LIQUEFACTION MITIGATION USING RAMMED AGGREGATE PIERS (RAP)\(^1\)

ABSTRACT

This paper is a digest of the studies and design that went into the decision to select the use of Rammed Aggregate Piers (RAPs) for the required ground improvement for a high value shipyard in Altinova, Turkey to address the liquefaction risks and very high Design Seismic acceleration of 0.85 g at a reclamation site. Aside from the relatively poor soils underlying the site, reclamation fill to heights of 12 meters would be required in order to reclaim the seaward portion of the Facility for the Synchrolift as well as the fabrication yard. To compound the situation relatively very high Design Ground acceleration of 0.85 g were required as well as very stringent requirements for maximum tolerable vertical deformations and horizontal spreading. The project was a great design challenge that was met by the authors through the use of detailed computerized analyses and the innovative use of proprietary Ground improvement measures in the form of Rammed Aggregate Piers.

This digest was directly lifted from the original studies made by the original authors 5 July, 2005 and additional background material gathered by the presenter. Any mistakes, omissions and typographical mistakes are entirely the responsibility of the Presenter.

INTRODUCTION

A shipyard is to be built in Altinova, Turkey which is partly to be reclaimed from the sea. The site soils both for the inland portion as well as the sub-seabed soils are relatively very poor and potentially liquefiable.

The Project is a State-of-the-Art Shipyard with a total area of 250,000 m\(^2\) and a uniform floor slab pressure of 150 kN/m\(^2\). 160,000 m\(^2\) of open sea are reclaimed with up to 12 m of granular fill. The ground improvement with Rammed Aggregate Piers is designed to limit total settlements and resist peak ground accelerations of up to 0,80g.

Soil Profile: Up to 20 m of very soft to medium stiff, silty sands and sandy clays. The water table is near the ground surface.

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**Fig 1.0 Layout of Ship Lift and Auxiliary Areas**  
Design Details: 68,000 Impact Piers to depths of up to 18 m were installed to prevent liquefaction and limit settlements. Settlements from the fill are expected to be 90% complete within 3 weeks after Rammed Aggregate Pier Installation. Modulus Load Test Results show a maximum of 2.5 cm of deflection at design stress level.  
Owner: Bogazici Shipyard, Turkey

This digest paper summarizes the studies, and the results of the extensive analyses undertaken by the Original Authors in order to mitigate the effects of liquefaction damage to the Reclaimed fill as well as the sub-seabed soils and the field procedures to realize these Target Objectives, despite the relatively large earthquake Design acceleration (0.85 g) and the very stringent requirements for Vertical deformations as well as Horizontal Spreading.

**PROJECT ANALYSIS AND DESIGN BACKGROUND**

The project is a shipyard partly to be reclaimed from the sea. *Synchrolifts* will be used to launch or *Drydock* ships.

**Fig 2.0 Bird’s eye view of the Facility.**  
The shipyard Facility consists of two main bays with cranes, a ship lift system (*Synchrolift*), a dry dock area and storage and Maintenance facilities. About 160,000 M$^2$ of the facility will be constructed seaward and 90,000 M$^2$ will be constructed on land. The seaward side will be reclaimed using granular material up to a maximum height of 12 meters to make it level with the land elevation.

The project is in *Altinova, Turkey* fronting the sea of *Marmara*. It is close to the *Anatolian Fault* and other major fault systems in the area. More specifically, the project lies about a kilometer from the *Anatolian Fault*.

The underlying soil conditions are relatively poor soils on which an average 12 m high embankment fill will be placed to reclaim the area.

Silty sands, silts and gravels located at depths of about 6 to 8 meters are most susceptible to liquefaction.

It was therefore necessary to improve the ground in order to mitigate liquefaction effects for this high value facility.

The liquefaction analyses before and after ground improvement are discussed in this digest paper.

**Fig 3.0 Fault Map and Epicenter of the 1999 Kocaeli Earthquake**

An earthquake of magnitude M = 7.4 occurred on the *North Anatolian Fault Zone* with a macroseismic epicenter near the town of *Gölcük* in the western part of Turkey. Known as the *Kocaeli Earthquake* of 1999 the earthquake was a very destructive one and affected large areas as well as Harbor facilities in the sea of *Marmara*. Most of the after-shock activity is confined to the region bounded by 40.5-40.8N LAT and 29.8-30.0E LONG, which
covers the area between Izmit and Adapazari to the east of the epicenter.

The project site is only about 1 kilometer away from the fault and thus Design considerations for Seismic loading were very stringent specifically as it pertained to dynamic and deformation analyses.

