SEISMIC RETROFIT OF UNREINFORCED STONE MASONRY BRIDGE PIERS AND DISCRETE ELEMENT ANALYSIS

Jaw-Nan (Joe) Wang[1] ; Michael J. Abrahams[2]

Abstract

The seismic behavior of unreinforced stone masonry structures can not be accurately studied by conventional force-based methods of analysis. The deformation and failure mechanism of this type of structures are governed by the mortar joints rather than the stone blocks. This paper discusses the discrete/distinct element method used in the seismic rehabilitation of the unreinforced stone masonry piers supporting the 145th Street Bridge over the Harlem River. It can be shown that this performance-based method is able to account for the important characteristics of the block interaction and mortar joint behavior.

Introduction

The 145th Street Bridge is a movable bridge crossing the Harlem River that provides access between Manhattan and the Bronx. The total length of the bridge is about 481.5 m, consisting of a 94.5 m steel through-trusses swing span, steel girder and truss approach spans and earth-filled approach ramps. The swing span is supported by a circular concrete pivot pier with a granite block fascia, as shown in Figure 1. The two river rest piers were built by unreinforced stone masonry blocks founded on concrete in-filled timber caissons.

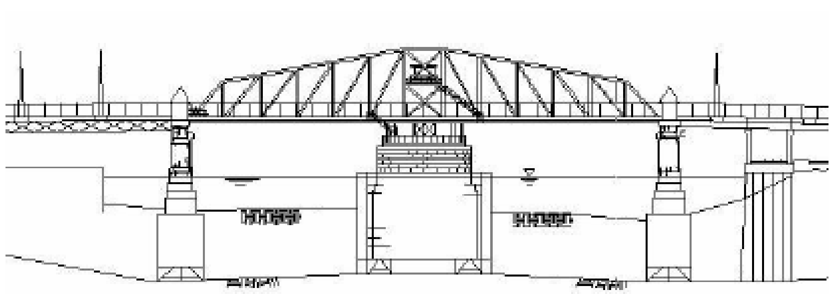

Figure 1 Swing Span Supported by Center Pivot Pier and Two Rest Piers

A design has been completed by the New York City Department of Transportation (NYCDOT) to replace the existing bridge superstructure. Concurrent with this activity, a seismic analysis and retrofit design for the entire bridge has been carried out. According to the seismic criteria established by the Department (1998), the

---

[1] Senior Engineering Manager, Parsons Brinckerhoff Quade & Douglas, Inc.
[2] Senior Vice President, Parsons Brinckerhoff Quade & Douglas, Inc.

bridge is to be evaluated and retrofitted for two earthquake levels: (1) a 2,500-year return period event, and (2) a 500-year return period event. It is intended that under the 2,500-year event the bridge shall achieve the following performance goals: no collapse; limited access for emergency traffic in 48 hours; and full service within months. Under the 500-year event, the performance goals are: no collapse; no damage to primary structural elements; and full access to normal traffic available immediately.

Based on the results of seismic evaluations, it was concluded in general that the 2,500-year event governs the retrofit design. Retrofit solutions for various portions of the bridge have been proposed (PBQ&D, 2001) and are currently being implemented.

Of particular interest is the retrofit solution derived for the two rest piers. As indicated in Figure 2, the piers are composed of unreinforced stone (granite) masonry blocks resting on top of a tremie-concrete-filled timber caisson.

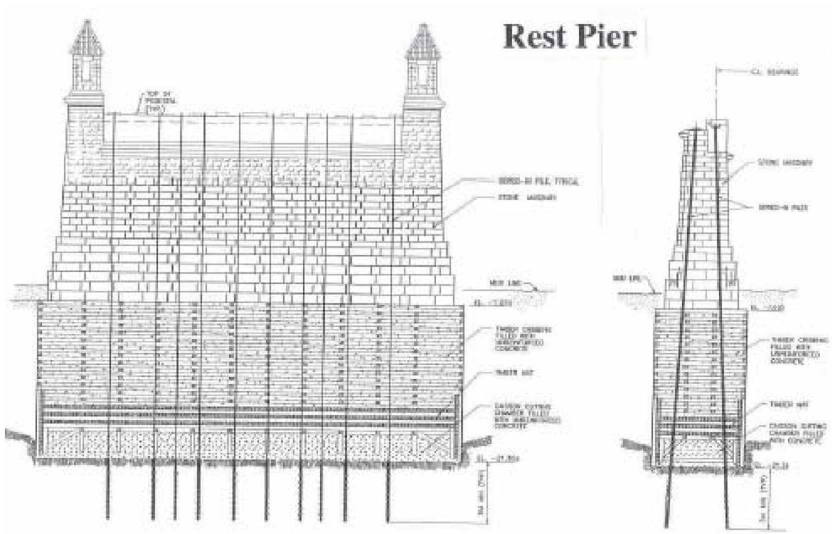

Figure 2 Typical Elevation Views of Rest Piers with Proposed Mini-piles as Strengthening Reinforcement

