BUILDINGS NEW YORK CITY DEPARTMENT OF BUILDINGS

Patricia J. Lancaster, FAIA, Commissioner

808 COLUMBUS AVENUE
REPORT OF INVESTIGATION

FEBRUARY 15, 2008

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DEPUTY ASSISTANT COMMISSIONER
FORENSIC ENGINEERING UNIT

I. EXECUTIVE SUMMARY

In the evening on July 25, 2007, a 30-foot long segment of a sheeting system collapsed at the construction site of 808 Columbus Avenue, Manhattan. The sheeting had been installed to prevent soil movement and to protect the foundations of adjacent buildings. The building nearest the collapse was 784 Columbus Avenue, a 16-story, 287-unit apartment building. 784 Columbus is owned PWV Acquisition, LLC, a limited liability company operated by The Chetrit Group and Stellar Management. Chetrit and Stellar also control the construction site at 808 Columbus.

The Department of Buildings vacated the residents at 784 Columbus pending a structural stability assessment. Following the assessment and immediate stabilization measures, the majority of the residents of 784 Columbus were permitted to reoccupy their apartments in the early morning hours of July 26, 2007. However, as a safety precaution, residents of the “P Line” of 784 Columbus, whose apartments were located immediately above the exposed foundations, were not permitted to return for six days. No injuries resulted from the collapse of the sheeting.

The General Contractor for the new building is Gotham Construction. Mayrich Construction is the sub-contractor for the excavation work, sheeting design and installation, and all blasting operations. Prior to and on the day of the incident, blasting of rock had been performed under a permit issued by the New York City Fire Department (FDNY).

The Department of Building’s Forensic Engineering Unit led this investigation to ascertain the cause of the sheeting failure. The Department was assisted by Stephen Young, P.E., and Dr. Chris Snee, C Eng, CPG, from Arup Engineering, an internationally renowned engineering firm. The Forensic Engineering Unit was represented by Dan Eschenasy, P.E.

The investigators conducted interviews, observed the debris layout and studied the rock and soil conditions. They also reviewed a geotechnical report prepared by RA Consultants, the geotechnical engineer of record for the new building and drawings and calculations by Mayrich. They compared this information with the conditions at the site and performed their own calculations in reaching their conclusions.

The investigation concluded that the segment of sheeting system that collapsed was erected on top of a highly weathered (fractured) weak rock formation, and not on the competent rock indicated on the design drawings. Calculations showed that sheeting set upon a weathered rock base would eventually fail once the adjoining competent rock holding the weathered rock base in place was destabilized, whether by blasting or by mechanical means. On the date of the collapse, the adjoining rock was in fact removed by blasting. Prior to the blasting, this rock had provided lateral restraint to the weathered rock mass underneath the sheeting. The fragmentation of the adjacent rock allowed the weathered rock on which the sheeting had been erected to be displaced and thus induced the collapse.

A lack of data prevented the investigators from formulating a comprehensive picture of how and if blasting may have affected the sheeting system and the nearby rock. For example, on the day of the incident and on several previous days of blasting, seismic monitoring of the blasting did not take place. And while the investigation did not find signs of excessive excavation from the blasting, the gas pressure and shock produced by blasting in close proximity to piles numbered 2 and 3 did have the potential to further weaken the already soft mass of weathered rock. Blasting had no observable effect on the structural stability or on any of the elements of the building at 784 Columbus.

The lack of proper observations and stabilization measures by the responsible supervising engineer throughout pile installation and subsequent excavation allowed this failure to occur. If the rock under the sheeting had been competent, as indicated in the design drawings, or if the weathered rock had been stabilized as soon as it was identified either in the field or from a reading of the geotechnical report submitted by RA Consultants prior to the excavation, the fragmentation of the adjacent rock would not have caused the sheeting to fail and the incident would not have occurred.

The fractured and weakened condition of the rock mass could have been identified by the controlled inspector employed by Mayrich, John Giacopelli, PE, and immediately brought to the attention of Gotham. The failure to identify the weakened condition of the rock should be referred to the appropriate units for issuance of violations or other appropriate action.

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II. REPORT OF INVESTIGATION

1 INTRODUCTION

1.1 Background

In the evening on July 25, 2007, the Department of Buildings responded to reports of a collapse at 808 Columbus Avenue, Manhattan. Upon arrival at the Upper West Side building site, Department engineers found that an approximately 30 foot temporary support of excavation system (referred to as "sheeting" in this report) had collapsed and exposed part of the foundation columns supporting an adjacent building, 784 Columbus Avenue. As a precautionary measure, residents of 784 Columbus Avenue were ordered to vacate. A temporary earthen berm was built in front of the collapse area using rock and soil available on site. Soon thereafter, when it was determined that the berm was sufficient to prevent any unsafe building movement and that the collapse had had no effect upon the safe occupancy of 784 Columbus other than the P-line apartments, the residents were allowed to return to their apartments. In the following days the Department supervised further stabilization measures and the residents of the P-line were allowed to return. The Department commenced a detailed investigation of the incident.

Photo 1 – The Collapsed Sheeting

1.2 Investigative Team

Immediately following the incident, Christopher Santulli, PE, Manhattan Borough Commissioner and Dan Eschenasy, PE, Deputy Assistant Commissioner of Safety & Emergency Operations and Chief of the Department's Forensic Engineering Unit, arrived at the site and participated in the Department's decisions regarding stabilization measures and public safety. Eschenasy was designated lead investigator; Department forensic engineers Eric Reid, PE, Timothy Lynch. PE, and Naweed Chaudhri, PE took part in the monitoring of the stabilization measures. Because the scope of the investigation required engineering capabilities beyond those of the Department's personnel, Arup Engineering, an internationally-renowned consulting firm, was asked to participate. Arup was represented by Stephen Young, PE, Associate Principal, and a sub-consultant, Dr. Christopher Snee. Dr. Snee is a rock and blasting expert.

1.3 Methods and Chapter Attribution

Each member of the investigation participated in the production of this document and commented upon the content. Conclusions were formulated based on the specific competence of the members: Snee observed the excavation and reconstruction in the area of collapse and examined blasting operation and records. He authored Chapter 5 –Geological Conditions and Chapter 7 – Blasting. Young led the Arup group that performed the engineering calculations. He wrote Chapter 8.1 - Computer Model.

2 TIMELINE

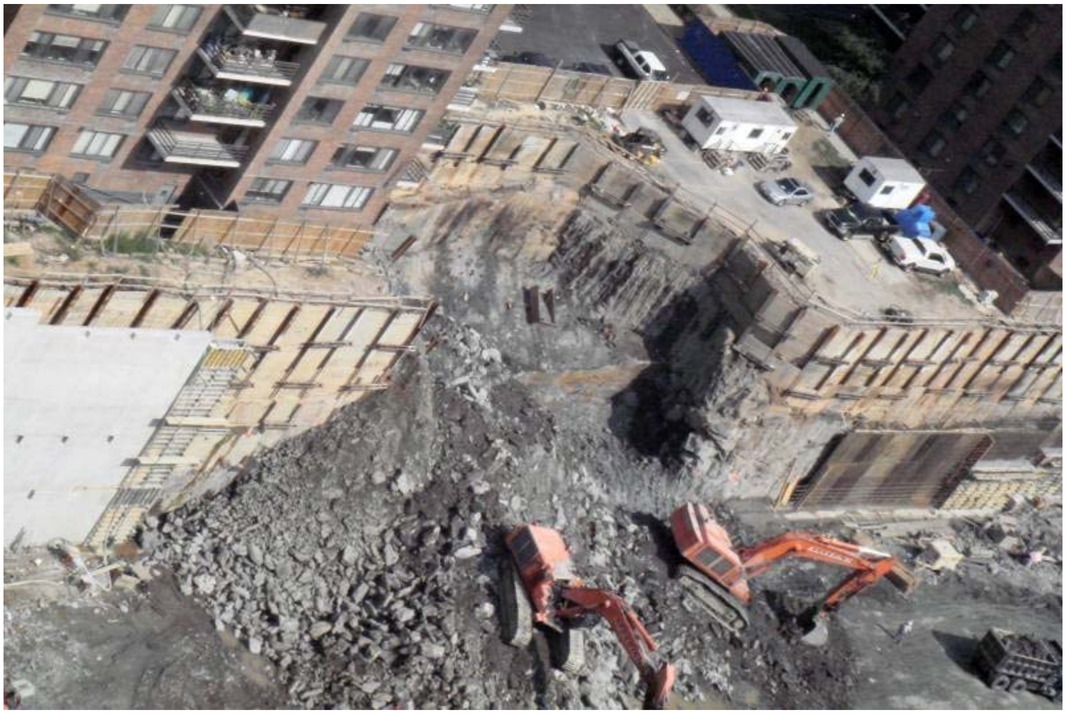

Photo 2 – Stabilization Berm

A new building is being erected at the site known as 808 Columbus Ave, Manhattan a.k.a. 100 West 100th Street. The architect of record for the new building application is Costas Kondylis and Partners, and the structural engineer is WSP Cantor Seinuk Group ("Cantor"). Site soil investigation was prepared by RA Consultants. Gotham Construction ("Gotham") is the General Contractor for the project and Mayrich Construction Corp. ("Mayrich") is the sheeting subcontractor. .

