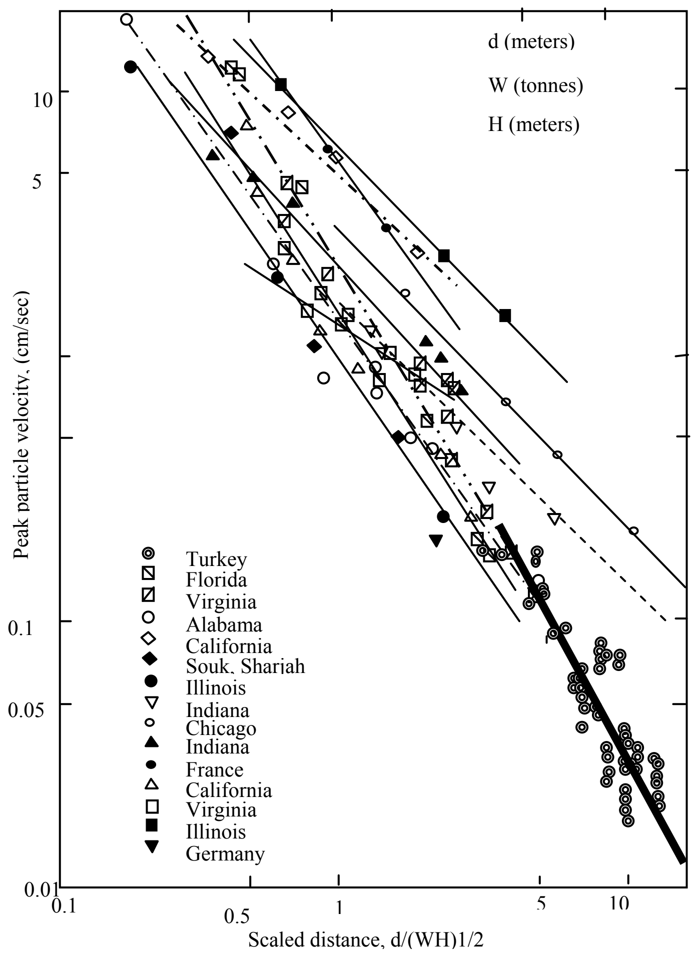

Fig. 17. The attenuation relationship of PGV with scaled distance in horizontal (all radial and five-tangential) directions, Data points plotted on a figure given in Mayne (1984).

TRESHOLD VALUES FOR VIBRATIONS

The frequency of vibrations emanating to the surrounding area during a DC generally varies between 2 and 12 Hz, Fang (1991). Predominant frequencies of these vibrations are reported to be either between 3-4 Hz or 6-10 Hz, Menard et al (1975). In a study conducted by Hwang and Tu, (2006), the primary frequency of vertical vibration is found to be in the range of 10–20 Hz, while the radial vibration has two primary frequencies, one is in the range of 3–4 Hz and the other is in the range of 12–13 Hz.

In this study, the predominant vibration frequencies were calculated by fast Fourier transform utilizing three representative records. These records are South Conveyor Line dated Oct 9, 2002, Military Building dated Oct 23, 2002 and Turkish Grain Board Silos Administration building on Oct 23, 2002. The predominant frequencies of these records are 3.7 Hz, 5.35 Hz and 6.7 Hz, respectively. These frequencies will be used for the assessment given in the following section. The predominant frequencies of the Fourier amplitude spectra given in Figures 11, 12, 13 and 14 vary between 3.25 Hz. and 5.75 Hz.

The threshold values for maximum particle velocities of ground vibrations within these frequency range, to cause damage to various structures are given in the codes. For example, in the German standards (DIN 4150- Now 3, May 1986) which is known to be on the safer side compared to other codes, the threshold velocity for industrial or similar structures is 20 mm/s and the threshold velocity for residences and similar structures are as low as 5 mm/s.

Similar threshold values for particle velocities which can cause damage could have been defined independent of type of structures, FHWA (1995). The particle velocities which are not high enough to cause crack development on the secondary members of structures such as separation walls, are required not to exceed 5 mm/s and 13-19 mm/s for the frequency band 2-12 Hz approximately.

Foundation system of structures, type of foundation soil, level of ground water, actual condition of structure and structural features are amongst the factors that should be kept in mind during the general assessment of the structures.

CONCLUSIONS

Velocity, acceleration and displacement records at radial and vertical directions were taken during three stages of DC at the Derince Harbor which was then owned by the government and recently was sold to a private company.

From the findings of the study following conclusions may be drawn:

Measurements taken by two separate recording systems are coherent. To justify this coherency between two measurements was important as they show that the measurements taken were correct.

Neither horizontal nor vertical velocity values recorded around the structures during each stage of DC exceeded 2.32 mm/s. The peak velocities measured in the landfill site and DC site are 2.55 mm/s and 3.47 mm/s, respectively. This level of velocities are considerably low to cause or increase any damage to structures, since they all are much lower than the threshold values for damage. The frequency of vibrations obtained in this study, required to use the threshold curves for damage are obtained by processing some of the vibration measurements. For example, the frequency determined from the records taken at one of the foundations of South Conveyor Line on Oct 9, 2002 was 3.7 Hz. The frequency determined from the records taken at Turkish Grain Board administration building on Oct 23, 2002 was 6.7 Hz. and the frequency determined from the records taken at the military building on Oct 23, 2002 was 5.35 Hz. These magnitudes of frequencies are within the expected frequency range that is 2Hz-12Hz frequency band.

Maximum acceleration of all the records is 271 mm/sn^2. This value is 28% of the acceleration, 0.1g=980mm/sn^2 which is the value suggested to use in the design of new buildings located in the areas with the lowest seismicity by the earthquake resistant design regulations. The level of accelerations induced by DC is too low for any structural damage within the surrounding area. On the other hand, the buildings examined are subjected to the 1999 Kocaeli-Gölcük earthquake with a maximum acceleration 0.5g = 4900 mm/sec, longer than the vibration duration of DC and they survived without any significant damage, Özer et al (2002).

In summary, the acceleration level occurred during the DC work was not as high as to cause damage on structures. In other word, it is impossible for that level of accelerations could bring the structures to the upper limit of elastic behavior.

The maximum velocity and calculated rms velocity of a representative record taken at the administration building of the silos are 2.11 mm/sec and 0.92 mm/sec, respectively. As the velocity is smaller than 3 mm/sec limit of ISO Guidelines, it can be expected that the vibrations from DC work are unable to give any damage even to very sensitive equipment in the administration building.

Attenuation of peak particle velocity in radial and vertical direction in the region is established and compared to worldwide data. A relatively steep attenuation relation for radial velocity is obtained.

ACKNOWLEDGEMENTS

The authors would like to extend their sincere gratitude to AYTAÇ Construction Company Pty. Ltd. for the limited financial support. Also, the authors thank Research Assistants Pınar Teymur and Gülşah Sağbaş for their valuable contributions to this study.

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