The height of the unreinforced stone masonry piers is about 15.5 m measured from base to top. The mean water level is about at the mid-height of the piers. The concrete caissons are entirely embedded below the mudline, laterally confined by a stiff varved silt and clay stratum with a medium stiff to stiff consistency. The height of the caissons is about 14 m. The bottom of the caissons is seated on bedrock. Based on the original plans as confirmed by foundation coring data, it appears that a timber mat exists in the lower portion of the caissons. Based on the core recoveries and materials washed out during the drilling the thickness of the timber mat is roughly in the range of 2 to 2.5 m.

Figure 2 also shows that the rest piers have a wide dimension in the transverse direction and are generally resistant to the seismic loading in the transverse direction. However, the longitudinal width of the rest piers is relatively small from the seismic standpoint, making the longitudinal overturning stability a major concern. Based on the structural analysis and capacity evaluation for the 2,500-year design earthquake, it was concluded that:

- The governing failure mode is the vertical tension fracture in the piers. D/C ratios for longitudinal overturning in the piers are greater than unity, if the tensile strength of the mortar is ignored.

- The D/C ratios for uplift at the edge of the timber mat and for sliding on the horizontal timber surface are also greater than unity. However, these modes do not necessarily lead to overall seismic instability due to the presence of the lateral passive resistance provided by the surrounding stiff silt and clay stratum.

- Substantial seismic forces in the piers are generated by their own mass. The majority of the longitudinal overturning forces in the piers were attributable to the mass of the piers.

To prevent longitudinal overturning problem, the rest piers are to be strengthened by the use of mini (micro) piles, extending from the top of the piers to a minimum of 5 m into the bedrock below the bottom of the caissons as shown in Figure 2. The mini-piles are used as reinforcement to provide tension capacity against overturning as well as to enhance the sliding resistance of the piers.

The seismic evaluations discussed above are based on the conventional force-based methods of analysis. The retrofitted piers were designed to effectively resist the elastic seismic forces derived from the global structural models and analysis methods, using multi-mode spectral response analyses. Hydrodynamic mass effects were included in the model. The seismic behavior of unreinforced stone masonry could not be accurately examined by these conventional methods, particularly in terms of residual (permanent) deformation, which is an important parameter for performance-based seismic design and retrofit studies.

In the following sections, the discrete/distinct element method will be presented. This method was used concurrently with the force-based analysis to verify the adequacy of the proposed retrofit solutions for the two rest piers. It will be demonstrated that this performance-based method is able to account for the important characteristics of the block interaction and mortar joint behavior.

Discrete/Distinct Element Method

Figure 3 shows a photograph of the existing west rest pier. The lower portion of the pier is below the water surface. As indicated in the photograph, the pier consists of an assemblage of very strong (granite) stone elements linked by weak mortar joints. The deformation and failure mechanism of this type of structures are therefore governed by the mortar joints rather than the stone blocks. The photograph also shows the pattern of the mortar joints -- parallel through joints along horizontal planes but discontinued and staggered in the vertical direction, suggesting some important interlocking effect between the blocks.

Figure 3 Masonry Blocks and Mortar Joint Pattern at West Rest Pier

Numerical modeling techniques based on continuum method of analysis are not appropriate. Instead, the model must account for the fundamental importance of the discontinuities (i.e., mortar joints in this case). To achieve this, the masonry piers were analyzed using the computer program UDEC. UDEC is a two-dimensional numerical program based on the discrete/distinct element method. The stone masonry as well as the timber mat was represented as an assemblage of discrete blocks, as shown in Figure 4. The mortar joints were treated as boundary conditions between blocks. Large displacements along mortar joints and rotations of stone blocks were allowed. Individual blocks behave as deformable material. Each deformable block was then subdivided into a mesh of finite-difference elements, and each element responds according to a prescribed linear or non-linear stress-strain law. For this study, the granite stones, timbers, and tremie concrete were modeled as linear elastic materials whose behaviors were characterized by unit weight, Young's modulus and Poisson's Ratio. The varved silt and clay (shown as the material surrounding the caisson in Figure 4) was modeled as a non-linear material following the Mohr-Coulomb failure criteria, characterized by un-drained shear and tensile strengths in addition to unit weight, Young's modulus and

Poisson's Ratio. The entire mesh including all discrete and finite difference elements is presented in Figure 5. The top of the rock was assumed to be the rigid base (bottom boundary) in the domain. Energy absorbing boundaries were assumed on the two sides of the mesh to prevent wave reflection in the analysis.