On October 13, 2006, three permits were issued to Gotham for excavation and foundation, sheeting, and underground plumbing work for the new building. The permits were renewed on 12/6/06 to expire 12/6/08. The applicant for the excavation and foundation work (Job No. 104563679) is Jeff Smilow, PE of Cantor. The applicant for the sheeting work (Job No. 104563679) is John Giacopelli, PE, of Mayrich. Giacopelli is also the TR-1 applicant for sheeting work, which means he is responsible for conducting the controlled inspections and certifying that the sheeting conforms to all applicable regulations.

According to Department and Gotham records, the following is a brief outline of events leading up to the collapse. The NB permit (no. 104464438) was first issued on 02/05/07. The chronology leading up to the collapse is as follows:

a) October 2006 – Excavation work, including mechanical rock chopping commences
b) November 2006 – Sheeting and soldier pile installation commences
c) March 5, 2007 – Blasting permit is obtained from FDNY
d) March 12, 2007 – Blasting test occurs under the supervision of FDNY; blasting

commences

e) March 12, 14, 19, 21, 23, 28 and April 2, 2007 – Blasting occurs with vibration monitoring by Metric and reports for these dates submitted to FDNY
f) March 23, 2007 – Department receives 311 complaints that allege blasting at 808
g) March 28, 2007 – Department receives 311 complaints that allege blasting at 808
h) Excavation moved on the north side and after rerouting of mechanical, electrical and plumbing lines
i) July 19, 2007 -- Meeting between Mayrich, Gotham and Stellar in which blasting is scheduled for the following Monday
j) July 24, 25, 2007 -- Blasting reports for these dates submitted to FDNY. No seismic monitoring takes place
k) July 24, 2007, 9:13 a.m. – Department receives 311 complaint alleging blasting is going off without proper notification and buildings are not protected
l) July 24, 2007, 9:15 a.m. – Caller states demolition at location unsafe, no notification provided for neighboring buildings
m) July 25, 2007 – Blasting operations continue
n) July 25, 2007, 1:49 p.m. – Department receives 311 complaint alleging excavation is done with explosive and building shakes
o) July 25, 2007, 3:18 p.m. – Caller states dynamite has caused building to vibrate and shake. States vibration is felt at 784 Columbus Avenue and has been taking place all day
p) July 25, 2007, 3:20 p.m. – Last blast occurs
q) July 25, 2007, 4:00 p.m. (app.) – Site work ends
r) July 25, 2007, 6:53 p.m. – 911 complaint alleges collapse has occurred Caller(784 Columbus Avenue) states there is a major construction site behind the location given and a part of one retaining walls looks as if it going to collapse
s) July 25, 2007, 6:55 p.m. (app.) – FDNY, NYPD and the Department respond; 784 Columbus Avenue is vacated
t) July 25, 2007, 7:29 p.m. – Caller states at the above mentioned address (808 Columbus Avenue), the retaining wall collapsed
u) July 25, 2007, 7:51 p.m. – Caller states that the retaining wall at this location (784 Columbus Avenue) has fallen
v) July 26, 2007, 1:00 a.m. (app.) – 784 Columbus Avenue is reoccupied .with the exception of apartment line P that will stay vacated till further investigation completed
w) July 25 – July 30, 2007, Consolidation of 784 Columbus foundation, re-inspection and monitoring of entire site
x) July 31st, 2007: apartments on line P are reoccupied

3 STABILIZATION AND INSPECTION FOLLOWING THE COLLAPSE

3.1 Immediate stabilization

Shortly after the collapse at approximately 10:00 p.m., the contractor created an earthen berm where the collapse occurred and against the foundation of 784 Columbus. In addition to the creation of the berm, soil remediation was performed over the next few days consisting of grouting the area around the possibly affected building foundation of 784. The remediation was designed by Robert Alperstein, P.E. of RA Consultants, the geotechnical engineer of record for the new building. The remediation consisted of grouting of the area around the corner foundation. During remediation the building was monitored with optical survey and dynamic monitoring.

Photo 3 – Stabilization –Earthen Berm

3.2 General Site Inspection and Control

At the request of the Department, after the collapse the developer hired Shapiro Engineering, P.C., ("Shapiro") as an independent inspector. Shapiro performed and submitted calculations to verify the original calculations. Several measures of site improvement were taken including the placement of additional rock bolts, the toe-pinning of four soldier piles and the repair of some lagging. Anchor testing was also performed (see 6.3.1).

3.3 Field Investigation

The area immediately surrounding the wall collapse was examined as work to install new sheeting took place. Debris of steel piles and anchors were examined.

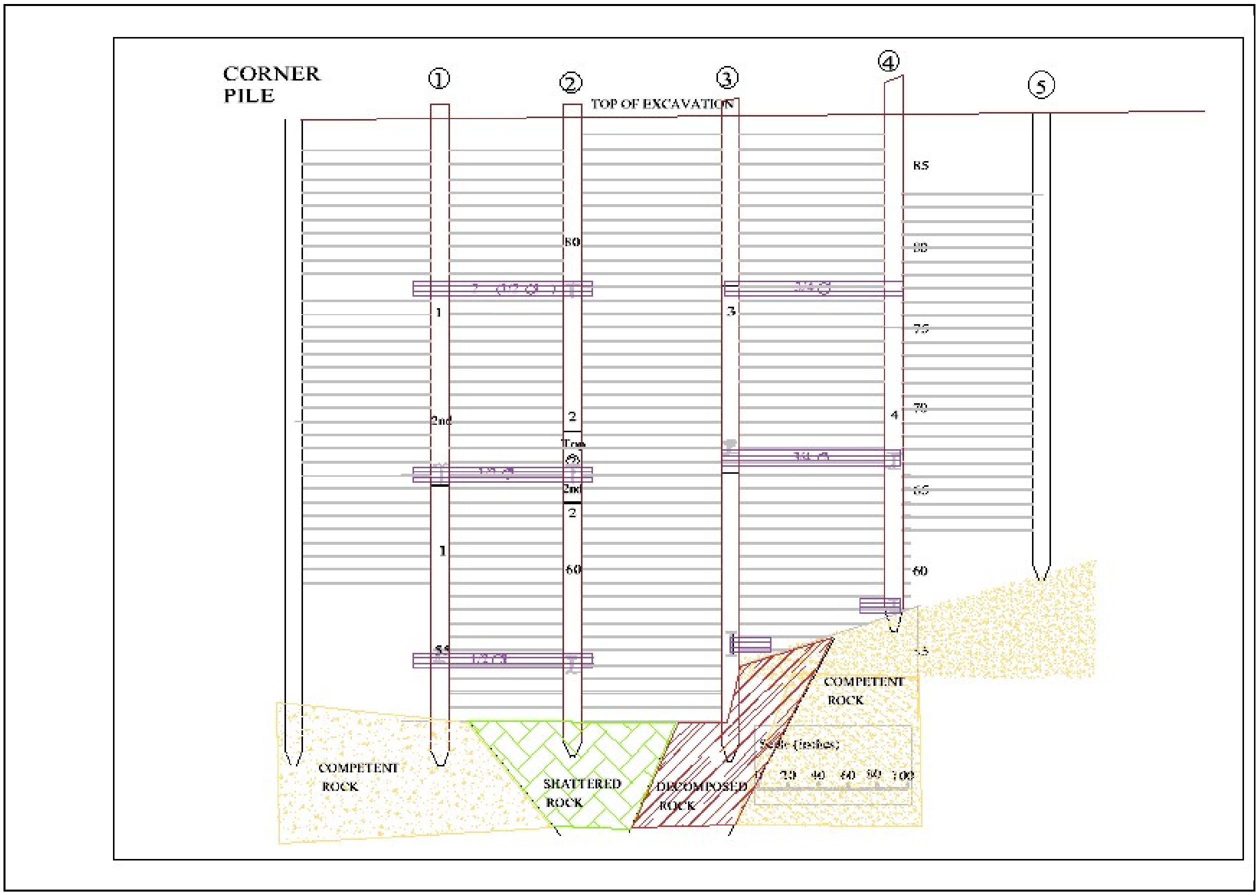

Figure 1 Installed Sheeting Elevation

4 ORIGINAL CONFIGURATION AND DEBRIS LAYOUT

4.1 Original Configuration

Photo 4 – The Sheeting Wall Pre-Collapse –Bernstein photo 7/20/2007

The segment of the sheeting system wall that failed was the southeast corner of a bay approximately mid-way along the length of the west wall of the excavation. The start of the bay was the Corner Pile and there is a curved section to #5. The investigation could not establish with absolute precision the location of piles 1, 2, 3 and 4. The attached plan sketch **[Figure 2]** was created based on field observations and drawings from Mayrich. The piles might have been located +/- 1 foot from the position shown on the drawing. The sketch of the vertical configuration is based on the Bernstein July 20 (photo 4), on photos submitted by neighbors (Photo 8) and on examination of steel debris.