The seismic analyses performed by the original authors requires acceleration time histories to represent the strong ground motion at the Altinova site. Records of the 1999 Kocaeli Earthquake were selected and spectrally matched to the design spectrum discussed in this paper. Two earthquake records were selected for the analyses: Izmit and Sakarya stations in Turkey. The Izmit station was located approximately 4.8 km from the fault rupture and is a Rock record (i.e. Vs at the site greater than 1500 m/s). PGA (Peak Ground Acceleration) recorded at this site was about 0.2 g. The Sakarya station on the other hand was located 3.1 km from the fault rupture and is a rock/stiff soil record (i.e. 750 <Vs<1500 m/s). PGA recorded at this site was about 0.4 g.

The sites were selected because they are near field effects that contain near field effects such as fault directivity. Near source effects that are important to the characteristics horizontal ground response are:

1. Higher levels of ground motions due to the close proximity to the active fault.
2. Directivity effects that increase the ground motions for periods greater than 5 seconds, if the fault rupture propagates towards the site (i.e Forward directivity) and
3. Directionality effects that increase ground motions for periods greater than 0.5 s in the direction normal (Perpendicular) to the strike of the fault.

Near source effects, such as fault directivity effects, are generally significant for sites located within 10 to 15 km from the causative fault. Fault directivity effects (or fault rupture directivity) is a well documented near source effect that influences deformation analyses. Fault directivity is a pulse or series of pulses of seismic energy that are preferentially generated in the direction of fault rupture. Because an earthquake is generated by a Shear dislocation that begins in the discrete area of the fault and spreads with a velocity almost equal to the velocity of shear wave propagation, this causes much of the seismic energy produced by the rupture to arrive in a single, large, long period pulse of motion that usually occurs near the beginning of the record.

This pulse of motion is referred to as fault Directivity and is similar to a Doppler effect for sound waves. Often the fault directivity pulse represents the accumulation of much of the seismic radiation from the fault. The radiation patterns of fault dislocation causes the largest pulses to be oriented in a direction that is perpendicular to the strike of the fault for a Normal and Reverse faults and parallel to the strike of the fault for Strike slip Faults.

A target Peak Ground Acceleration PGA of 0.85 g was the basis of design for the ground improvement studies that were done. A M=7.6 which is slightly higher than the design Earthquake of M=7.4 was used in order to produce a PGA of 0.85 for rock and for soil.

These were arrived at using the Abrahamson and Silva (1977) attenuation relation in order to develop the design acceleration Response Spectrum (5% Damping) for the proposed facility. Both rock and deep soil Spectra were developed for the site. The deep soil spectrum is recommended for the facility because of the deep soil profile that is present at the Altinova site.

The A & S attenuation relation generally provides a conservative (i.e overestimates) PGA at site distances less than 10 km from the fault. Nevertheless, the the PGA Target value of 0.85 g was used as the the A&S attenuation relation provided a good match to the design criterion provided to the authors. One important issue arose which was whether or not the the A & S (1997) relation for deep soil conditions was applicable to the Altinova site because of the profile of deep very soft clays.
Experience has shown that the A&S attenuation relation is applicable to deep soil sites but it may NOT be as applicable to soft soil sites (Bartlett 2004).

Thus, and in order to evaluate this issue, a 1-D ground response analysis for the Altinova site using a 1-D equivalent linear Model SHAKE91 (Idriss and Sun 1992) was made. This analysis was done using the figure below(Fig 5). The shear wave velocity values, Vs below layer 14 were estimated. Also the depth to bedrock was estimated.

Fig 5.0 Comparison of Rock surface Spectrum , deep soil spectrum from A & S (19197) and surface soil spectrum predicted by SHAKE91 for the Altinova site.

Fig 6.0 Design surface Rock response and deep soil Spectra for M=7.6 and R=1 km from A & S Attenuation Relation.

Fig 4.0 V_s Profiles for the SHAKE91 Analysis.

The Analysis produced a good match between the computer results and the A & S Spectrum (Fig 5). Thus, the A & S spectrum for M=7.6 and R=1 Km appears to provide a reasonable estimate of the surface soil response-for the Altinova site conditions. This good match also gave confidence to the authors that the dynamic properties of the model were reasonable.
SITE CONDITIONS

The underlying site soil conditions at the Altinova site are relatively poor with deep soft clays underlying the areas.

![Figure 7.0 Characteristic Profile of the site](image)

**Fig 7.0 Characteristic Profile of the site**

The characteristic N-values vs Depth for the landside boreholes is shown in Fig 6.0.