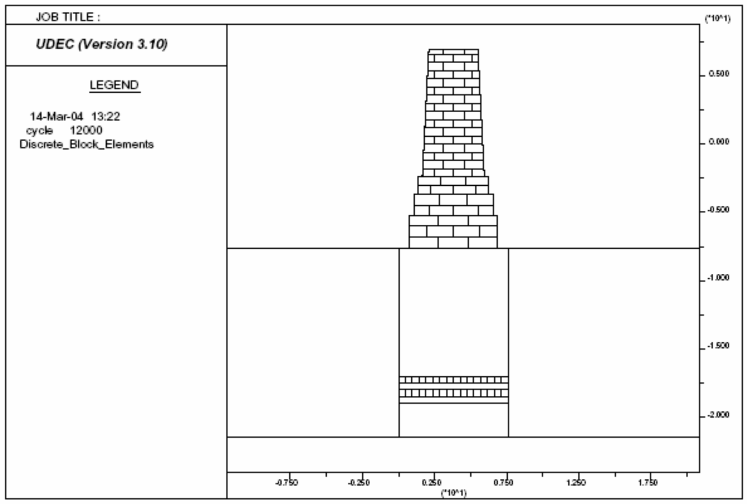

Figure 4 Discrete/distinct Element Mesh of Rest Piers

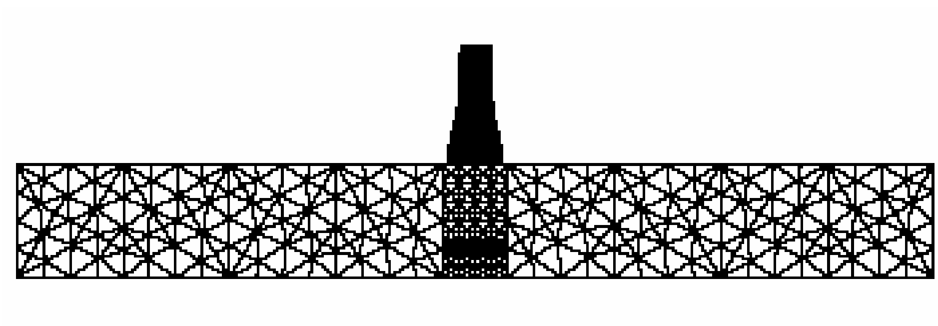

Figure 5 Global Discrete and Finite Difference Mesh

The shear resistance of the mortar joints and the contact surfaces between two dissimilar materials is governed by the Coulomb slip criterion, which assigns elastic stiffness (K_s), frictional angle (φ), cohesive strength (c), and tensile strength (σ_t) to a joint or contact surface. The maximum shear resistance is thus defined as (Figure 6):

$\tau_{max} = c + \sigma_n \tan\varphi$

where σ_n is the effective normal stress.

As discussed earlier, the mean water level is roughly at the mid-height of the piers. Hydrostatic water pressures were calculated and assigned to the mortar joints located below the water surface to account for the buoyancy effect on the shear resistance of the joints (i.e., reduction on σ_n and hence τ_{max}).

In the normal direction, the joints and contacts are allowed to resist tensile forces. For conservative purpose, the tensile strength of all mortar joints, contacts between timbers, and contacts between concrete and timber mat was ignored. Table 1 lists some of the important properties assumed for the mortar joints and contact surfaces.