The top of the rock slopes down from Pile #5 to the Corner Pile from about elevation 65 to elevation 50. The top of the piles is typically elevation 85 except for Pile #3 which is significantly lower than the other piles at approximately elevation 80. The pile appears to have been extended with timber to the top of the wall at elevation 85.

The Bernstein July 20 (photograph 4) is unclear but it appears that the toe of the wall is covered with rubble. The debris is more typical of blast rock rather than the larger blocks typically produced by a hydraulic breaker. It is unknown what method was used to excavate the rock in this area prior to July 20.

These and other photographs show that the excavation was down to the final grade level as far as Pile #2 and all of the toe of the wall was exposed, apart form the rubble covering.

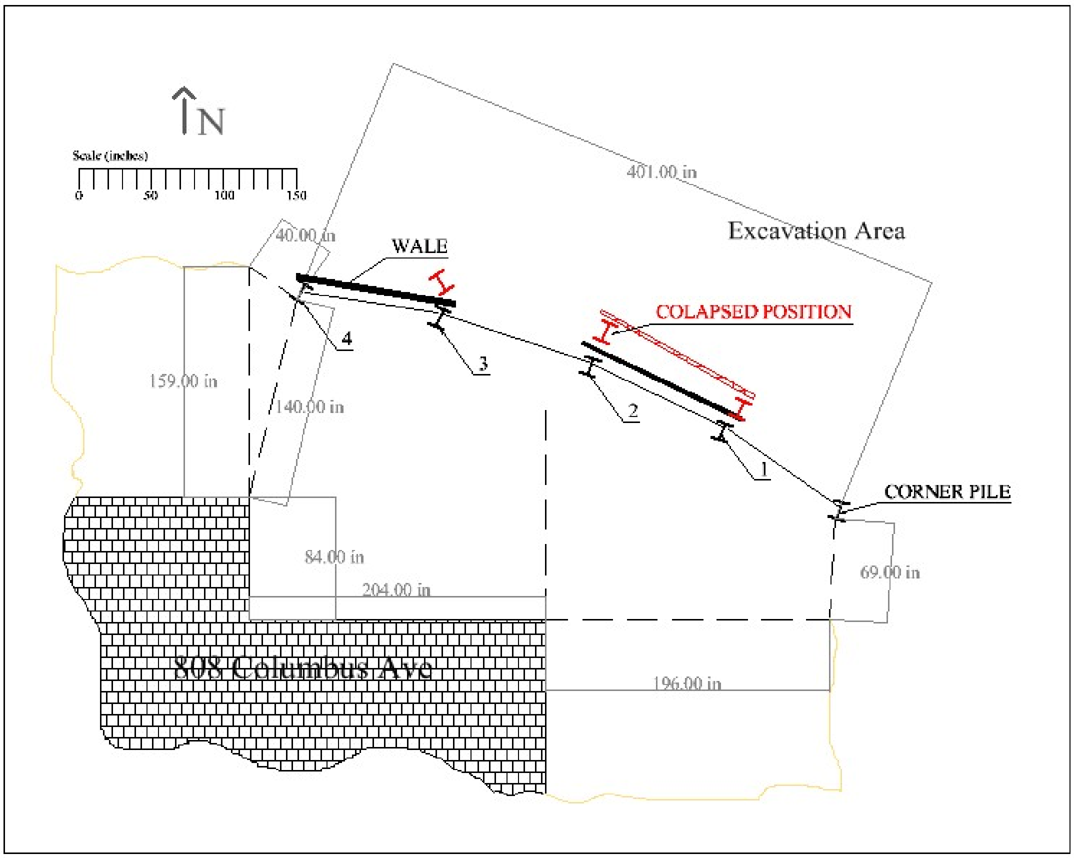

Figure 2 – Approximate Layout of Sheeting Before and After Collapse. The number identifying each pile is for this investigation only.

4.2 Debris Layout

Debris layout is shown in photo 5 and figure 4. Only pictures produced immediately after the collapse were considered. Later pictures were not considered meaningful since the installation of the berm, which started about 9:00 pm, most likely disturbed the configuration of the debris.

Photo 5 – Collapsed Sheeting

4.2.1 Piles

The steel beams that are driven into vertically into the ground are known as "piles." The Corner Pile and Pile #5 do not appear to have moved. Piles #1, #2 and #3 are completely separated from the wall and Pile #4 is fixed to the adjacent Pile #5 by lagging below elevation 73.

Pile #1, #2 and #3 are standing sub-vertical surrounded by a slope of soil that has run from the back of the lagging. The tops of the piles appear to have dropped by about 5 feet and are presumably standing on the floor of the excavation. The piles had moved outward. The extent of this lateral movement was not established.

Pile #1 appears to be the most vertical and in its original orientation. Pile #2 is tilting and has rotated slightly clockwise from the original position. Pile #3 is tilting and has rotated 90 degrees counter-clockwise. Pile #4 has tilted out from the original position.

The piles were not recovered in their entirety. Several top segments of the soldier piles were cut. The removal from site by the contractor of these segments was done in spite of a SWO that allowed work for stabilization only. However, the missing pieces are not deemed to have contained any essential data. The welds around the plates connecting the wales to the soldier piles failed. The welds measured 3/16 of one inch.