![Figure 12. SPT (N) values versus depth for landside boreholes.](image)

**Fig 8.0 Chart Showing characteristic N_values vs Depth for the Landside boreholes.**

LIQUEFACTION ANALYSES

The underlying soils as earlier discussed, consist of silty sands, silts and Gravels located at depths from 6 to 8 meters from existing natural ground line or sea bed.

The liquefaction analyses used the simplified procedure as proposed by *Seed and Idriss* to calculate the Factor of Safety (FOS) against the occurrence of liquefaction for a worst case scenario:

Using the simplified method, the factor of safety against the occurrence of liquefaction (FS) is calculated as follows:

\[
FS = \frac{CRR}{CSR} \tag{1}
\]

where

- \( CRR \) = cyclic resistance ratio
- \( CSR \) = cyclic stress ratio

\( CSR \) can be approximated using the following equation, which is based on the assumption that the stress cycles are uniform:

\[
CSR = \frac{\sigma_u}{c_u} = 0.65 \left( \frac{\sigma_{uu}}{\sigma_{ed}} \right) \left( \frac{\sigma_{ed}}{\sigma_{oo}} \right) \tag{2}
\]

where

- \( c_u \) = uniform cyclic shear stress amplitude
- \( \sigma_{uu} \) = effective overburden pressure at the depth of interest
- \( \sigma_{ed} \) = total overburden pressure at the depth of interest
- \( \sigma_{oo} \) = peak horizontal acceleration at the ground surface
- \( r_s \) = stress reduction factor

Evaluation was conducted using three situations:

1. without Ground Improvement treatment
2. treatment with *Impact ®Piers Rammed Aggregate Piers* (RAPs) considering only shear stress redistribution and...
3. treatment with *Impact ®Piers Rammed Aggregate Piers* (RAPs) considering shear stress redistribution and reduction in pore pressure as a result of radial drainage through the granular columns of the RAP.

The worst case was considered to be an \( N_{value} (N_1) \) \( 60 \) of 15 at an average depth of 7.0 meters below the native ground surface.
Results of the liquefaction analyses is tabulated below:

<table>
<thead>
<tr>
<th>Situation</th>
<th>FOS</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.61</td>
<td>Liquefaction</td>
</tr>
<tr>
<td>2</td>
<td>1.05</td>
<td>Liquefaction may not occur</td>
</tr>
<tr>
<td>3</td>
<td>2.42</td>
<td>No Liquefaction</td>
</tr>
</tbody>
</table>

In the case of the untreated ground (Situation 1) liquefaction can occur thus threatening the Facility.

For Situation 2, the FOS is 1.05 and thus Liquefaction may not occur. However a higher FOS is desirable for this critical facility.

For Situation 3, the very high FOS was achieved considering the contribution of Rapid Pore pressure evacuation or relief by mobilizing the Open graded granular columns of the RAPs as “Chimney drains” thus the onset of liquefaction is arrested.

The combined beneficial effects of Pore pressure relief through radial drainage and shear stress redistribution resulted in a higher and very acceptable FOS.

It has been proven, particularly in the Nambu Hyogen Earthquake in Kobe, Japan that sites stabilized with granular columns did not suffer any liquefaction due to pore pressure relief, preventing build up of pore pressure to critical levels that could trigger liquefaction due to the closely spaced Granular Columns promoting Radial drainage.

The reduction in pore water pressures due to Radial drainage into the Impact Pier was combined with the redistribution of shear stresses.

Shear stress redistribution to the very very stiff granular inclusions (RAP) afford a “Stress sink” which could funnel the shear stresses and thus reduce the shear stresses in the matrix soil.

Of importance also is the fact that the matrix soil surrounding the RAPs are also laterally prestressed and prestraned and thus the shear strength is increased.

Additional beneficial strength gain also results from the localized increase in pore pressure during installation due to lateral compaction followed immediately by rapid radial drainage.

In this extended analysis, the magnitude of porewater pressure was calculated using the Equation presented by Seed and Booker (1976) for pure radial flow within a gravel drain:

\[ \frac{\partial u}{\partial r} \left( \frac{\partial u}{\partial r} + \frac{\partial u}{\partial r} \right) - \frac{\partial N}{\partial t} \frac{\partial u}{\partial r} = \frac{1}{N} \frac{\partial N}{\partial t} \left( \frac{\partial u}{\partial r} \right) \]

An approximate numerical solution to Eq. 5 using the finite difference method was derived, with the following result:

\[ u_{i,j} = a_1 \left( 1 - \frac{x}{2r} \right) - a_2 \left( 1 - 2x \right) + a_3 \left( 1 + \frac{x}{2r} \right) + \Delta t \frac{\partial u}{\partial r} \frac{\partial N}{\partial t} \]

An approximate Numerical Solution to Equation 5 using Finite Differences was derived as shown in Equation 6 Above.