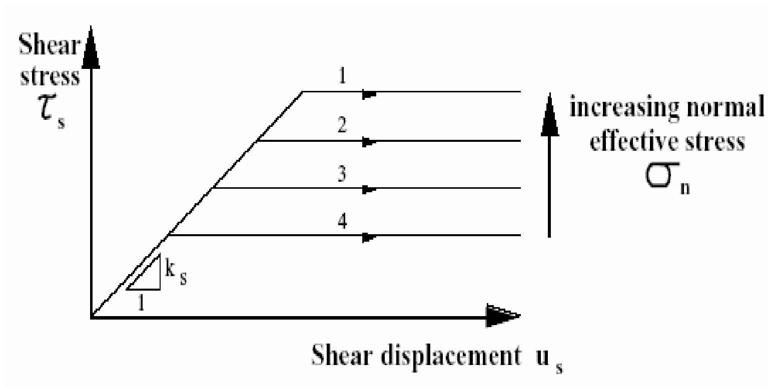

Figure 6 Coulomb Slip Criterion - Joint Shear Resistance

Table 1 Properties of Mortar Joints and Contacts

	K_s (Gpa/m)	c (Kpa)	φ (deg)	σ_t (Kpa)
Mortar Joints	100	0	30	0
Timber & Timber	20	0	17	0
Concrete & Timber	100	0	17	0
Concrete & Soil	100	17	0	34
Timber & Soil	20	17	0	34

Input Rock Motions

Input rock motions were developed based on the NYC Seismic Hazard Study Report (1998). Three sets of time histories were developed for 2,500-year using actually recorded earthquake acceleration time histories as seeds and then modified to match the target design response spectra. Figure 7 shows the 2,500-year rock motion response spectra (for a B-C Boundary firm ground site) used for this study compared to those published by AASHTO and the USGS hazard study (1996).

Figure 8 presents one of the horizontal acceleration time histories developed for the 2,500-year earthquake. Both Figures 7 and 8 show that the design rock motions are very rich in high frequency content, suggesting that they are not likely to contain high velocity impulse or large ground displacement amplitude. This is evidenced by the corresponding displacement time history record shown in Figure 9, where the magnitude of the peak ground displacement is at a very moderate level (about 6 cm). This is encouraging because high velocity impulse and large ground displacement motions are particularly harmful to structures from the overturning and sliding stability standpoint.