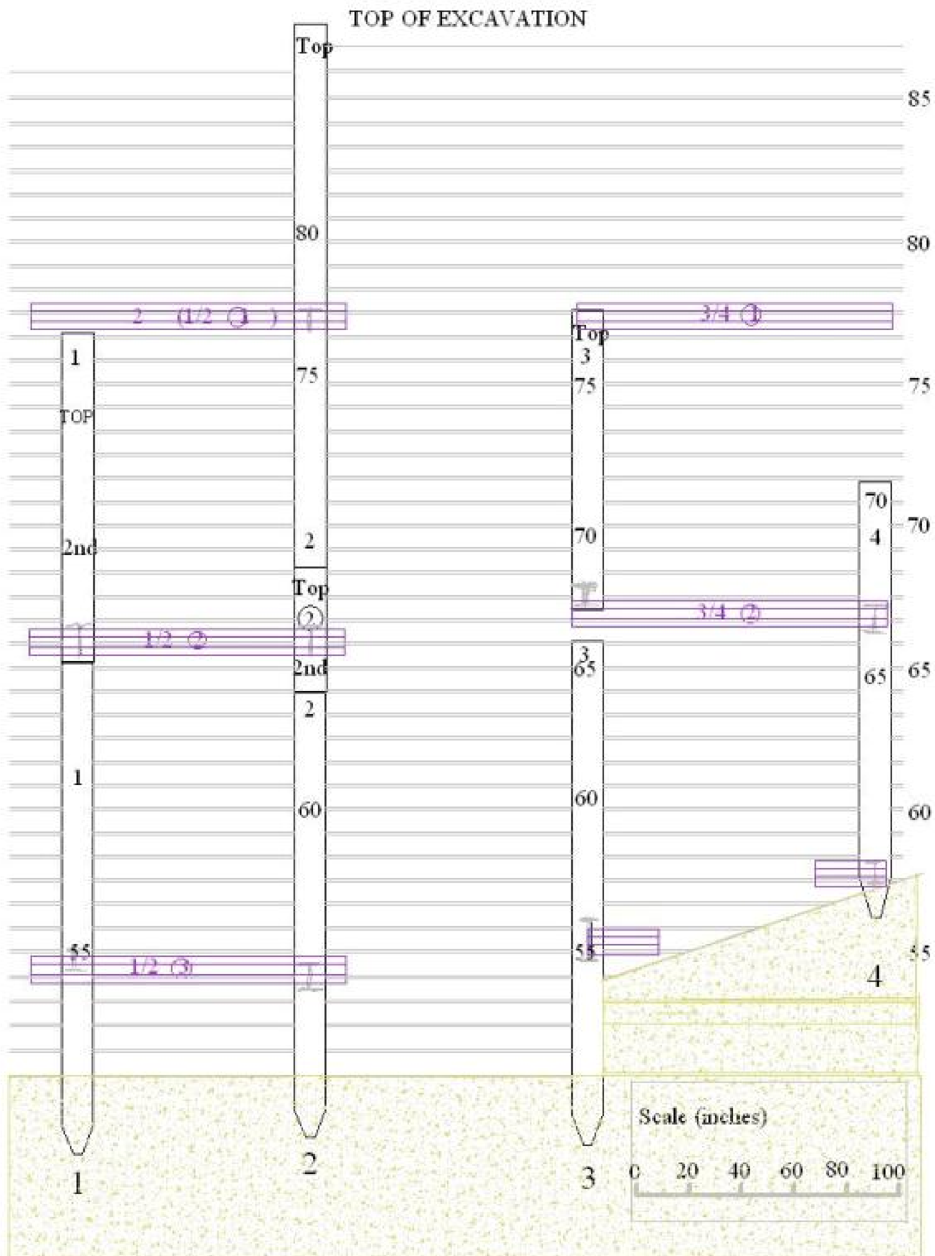

Figure 3 Recovered Pile Segments -Set-up

4.2.2 Wales

Wales are the horizontal metal supports used to connect and add stability to the vertical piles holding the wooden lagging in place. Upon examination at the site, all of the wales had broken from their welds to the piles except for the 2nd and 3rd wales between Pile #1 and #2 that had remained welded at Pile #1. All wales were recovered in the debris pile.

4.2.3 Lagging

The wood planking that is installed between the piles is known as "lagging." From examination of the photographs after the collapse, all of the wooden lagging between the piles was out of place except between Pile #4 and Pile #5 below elevation 73. The lagging between #1 and #2 appears to be in front of the piles. Most of the lagging is obscured by the debris pile between Pile #2 and #3 and the lagging between Pile #3 and #4 appears to be behind Pile #3. In many cases, the lagging lies close to the piles is partly fixed. The lagging was significantly damaged when the berm was built because heavy rocks were dropped on top of the debris.

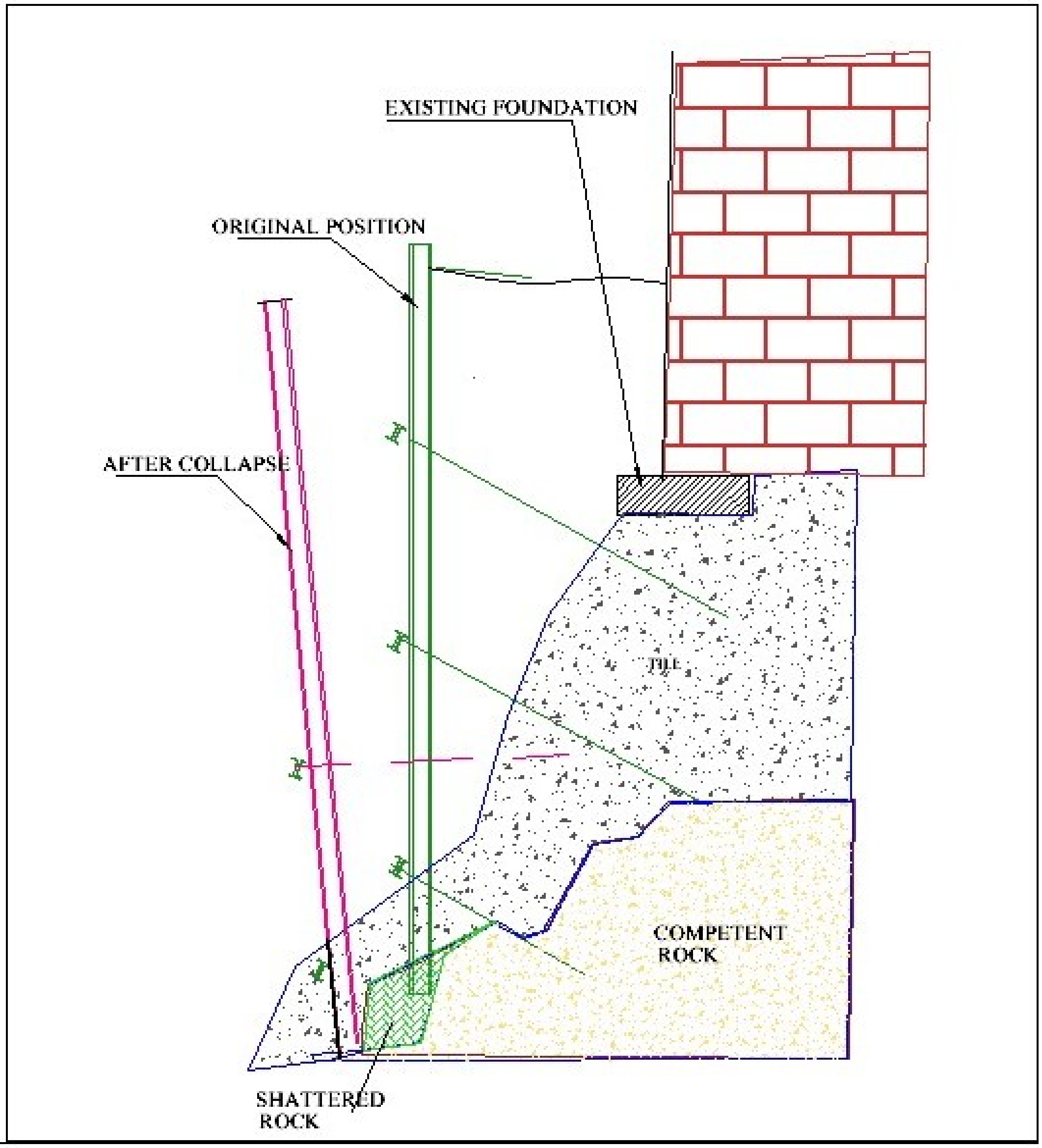

Figure 4 Rendering Section Looking East –Sheeting before and after collapse.

4.2.4 Anchors

The piles were stabilized in the adjacent soil through the use of anchors (also commonly referred to as "tiebacks") embedded in the soil behind the sheeting (**Figure 6).** The anchor heads are still visible in **Photo 1** in the wales that are exposed. There is a poor image of an anchor exposed in the soil wall between pile #1 and #2 sloping down to the right at approximate elevation 65. All anchors heads were recovered, some of which appear to have been torn during the remedial work. One anchor head might have failed before the remedial work.

4.2.5 Soil

The soil remaining behind after failure is standing close to vertical. The layered state of the fill overlying glacial till can be seen. There is no water issuing from the face.

The bulk of the soil that moved forms a cone with an apex between Pile #2 and Pile #3. There is additional soil build up behind one of the wales and Pile #2. The cone of soil has the same color as the fill and till, and the surface is covered with brick fragments presumably from the fill.

A green drainage pipe lies on top of and at the base of the cone. The exposed soil shows the location of the sewer pipe and the open ends left by the loss of the piece found on the debris pile. No liquid was observed running from the pipe during the investigation. The top of the inclined face of the standing soil reached the building line.

5 GEOLOGICAL CONDITIONS

5.1 General Conditions

The site is located in the north of the Manhattan Prong and to the south of the major Manhattanville Fault that traverses the island in a northwest to southeast trend. The rock is typical of the metamorphic material in this area and comprises fresh dark grey biotite rich mica schist. The rock mass is fractured with a systematic joint system dominated by fractures parallel to the foliation planes defined by the metamorphic fabric, a joint set conjugate to the foliation and a sub-vertical to vertical joint set. This joint system is typical of Manhattan schistose rocks. The site contains numerous minor faults, particularly foliation shears, which are also typical of Manhattan geology particularly in proximity to the major Manhattanville Fault.

The rock mass exposed in the walls of the excavation adjacent to the collapse was generally stable but there was local instability at re-entrant corners as kinematic wedges were defined by the return in the excavation profile and by small wedges associated with minor foliation shears. These potentially unstable features were stabilized by grouted rock-bolts and pinning steel channel to the walls. This method is normal for rock excavation in Manhattan and is considered to have been appropriate.