The final term of Equation 6 corresponds to the porewater pressure generated by the Earthquake (Ug) and was calculated using the following equation established by Lee (1974) and DeAlba et al (1975).

\[ U_0 = u_{00} \left( -\frac{1}{2} + \frac{1}{2} \sin \left( \frac{N}{N_L} \right) - 1 \right) \]

where \( N_L \) = number of cycles required to cause liquefaction in the treated soil
\( \alpha \) = a best-fit empirical coefficient = 0.7

In the above analysis, \( N/N_L \) was approximated by \( t/t_L \) where \( t_L \) for the treated soil was found as follows:

\[ t_L(\text{treated}) = t_B \cdot \text{FS(treated)} = 29(1.05) = 30.5 \text{ sec} \]
The Value of $C_h$ for the matrix soils was estimated to be 0.02 m$^2$/sec. Values of $\Delta_t = 0.125$ m and $\Delta t = 0.100$ s were used which provided a stable numerical solution.

The problem was solved using a spreadsheet.

According to the results, the estimated maximum value of excess pore water pressure ($U_{e\text{-max}}$) would occur at a time $t$ of 29 s (end of ground shaking) in the node farthest from the impact pier and would be about equal to 82.5 KPa. This value is much less than the value required to initiate liquefaction ($\sigma^*_{v0} = -200$ KPa) and the factor of Safety against liquefaction for this case is computed as:

$$FS = \frac{\sigma^*_{v0}}{U_{e\text{-max}}} = \frac{200}{82.5} = 2.42$$

In the case of the Impact Pier system, aside from the significant pore pressure relief that occurs through the open graded gravels within the Granular columns, significant stiffness of the surrounding matrix soils is produced by the lateral stressing and restraining effect induced by the bevelled tamper as it is driven by high impact high frequency hammering from the vibratory hammer.

Compaction of the columnar body leads to a very very stiff granular pile sometimes increasing the stiffness by a factor of 40 compared to the original matrix soils. The very stiff Impact Pier inclusions results in a reduction in shear stress in the Matrix soil because the very much stiffer Granular columns sustains a greater proportion of the shear stresses.

Pore pressure relief prevents the build up of pore pressures that could trigger liquefaction.

Studies have shown that the excess pore water pressure increases with increasing time at any radial distance and increases with increasing radial distance from the piers at any given time. It was also observed that the excess pore water pressures begin to dissipate very rapidly as soon as the the ground stops shaking.

The highly densified columnar body of granular materials resists shear stresses and enhances the load carrying capacity of the soil while at the same time limiting deformations to acceptable or near acceptable levels.

The foregoing observations indicate that the Rammed aggregate piers contributed greatly to the achievement of the project goals despite the very large ground Design accelerations (0.85 g) and the severe limitations on vertical deformations and lateral spreading. The other alternative was to use expensive driven or bored piles to support the very large loaded areas.

One other factor which has been largely ignored in the study is the increase in horizontal stress levels within the Matrix soil during the installation of the Impact Piers. Enhanced stiffness (often by a Factor of 10 from the original matrix soil stiffness level) of the surrounding matrix soil is due to the lateral prestressing and restraining effect of the bevelled tamper during installation which causes pushing and shoving of the aggregates sideward.

Thus, the calculated FOS for Situation 3 of 2.42 when the Impact piers together with radial drainage are considered, is probably somewhat in the conservative side because of the additional effect of prestressing of the matrix soils.

All of these foregoing characteristics made it possible to meet the relatively high design PGA and at the same time limit the deformations.

GROUND SHAKING AND DEFORMATION ANALYSES

Even if the deformations from liquefaction and lateral spread are marginal at the site, there could still be considerable deformation of the foundation soils underneath the 12 m high embankment planned at the shipyard.
The embankment and foundation deformations may result from liquefaction or excessive deformation associated with the soft, clayey sediments that underlie the granular soils.

The computer program FLAC (for Fast Lagrangian Analysis of Continua) was used to evaluate several cases and the results are as follows:

- **No Treatment** - The analyses results indicated that large deformation in the order of 0.8 to 1.5 m of horizontal displacements are possible near the crest of the embankment. These displacements are very large and damaging within the 50 meter of the embankment slope but the embankment may remain intact and not suffer catastrophic failure.