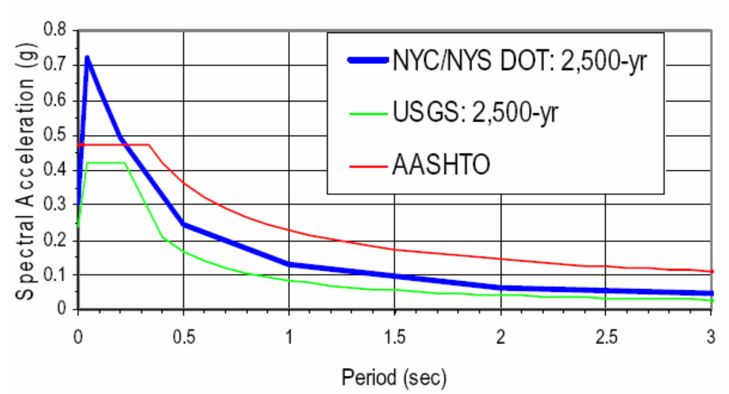

Figure 7 Design Rock Response Spectra Comparison

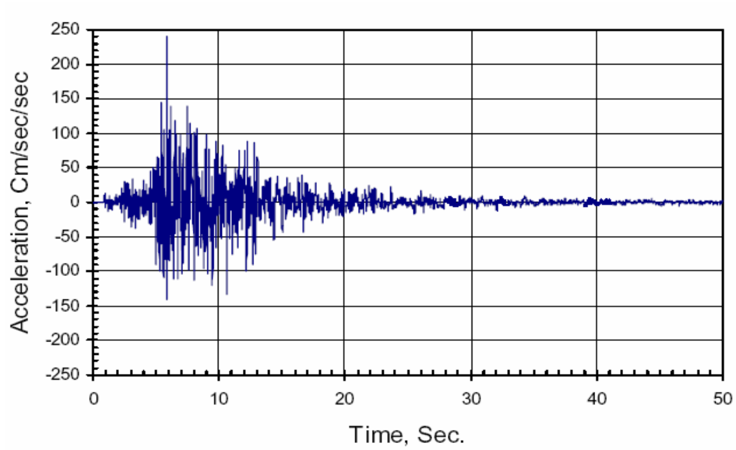

Figure 8 Input Rock Motions - Acceleration Time History

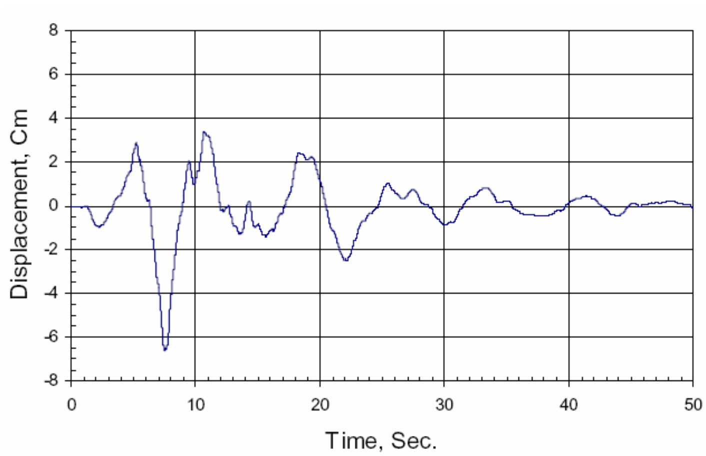

Figure 9 Input Rock Motions - Displacement Time History

The acceleration time history presented in Figure 8 was specified at the bottom boundary of the domain being analyzed (refer to Figure 5). The results of the analysis are discussed in the following sections.

Seismic Response Of Stone Masonry Piers

The discrete element analysis was performed for three cases. Case 1 evaluated the seismic behavior of the stone masonry pier in its existing condition (without retrofit measures). Case 2 analyzed the pier strengthened with 20 mini-piles (refer to Figure 2). For Case 3, the assumptions used were the same as those in Case 1 except that the vertical mortar joints pattern was assumed to run parallel and continuously. Case 3 is intended to show the effect of block interlocking on pier stability.

In all analyses, the vertical and lateral loads from the superstructure were assumed as static loads applied on top of the pier. It may not be a good representation to treat the superstructure lateral load as static load during for the dynamic analysis of the substructure. Nevertheless, this assumption is considered to be on the safe side.

Case 1 - Existing Pier Condition

The seismic response of the pier prior to being strengthened with mini-piles is displayed in Figures 10 through 12 at different time instants. Figures 10 and 11 are snapshots of displacements at T=4.7 sec and T=9.3 sec respectively. Figure 12 shows the permanent (residual) displacement after the shaking has ended.

At T=4.7 sec, the level of ground excitation remained fairly low. The pier, the caisson, and the base rock appeared to be moving in the same direction and no out-of-phase motions were observed. The displacements at the top and bottom of the pier and at the base rock were estimated to be about 0.7 cm, 1.3 cm and 1.5 cm respectively. The relative displacements appear to be small.

At T=9.3 sec, the results indicate that the base rock motions and the pier motions are out-of-phase (refer to Figure 11). While the base rock is moving toward the right hand side by a magnitude of 1.7 cm, the top of the pier is moving left by about 4.0 cm. Global instability or falling of masonry blocks were not observed, but some permanent deformations may have already resulted.

As indicated in Figure 12, the residual deformations of the caisson, including the timber mat, are negligibly small at the end of the shaking, thanks to the lateral confining resistance of the surrounding soils. The top of the pier, however, has permanently displaced by about 3.5 cm (toward the left). At the bottom of the pier (submerged portion), the masonry blocks have shifted outward by about 2.5 cm on both sides.

Figure 13 compares the displacement time histories of the base rock and the pier top. It is clear from this plot that the permanent displacement at the pier top developed at around T=8 sec, when the peak ground displacement occurred. After T=8 sec, the level of ground excitation was not sufficiently strong to cause additional permanent displacement.

Figure 14 displays the resulting crack opening of the mortar joints, magnified by 15 times for illustration purpose.