There were a few dispersed seepage points from fractures along the walls of the excavation but flowing conditions were not observed.

5.2 Specific Conditions - Soil

According to the November 7, 2006, Preliminary Geotechnical Investigation Report (Phase 2) prepared by RA Consultants, the retained soils consist of medium dense fill from ground surface to elevation +75 feet, underlain by a stiff clay stratum which is approximately 3 feet thick. Beneath the clay very dense glacial till is encountered to elevation +57 feet, which overlaid the decomposed rock. The report states that the groundwater measured from about elevation +74 to +76 feet, approximately 10 feet below ground surface. However, the mode of installation of the soldier pile and lagging wall was intended to facilitate the drainage of water through the wall, and there should be no water pressure build-up behind the wall.

5.3 Specific Conditions - Rock

The rock mass in the area of the collapse was observed and mapped in detail during excavation on August 27, 2007 and August 28, 2007. Figure 1 is a sketch of the exposed rock face between new piles #1 and #3. Figure 2 is a sketch in the vicinity of pile #1 and the SE corner of the bay.

The investigation revealed that the area of the collapse coincided with a geological structure trending approximately north-south. The feature is most likely a complex fault due to intersection of more than one fault, possibly a conjugate system. The faulting has caused extensive fracturing and reworking of the material to produce angular sand to coarse gravel size fragments of rock to platy shards of brittle rock bordered by closely spaced blocky rock. The fault extends from between the corner pile and #1 pile and almost to #3 pile which is nearly 20 feet wide. The fault is made up of two distinct components: an altered region and a shattered region.

Photo 6 – Weathered Rock Below Sheeting

Photo 7 New Sheeting Installed Several Feet Away From Original. Note Weathered Condition of Rock

The altered region is approximately 6 feet thick between #2 and #3 piles dipping between 50^o and 60^o to the southwest trending 194^o. It is greenish grey, loose, highly to completely fractured and chloritic comprising silt to gravel-sized lithorelics with released quartz, plagioclase feldspar and mica. The quartz and feldspar are not discolored but the surface of the feldspar crystals is powdery and matte. The mica is dominantly biotite and softened, possibly transitioning to chlorite. There is evidence of oxidation of minerals and other mineralization that indicates accelerated decomposition due to exposure to near surface conditions but there may be a history of hydrothermal alteration in this zone as well.

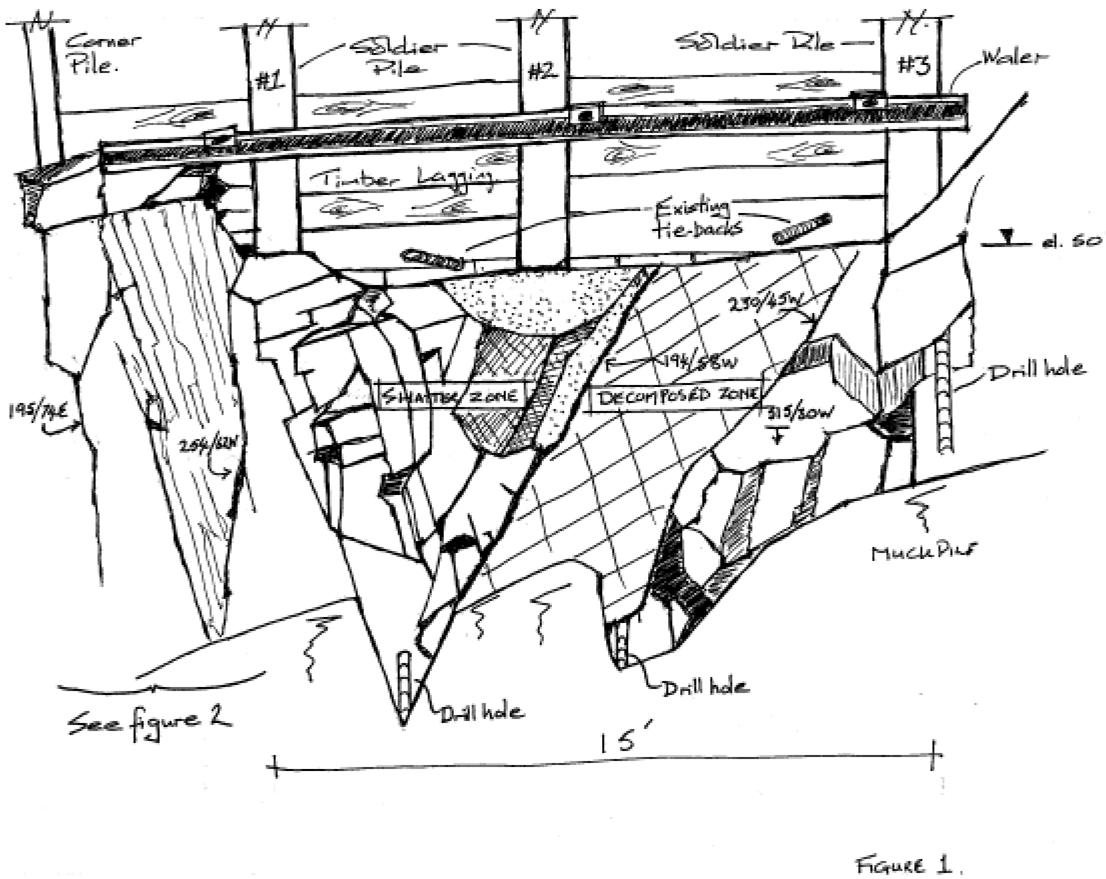

Figure 5 Sketch Rock Condition Under New Sheeting
(several feet away from collapsed area)

The weathered zone extends from the corner pile to pile #1 forming the westerly border of the altered region and is described as dark grey to black mica and possibly amphibole rich schist with extremely close to closely spaced, carbonate, hematite and chlorite coated sub-vertical fractures with sub-vertical conformable seams of decomposed mica schist and vertical convoluted foliation.

These conditions indicate a complex faulted region with significant drag and destruction. It is more than likely that this is an intersection of at least two faults with advanced decomposition and weathering. The product is an extremely weak rock mass with the properties of a loose soil in places.

The rock in this area had been blasted to grade and the line holes could be seen clearly except for the decomposed region where they were not visible. The rock mass behind the line holes could be excavated by a machine bucket because of its highly fractured and decomposed condition.

The faulted ground stood vertically for the time it was exposed. The top of rock was exposed for approximately seven hours before being encased in concrete. The lower slope was still exposed when the observations were complete which is approximately two hours. The material became unstable on contact with hand tools and light excavation machinery and it was excavated easily by hand tools. The decomposed region collapsed on contact with hand tools and rock fragments in the shattered zone could be removed by light pressure from a geological pick.

6 MECHANICAL EXCAVATION

Photo 8 – Area of the Collapse of the Sheeting Is Marked

Excavation at this site began in October 2006. The work was to be performed in accordance with drawings prepared by Giacopelli and in reliance upon a map depicting the results of soil boring tests performed by RA Consultants. The bottom of the excavation was intended to accommodate a top of slab on grade at elevation 52.25 feet. The footings depth was intended to be three or four feet, and should have had a bottom around elevation 47-48. The rock was expected to have been able to accommodate 40 tons per square foot. The drawing instruction required side walls to be embedded in about one to six inches in rock. Grade elevation is around 84-88.

The drawings filed with the Department show that the excavation at this site was planned with a sheeting system supported by soldier piles held in place by steel bars anchored to the geological material around the excavation. The bottom of the sheeting system was designed to have been placed on and supported by competent rock. To reach the intended bottom elevation the excavation had to go beyond the soil layers and into 10 to 30 feet of rock. According to testimony provide by various Mayrich employees, rock excavation was intended to have been performed by mechanical means until doing so became unfeasible due to rock solidity. Solid rock was to have been excavated by blasting.

6.1 Sheeting System Drawings

The drawings show typical lengths of soldier piles, and the required number and location of wales. The drawings show typical sheeting installation for depths of 16-20, 21-25, 26-30, 31-35, 36-40 feet. The drawings required the soldier piles to be placed in a four or five foot deep rock socket for sheeting excavations down to 30 feet. Below 30 feet the drawings require a rock pin at the base and no socket. In lieu of a rock socket, a bottom pin or "anchor" must to be installed. This anchor is not tensioned since it is installed almost entirely in rock.

The soldier piles must be at least W12x53 steel beams with a spacing of eight feet, and the anchor bar must be R51N. Each anchor bar has an indicated anchoring length (grouted) and a free length. For depths under 16 feet a concrete pier is indicated. Size and type of wale and lagging is not indicated on the drawings' typical sections. Lagging is indicated on section BB as 3x10.