- **Impact Pier Treatment beneath Full Depth Portions of Embankment** - This 2nd case was analyzed and showed results that deformations are still relatively large near the toe of the embankment. The FLAC MODEL produced between 0.9 to 1.0 meter horizontal Displacements near the crest of the embankment. Because the magnitude of the horizontal displacements are very similar to the case with NO treatment, this does not imply that the Impact piers are not helping to reduce deformations. The Impact piers will REDUCE the amount of of Horizontal Ground deformations and vertical settlements at distances away from the embankment face, but are Not very effective at the face of the embankment (Within 50 m from the embankment slope). There is very little Impact pier treatment at this zone as shown in the figure 9 below:

- **Impact Pier Treatment beneath Full Depth Portions of Embankment Plus Additional Treatment of Face of Embankment and Underlying Foundation soils** - Figure 10 below shows the treatment scheme for Impact piers to include the embankment toe. With the additional protection of the embankment toe assured, the displacements as computed in the FLAC Program are to within 0.1 to 02 meter of horizontal displacements near the crest of the embankment. Although these values slightly exceed the deformation parameters given in the terms of Reference, the authors are confident that depending on the foundation type and other mitigation measures, the predicted displacements would be acceptable.

*Fig 9 Shows the Original Limited Extent of the Impact Pier Treatment*
DESIGN OF THE RAPs AND CONSTRUCTION SEQUENCING

The facility will be designed for an M7.4 Earthquake at a distance of 1.0 km. The design event is a repeat of the 1999 Kocaeli Earthquake. The design Peak Ground Acceleration PGA is 0.85 g. However, much of the facility will be located on a deep, soft soils (marine muds) and significant deamplification of the high frequency ground motion and soft soil effects is expected.

The project will be designed for a uniform floor slab load of 150 KPa across the entire 250,000 square meters area. The maximum vertical and horizontal desired to be less than 25 mm (both for horizontal and vertical) over 40 meters, which is a typical floor slab dimension of 40 m x 40 m.

The Rammed Aggregate Piers (RAPs) known as IMPACT® Piers will be installed in a regular 2.0 m x 2.0 m grid. The RAPs will have an average installed diameter of 600 mm across the entire infilled area using a total of 62,500 IMPACT® Piers. The installation lengths vary from 9.0 meters at the landside to 17.0 meters on the the seaward side(reclamation area).

The goal of the installation is to improve the soils sufficiently to control settlement and lateral spreading resulting from liquefaction and/or excessive deformation of the soft clayey foundation soils.

The construction sequencing will begin by improving the landside facility first with a length of 350 meters.

Then from the land, a perimeter sea dike will be built around the entire seaward perimeter with a length of 450 meters. This perimeter dike will be constructed in 100 meter long stretches and stabilized by IMPACT® Piers. Following the completion of the perimeter dike, the seaward part of the Facility will be infilled with granular material. The maximum dike and embankment height is about 12 meters on the reclamation portion.
Fig 12 The Business end of the mandrel showing the patented Bevelled Tamper and the hollow cavity from which aggregates are dropped during vibration and withdrawal of the mandrel.

Fig 12 The Modulus load Test Performed to verify load and deformation Response of the RAP.

SUMMARY AND RESULTS

The analyses conducted for this study showed that the use of Impact ® Piers within and beneath the full depth portion of the embankment fill are necessary to reduce movements and concomitant problems associated with liquefaction and lateral spread. The proposed average Diameter of 600 mm and spacing of 2.0 meters O.C. of the Impact Piers appears to provide adequate safety against large deformations associated with Liquefaction and Lateral Spread.

Based on analysis conducted using the Numerical Program FLAC, the horizontal and vertical movements associated with ground shaking may be greater than the design criteria provided to the authors this is primarily because the Sope and toe were not originally considered as requiring ground improvement. The results showed that the ground deformations at the slope and toe of the embankment and propagating 50 meters into the filled area would exceed the design criteria.

These additional analyses were conducted to find a potential solution to reduce the movements. Although these analyses showed that the movements were substantially reduced with the additional Treatment, the magnitude of movements still may be greater than the design Criteria. thus the analyses indicated that additional stabilization and treatment (Fig9.0) at the toe portion and slope areas may be required to reduce deformations from ground shaking at the Altinova site to acceptable levels.

In summary, the initial studies performed indicate that the Impact ® Pier solution would address the design Criteria adequately except at the sloped and toe portions which would require additional ground improvement. This was not part of this study but certainly would totally address the project requirements. thus the ground deformations can be brought down to acceptable levels with additional ground improvement.

The Impact Pier system therefore was finally adopted by the project owners and further expansion of the stabilized areas using Impact Piers has started.