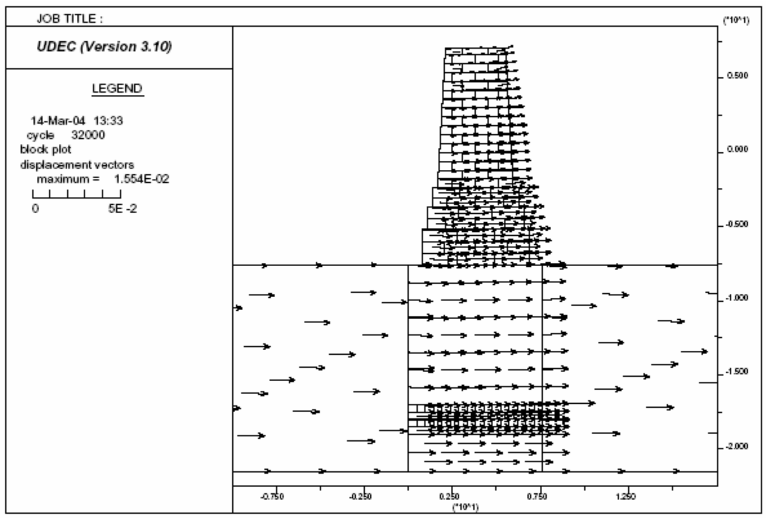

Figure 10 Displacement Vectors at T=4.7 sec

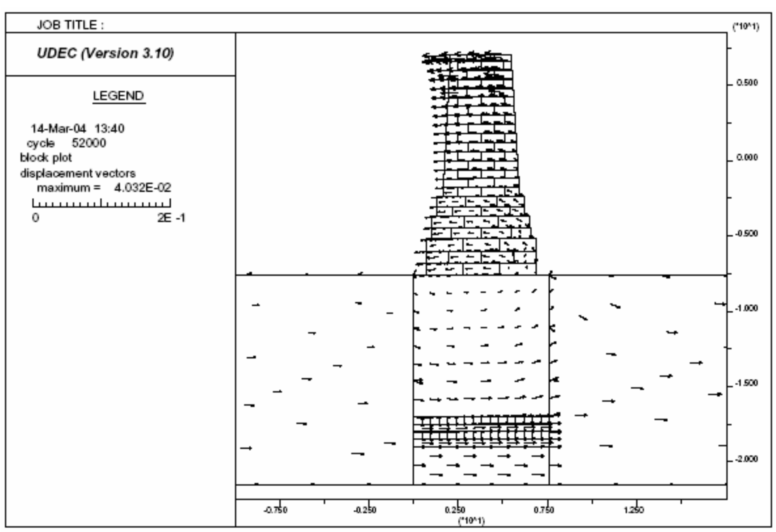

Figure 11 Displacement Vectors at T=9.3 sec

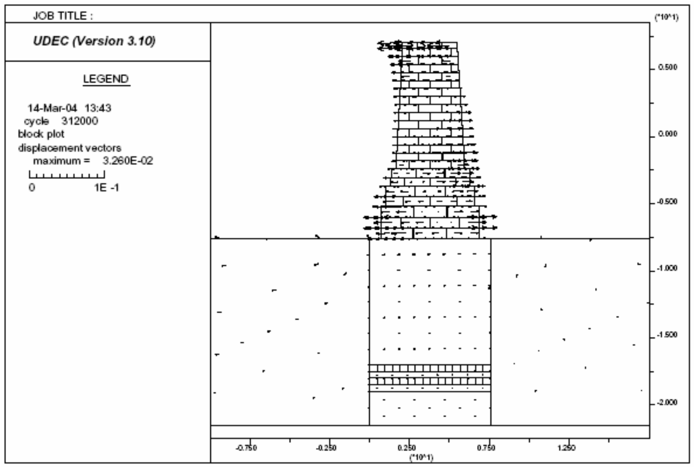

Figure 12 Permanent (Residual) Displacements

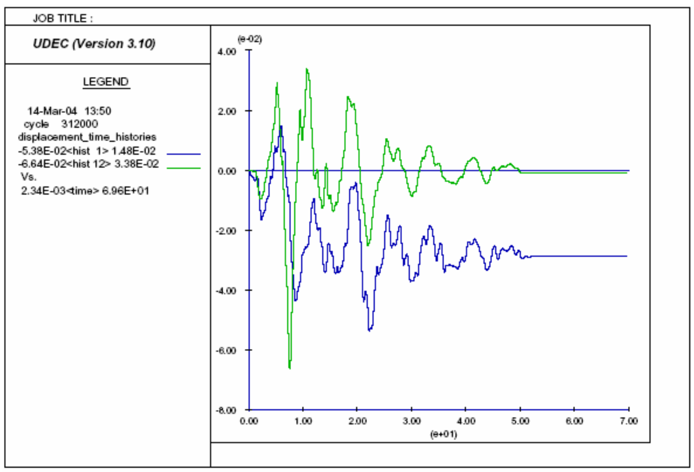

Figure 13 Pier Top vs Base Rock Displacement Time Histories

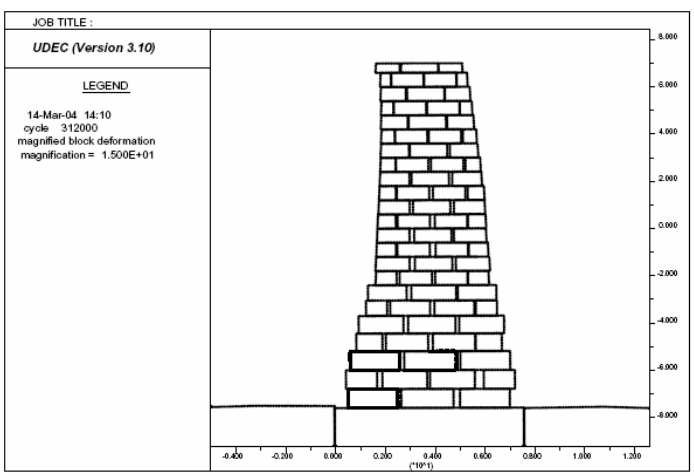

Figure 14 Width of Joint Opening (Magnified by 15 times)

Case 2 - Pier Strengthened with Mini-piles

In this analysis the 20 mini-piles were modeled as structural reinforcement elements. This model considers only the local reinforcement effect of the mini-pile when it passes through existing mortar joints and contact surfaces. The formulation is based on results from laboratory testing of fully-grouted un-tensioned reinforcement in good quality rocks (Bjurstrom, 1974; and Pells, 1974).

It should be realized that due to the 2-D plane strain modeling assumption, the effect of the mini-pile reinforcement is averaged over the spacing in the direction normal to the plane of analysis.

Figure 15 shows the discrete element mesh incorporating the mini-pile reinforcement. For this analysis it was assumed that the mini-piles will be installed using deformed #20 bars (dia. 63.5mm), Grade 75 (520 MPa), grouted in a borehole with a minimum hole diameter of 190mm.

The results of the analysis are presented in Figure 16, where the base rock motions are compared to that at the pier top. The results show that the use of mini-pile reinforcement can effectively reduce the residual displacement to an insignificant level at the pier top for the 2,500-year design earthquake.