6.2 Installation of the Sheeting System

The description of the installation method in this paragraph is based on interviews with Mayrich personnel. Some of the activities were observed as the remedial sheeting was installed.

The installation of the sheeting system commenced after only minimal excavation, and the first step was the drilling of a "bucket" into rock. Drilling stopped after penetrating the rock for a depth determined by the driller. The driller recognized the presence of rock by the fumes and/or dust emanating from the drill, as well as by an increase in resistance to drilling. Thereafter, soldier piles were lowered—not hammered—into the newly-drilled holes.

Next, mechanical excavation started along the pile line and lagging was installed. A shimmed gap measuring one-half to three-quarters of one inch was created between each piece of lagging, and lagging was set in place with nails bent around the soldier pile flange, and set at the back or front of the soldier pile section. Wood blocks were placed between the flange of soldier beam and the lagging. After lagging was installed soil fill was installed and tamped down at the back of the lagging to create direct contact between the face of the soil excavation and the sheeting.

As excavation advanced, wales were placed at or about the location (depth) indicated on the drawing. Next, under the supervision of field engineers, anchors were drilled at angles and lengths provided for on the drawings. Because the anchors measured 10 feet in length, the installed length was often larger than prescribed. The inclination of the anchors was typically 30 degrees from horizontal.

After the anchors were placed they were grouted. Grouting was not done under pressure and it took approximately three days for the grouting to reach its desired strength, at which point the anchors were tensioned according to the value indicated on the sheeting plans. As excavation continued the entire procedure was repeated.

After reaching rock with the excavation, a "line drill" was created in the rock. This line drill was obtained by drilling holes with a two and one-quarter inch bit 8 to 12 inches apart. Such drilling was intended to limit the breakage of the rock along that line, and the line drill was placed as close as possible to the soldier pile lines. About one foot of clearance was usually needed by the driller to clear the existing wales. The result of the line drill was the creation of a face of rock that provided about three feet of clearance from the outside face of the sheeting wall.

6.3 Observations of the Sheeting System

Following the incident a detailed inspection of the existing sheeting system was performed separately by the Department and separately by James Scheld, PE of Shapiro. The findings are as follows.

6.3.1 Controlled Inspection and Anchorage

No records of controlled inspections were kept. Consequently, verification of the adequacy of the anchorage required a series of tests in which the anchorages were tensioned beyond design capacity. Only a test to failure could have provided results with a high degree of reliability; however failure was intentionally not reached because of the risk of collapsing the entire system. Nevertheless, the loads applied, which exceeded the strength necessary for the system, generally failed to disturb the system -- i.e. they failed to create movement at the top of the pile, except in a single instance. In addition, the anchorage nut separated from the pile in a number of instances. A complete description of the testing and result can be found at an appendix. Based on the test results, Scheld opined that anchorage strength was "satisfactory." The Department requested monitoring to continue for the duration of the construct ion work.

6.3.2 Sockets

The drawings indicated that sockets for the piles were to be installed at sheeting depths less than 30 feet, and then filled with lean concrete. According to the drawings, socket depths would vary from four to five feet. During interviews, it was suggested that the sockets were drilled as intended. Visual observations are inconsistent with this characterization. Rock regularly crumbled during the line drilling, and only some piles were found to have been placed in sockets. This crumbling of socket walls could have been avoided if the line drilling had been performed farther away from the soldier piles.

6.3.3 Installation of Sheeting and Soldier Piles in the Area of Collapse

In the area of the incident about 15 feet from the sheeting exists a building foundation with bottom at elevation 71 (7'-10" x 7'-10"). This foundation is about 16 feet above rock, at estimated elevation 55. Given the proximity and disparate elevations, the installer of the sheeting system had to be careful not to penetrate the existing foundation.

Installation of the sheeting in the area of the collapse was further complicated by its required shape, which followed the chords of a circle. This circular shape made it more difficult to join lagging that was not parallel with the soldier pile flanges. Further, the geometry of such a system creates some minor concerns *vis à vis* direction of soil pressure and resolving of tension forces created in the tendons.

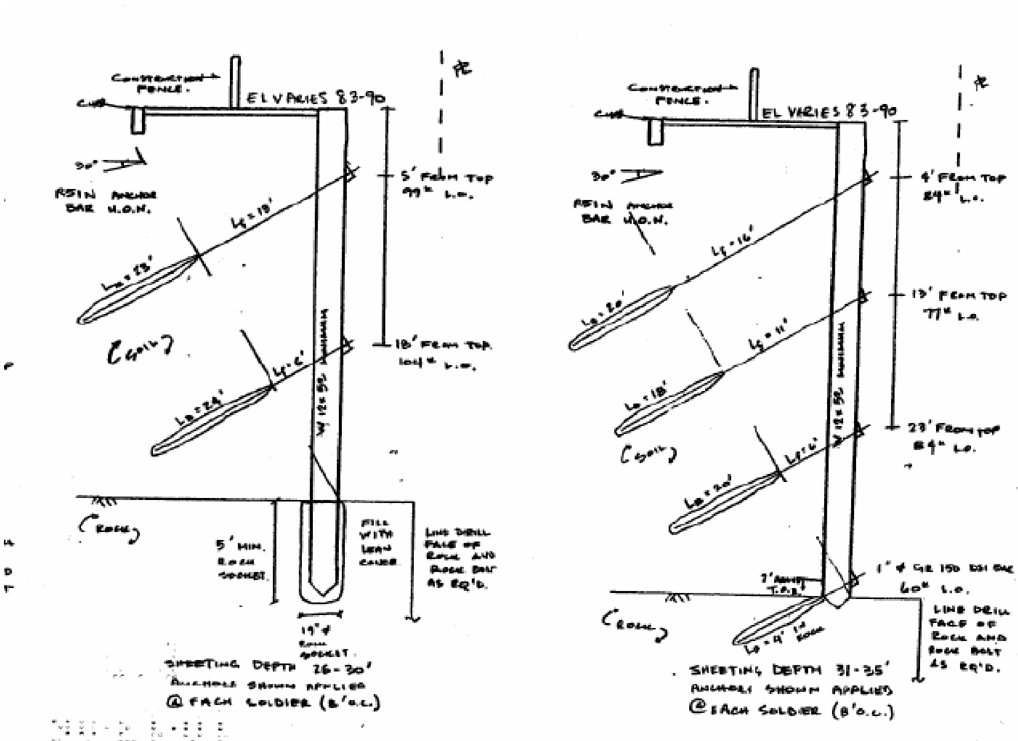

Figure 6 Sheeting Installation (from Drawing No. 2006-45-06s)
Figure shows the approximate bottom of piles obtained from observations of the debris. The drawings require four anchors be installed for piles at sheeting deeper than 30 feet (no socket). According to James Sheld, the piles were HP 12x74, grade 50.

Sheeting at piles #1, #2, #3 was approximately 35 feet. Post collapse investigation found that the piles (#1, #2, #3) were not set on competent rock. See 5.3. Sockets were not required by the design drawings for these piles; as a result, they were not installed. Socket marks were not found during the investigation, and it was revealed that the soil was not competent to sustain sockets. In addition, the pictures and measurements do not give a length of pile sufficient to fit a socket.

A 35 foot sheeting -- the design shown on the drawings -- requires four anchors. Only three were provided. While the drawings place the first wale about 4 feet from top of the soil, observations indicate that in the collapsed area, the first wale in fact was installed 7 or 8 feet below the top of the soil. The investigators assume that this placement could be explained as a precaution taken to avoid drilling the anchor in the existing foundation at 784.

The installation of the soldier piles did not follow the drawings in several ways. Whereas the drawings indicate that the pin at the bottom of the soldier pile should be placed at about two feet from the bottom of the pile, at pile 2, the first anchorage was placed about six feet from the bottom. The piles installed were HP12 x 74, stronger than the required W12x54.

The design required each of the soldier piles to have its toe founded on competent rock. However the exposed rock in the area below piles #2 and #3 is a severely weathered rock which was not the design intent.

7 BLASTING

7.1 Procedures

The procedures for blasting have been determined from discussions and interviews with individuals associated with the project, explosives records and monitoring results.

7.2 Preparation and Planning

The method of preparing and planning the blasting program for the project was described by Anthony Mignogna of Mayrich on 8/29/2007. New York City Fire Department ("FDNY") Chief Jim Lauer corroborated Mignogna's statements and sometimes provided additional explanation. The supplier of the explosives, Austin Powder Company, submitted to FDNY a proposed blast design that included the spacing of the holes, the configuration of the charge in each hole, and the detonation sequence. The design was subsequently authorized by FDNY.