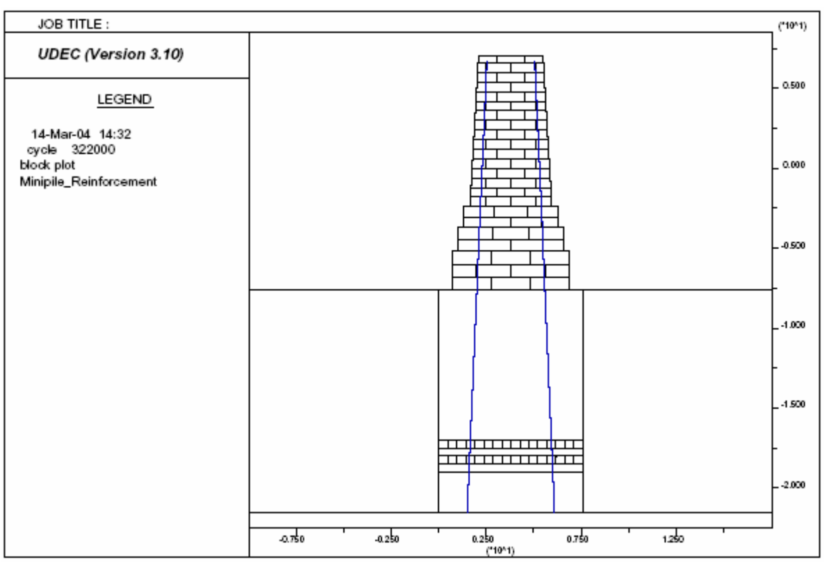

Figure 15 Discrete Element Mesh Incorporating Mini-Pile Reinforcement

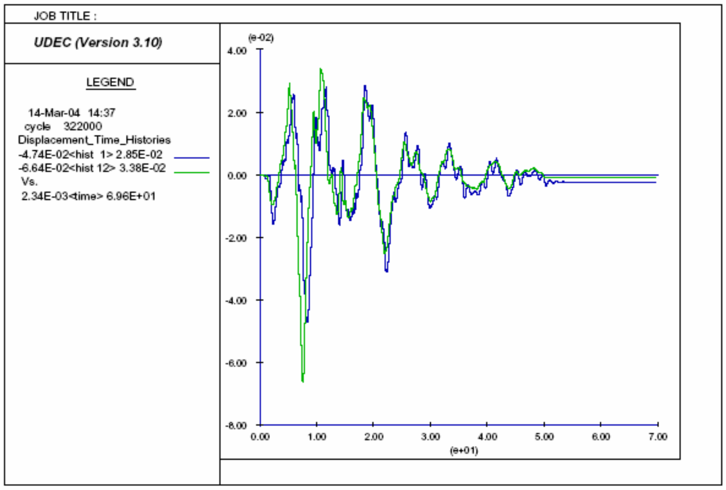

Figure 16 Pier Top vs Base Rock Displacement Time Histories – with Mini-piles

<u>Case 3 – Pier without Interlocking Joint Pattern</u>

In this case, the pattern of the mortar joints was purposely assumed to render no interlocking action between adjacent blocks. It differs from the actual joint pattern that was used to construct the rest piers (see Figure 3). Figure 17 shows the assumed vertical joint pattern and the resulting instability of the pier. Because there is no tension resistance along vertical joints (which is a reasonable assumption), if the joints are continuous then they are essentially creating several slender columns with its width equal to that of a single block. These individual columns tend to topple and progressively break away from the body of the pier, as indicated in Figure 17. With interlocking, such as that shown in Figures 3 and 4, the adjacent columns are tied together through shear resistance along the horizontal joint plane, making the pier more resistance to seismic excitations.

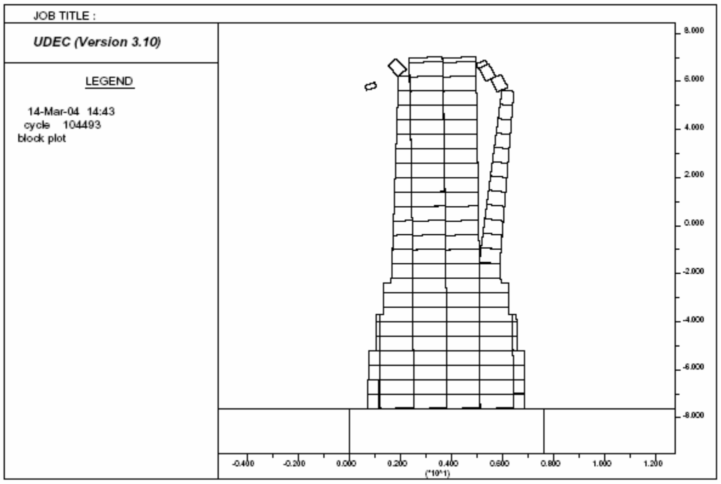

Figure 17 Effect of Joint Pattern on Pier Stability

Conclusions

The discrete element method is able to more accurately predict the seismic behavior of unreinforced masonry structures. It has the ability to simulate large deformation response and progressive failure of a structure and thus is a performance-based method of analysis.

Properties of the individual deformable blocks are generally not critical to the global stability of the structures, particularly if the blocks are made of strong material. Seismic behavior of unreinforced masonry structures is governed by the properties of the mortar joints and contact surfaces.

The overturning potential could be reduced if the masonry blocks are allowed to slide along their horizontal joint planes, because the transmittable horizontal inertial force could not exceed the sliding resistance. The sliding displacements, however, have to be controllable and within the allowable limit to meet the performance requirements of the structures.

From global stability and overturning standpoint, ground motions that are rich in high frequency content are generally not as damaging as ground motions that contain high velocity impulse and large ground displacement amplitudes.

Interlocking effect between masonry blocks can contribute considerably to the seismic stability of the unreinforced masonry structures.

References

AASHTO (1998), “Standard Specifications for Highway Bridges,” Sixteenth Edition, American Association of State Highway and Transportation Officials, 1996, with Interim Revisions through 1998.

Bjurstrom, S. (1974) “Shear Strength on Hard Rock Joints Reinforced by Grouted Untensioned Bolts,” in Proceedings of the 3rd International Congress on Rock Mechanics, Vol. II, Part B, pp. 1194-1199. Washington, D.C.: National Academy of Sciences.

New York City Department of Transportation (1998), “New York City Seismic Hazard Study and its Engineering Application – Final Report,” New York.

New York State Department of Transportation (1998), "Performance Criteria and

Seismic Hazard Level for Downstate Bridge Projects," New York.

Parsons Brinckerhoff Quade & Douglas, Inc. (2001), “Retrofit Assessment Report - 145th Street Bridge,” Prepared for New York City Department of Transportation.

Pells, P. J. N. (1974) “The Behaviour of Fully Bonded Rock Bolts,” in Proceedings of the 3rd International Congress on Rock Mechanics, Vol. 2, pp. 1212-1217.