A test blast was carried out on the site and the FDNY and a representative from Austin Powder Co., Vic Sternum, was present. The purpose of the test was to check that the round did not generate a peak particle velocity (PPV) greater than 1.0 inches per second (ips). The results of the blast that day, which was March 12, 2006, were monitored by Metric, an instrumentation and monitoring company.

According to FDNY, blasting approval was given on the spot as the test was deemed successful and production blasting followed the same day. The first shots on record represent the vibrations produced during the test. As described, the test essentially comprised blast holes drilled to 12 feet using a three feet by three feet pattern with several load combinations. Because the vibrations did not exceed the required ips, approval was given. According to Metric, it did not know that these blasts were part of a test. None of the parties involved created a formal record of the test blast.

The holes for the blasting scheme were pre-drilled to depths calculated by Mayrich to achieve the required fragmentation to sub-grade level. This ranged between 8 and 12 feet depending on the profile of the existing rock bench. The outer profile of the bay was line drilled with closely spaced holes at approximately 8 inch centers. Reliever holes were drilled approximately two feet in from the line holes at two-foot centers followed by the standard three feet by three feet pattern. It is unclear whether the reliever and line holes were charged or even actually installed. Some site personnel said they were; others said they were not. The actual blaster stated that they were *not* charged, and the line holes observed in the wall at the location of the failure did not show signs of being charged.

Mayrich did not furnish the investigators any other documentation on the blast plan. Specifically, there is no design of the hole layout, detonation sequence, calculation of charge weights per delay, powder factor, predicted peak particle velocity or scaled distance.

The first phase of blasting took place between March 12th and April 2nd. Blasting could not continue into the bay because of access restrictions. However, the blaster stated that he was told that he would have to return at a later date to blast the rock in this area because it was known to be hard and not suited to the large breakers used elsewhere on the site.

Photo 9 – Marks of Drillings

The bay was line drilled and all the blast holes were drilled and marked. The blaster noted during drilling the blast holes that the ground was soft in the area of the failure. However, he did not consider it significant enough to alter the round or notify others.

Photo 10 – Face of Rock after Blasting on 7/25/07

7.3 Blasting Records and Material Usage

Blasting occurred at the following dates:

3/12/2007
3/14/2007
3/19/2007
3/21/2007
3/23/2007
3/28/2007
2/4/2007
7/24/2007
7/25/2007

The blasting records submitted to FDNY as required by regulations comprise three tables per day: a) Nonel Blasting Cap Record; b) Dynamite Record; and c) Shot Record and Blaster's Daily Report. These records were submitted for all days blasting took place.

Additionally, FDNY had asked for seismograph monitoring and submittal of respective daily seismograph readings. For March 12th, March 13th and March 19th, Metric submitted seismograph records that also include more details of the blast round, specifically:

- number of holes
- hole depth
- hole diameter
- hole spacing
- total weight of explosives
- type of blasting cap
- delay series
- maximum holes per delay
- maximum charge weight per delay
- stemming height.

It should be noted that only the velocity measurements were recorded by Metric for March 21st, March 23rd, March 28th and April 2nd. No seismograph monitoring took place on July 24th and 25th (the date of the collapse).

7.3.1 Discussion of Records – Pre-Incident

Distance of blast location to seismograph. This distance is a critical parameter for understanding the response of the ground to the blast. On the Metric submittals the distance of the blast from the seismograph is written as approximate with the symbol "~" and the number does not change with subsequent blasts on a given day. In fact, for the first 27 blasts the distance between the source and the seismograph is given as "~100w," which is taken be approximately 100 feet west of the source. The Contractor's description of the method of blasting is that the face was advanced progressively by each blast. Accordingly, this parameter must have changed after each blast.

Explosive and weight. The Shot Record provides the number of caps used and the number of sticks of explosive for each blast. The Contractor was using Emulex 927 which weighs 0.88 pounds per stick so the total weight of explosives used per round can be calculated. The total weight of explosives shown in the shot record matches the Metric record.

Caps and delays. The number of holes is given in the Metric record and it shows that one delay per hole was used. This does not match the Shot Record. In fact, the Shot Record shows *twice* as many delays were used per blast round. Assuming the "number of holes" record is correct, it seems more than one cap was used in some of the holes. However, because the Shot Record form does not require entering the number of holes that were charged, and the blaster has stated that more than one cap was used per hole, it is impossible to know for certain. Indeed, it appears that even three caps may have been used occasionally. As for delays, in addition to delays between holes, delays within the same holes might have been used. The delay sequence of the multiple caps in each hole is not known.

The lack of complete records and the discrepancies between sources of information mean that calculating the scaled distance and predicting the likely PPVs at the time of the failure or for any other part of the project is unreliable and potentially misleading. The only reliable information is the total amount of explosives used.

7.4 *Instrumentation and Monitoring of Vibration*

The seismograph locations were decided by FDNY. The purpose of seismographic instrumentation was to ensure that the blasting did not exceed a PPV of 2.0 ips (inch per second). In fact, ideally blasting was to be limited to 1.0 ips, and readings greater than 1.0 ips were to be reported by Metric to the blaster so that appropriate adjustments could be made to reduce the PPV. According to the blaster, the PPV did not exceed 1 ips during this blasting phase of the job as far as he knew.

The initial instrumentation and monitoring of blasting, "phase one," occurred between March 12th and April 2nd, and it was carried out by Metric. Metric installed Geosonics seismographs in proximate buildings, including the building closest to the failed wall, 784 Columbus. These instruments record velocity and frequency in orthogonal directions and the vector sum is calculated from the raw data. The vector sum was provided for all phase one blast events except for the first day, March 12th. The last daily record from Metric is for 2nd April 2007.

The second phase of instrumentation and monitoring was supposed to have been carried out by Dayton Inspection Services, Inc of Delanco, New Jersey. However, according to Val Norets, Stellar Management omitted to call Dayton to monitor on July 24th and 25th. However, Mayrich and the FDNY assumed the instruments were installed, and that Stellar and its General Contractor, Gotham, would have notified the instrumentation contractor to be on-site to take the measurements.

7.5 *Vibration Measurements*

7.5.1 Particle Velocity

As explained above the exact location of the blast in relation to the instruments was not recorded in the information provided. Thus, there is little else that can be done to understand the data other than refer to measured particle velocities (PPVs). The daily logs for the monitoring in March and April show PPVs range from 0.2 ips to 1.4 ips. The number of events less than 0.5 ips and the number between 0.5 and 1.0 ips are the same at 47%. 6% of events exceeded 1.0 ips.

The high PPV values (0.9 ips or greater occurred) in a cluster between March 19th and March 21st. Four of the last five blasts on March 19th and the first five blasts on March 21st were 0.9 ips or greater. Notably, these high PPV values do not coincide with the high charge weights (assuming the assumptions described above are valid). There is no record that these values were communicated to the blaster.

7.5.2 Vibration Frequency

It is clear from the vibration records that with one exception all vibration frequencies exceeded 20 Hz. Most of the frequencies were in the 40-60 Hz range. This compares well with the usual building structures frequencies that are in the 5-10 Hz range.

The graph of particle velocity versus frequency meets the criteria of the Bureau of Mines study. These data are represented by the pink line in the chart. The particle velocity also met, except in two cases, the requirements of FDNY.

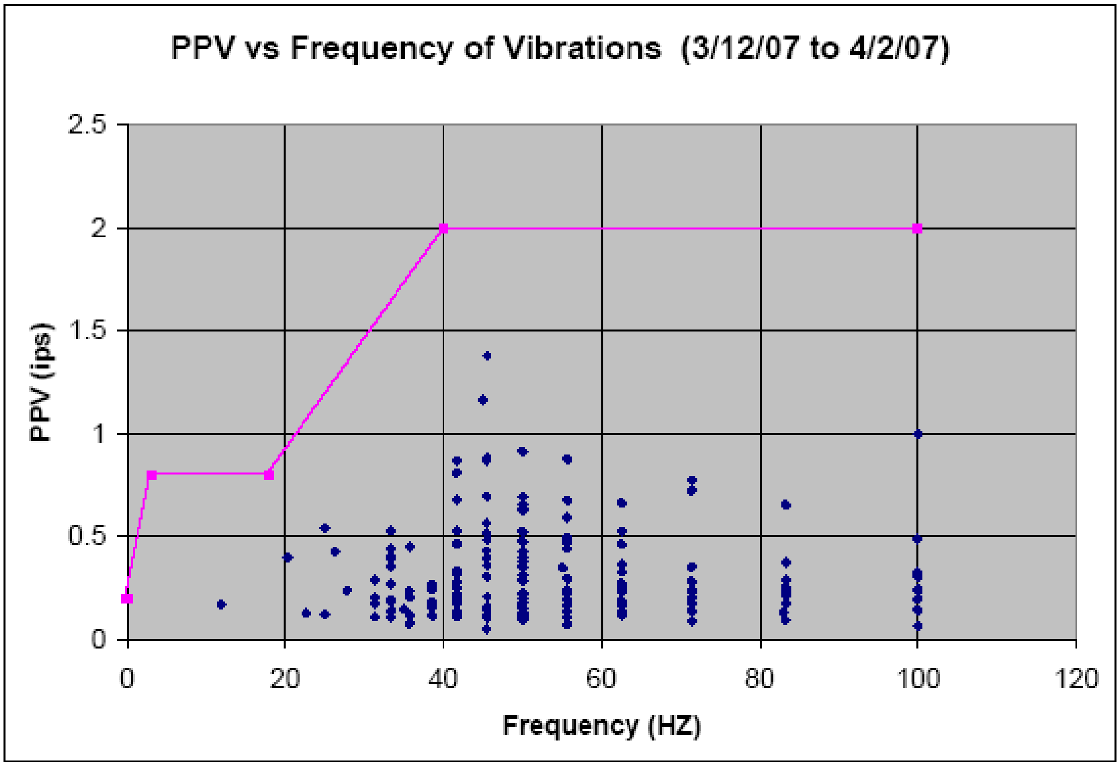

Figure 7 – Particle Velocity v. Frequency

7.6 Blasting on the Day of the Incident

The Blasting, Cap and Shot Records for the day of the collapse, July 25th, are provided and have been examined. As mentioned previously, there are no seismograph logs for this day.

The available records show that there were nine blasts on the 25th, and that the final blast of the day was at 3:20 p.m. The records do not show where the blasts were located. It was understood from discussions with Mignogna, that the method was to blast crescent shaped rounds approaching the bay. This testimony is not strongly supported by photos. In photo 10, the bench appears almost straight, not rounded.

Mignogna recalled that the last blast in the vicinity of the failure was "around lunch-time." The Shot Record shows that blasts took place at 11:20 a.m. and 1:00 p.m. The round at 11:20 a.m. used 150lbs of explosive and the round at 1:00 p.m. used 250lbs. If it is assumed that the blaster used two caps with two delays per hole, the charge weight per delay is 4.2 pounds and 3.3 pounds, respectively. If the blaster used one delay per hole the charge weight is double this value, 8.4 pounds and 6.6 pounds.

7.7 Effect of Blasting

7.7.1 Potential Effects – Airblast and Flyrock

Blasting has the potential to develop several effects. "Airblast" is a vibration propagated through air. It manifests mainly as noise and vibration discomfort to the public. "Flyrock" are the small rocks that project out from the site of the blast. Here, while there is evidence of Airblast in the form of 311 complaints to the Department, there are no allegations or evidence of Flyrock. This is most likely due to the blaster's use of Blast Mats, which are the industry standard for preventing Flyrock.

7.7.2 Potential Effects – Fragmentation and Vibrations

"Fragmentation" is the breakage of rock in the immediate vicinity of the blast. "Vibration" refers to the shaking of rock in the immediate vicinity of the blast. Fragmentation and vibrations can cause over-excavation, undermining of adjacent wall foundations, and damage to the sheeting system. More specifically, the amount of energy developed during a blast that is not spent in noise and fragmenting the rock is used-up as ground vibrations. Soil vibrations are transmitted to the various structures around the blast. This vibration develops accelerations in the various components. Forces and stresses develop in the materials.

In fact, the effect of vibrations on adjoining buildings has been the subject of various studies, most of which were conducted with an eye towards developing regulations that would protect buildings in areas adjoining blasts. The basis of blast induced vibration regulation is the Bureau of Mines Report 8507: Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting, a report that expresses vibration limits as a combination of frequency and particle speed. The report concludes that "practical safe criteria for blasts that generate low-frequency ground vibrations are .75 in/sec for modern gypsum board houses and 2 for frequencies above 40 HZ, a safe particle velocity maximum of 2.0 in/sec is recommended."

It should be noted that the Bureau of Mines study was performed on one- or two -story wood or masonry houses, and concerned blasting that led to the formation of mostly cosmetic cracks. Consequently, its recommendations regarding acceptable blasting vibrations must be considered in light of the nature of the potentially affected building, 784 Columbus, which in this case is a sturdy urban tower comprised of substantially more robust materials than the ordinary one- or two-story house.

7.7.3 Potential Effects – Discomfort for Public

Vibration resulting from blasting operations can be annoying and discomforting to the public, even when the blasting procedure is safe. The human reaction to vibration is subjective and many studies have been performed to analyze the level of public complaints. See attached graph from EM 1110-2-3800. An interpretation of seismic readings from the Metric monitoring places some of the vibrations between the unpleasant and intolerable level. This explains the complaints from residents of 784 Columbus.

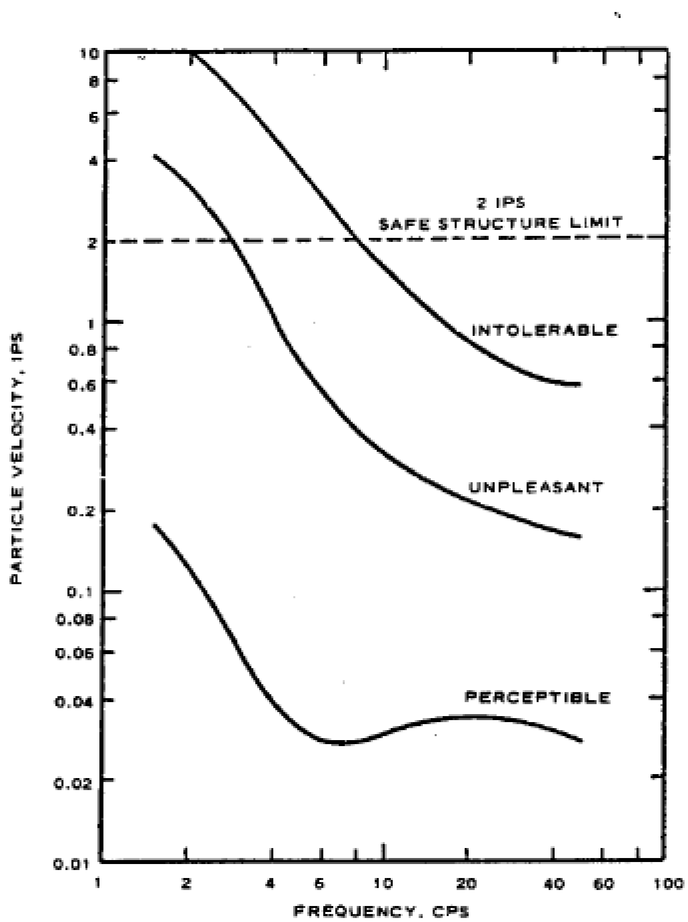

Figure 8 Subjective response of the human body to vibrations
From EM 1110-2-3800

7.7.4 Observed Vibration Effects on 784 Columbus

Inspections of several apartment interiors performed immediately after the accident did not reveal any crack that could be attributed to blasting or the collapse of the sheeting and subsequent soil displacement. In the days following the collapse, additional apartments were inspected. These inspections failed to uncover any signs of structural distress or even cosmetic defect attributable to blasting.

The absence of cracks on the building interior and exterior is consistent with the vibration measurements performed by Metric (see Figure 7). The frequency is higher than 20Hz. Of the few cracks observed in the building practically none could be attributed to blast. This is a positive indication that the level of blasting that occurred without dynamic monitoring was within acceptable limits for cosmetic damage of permanent buildings.

7.7.5 Observed Blast Vibration Effects on the Sheeting System

Blasting had no observable or discernable effect on the remaining sheeting system although some of these sheeting areas were exposed to similar explosive loads and distances from the blast holes

8 ANALYSIS AND DISCUSSION

8.1 Computer Model

8.1.1 Design Wall System

The original design of the temporary soldier pile and lagging wall, and the anchor system is shown on Drawing No. 2006-45-06, Revision 1 prepared by Mayrich. Figure 6 shows the design drawing for the wall section that failed.

Each soldier pile consists of a W12x53 steel section, and the soldier piles are spaced at eight-foot centers. The drawing does not provide the grade of the steel section. The soldier pile is designed to support on rock, and no rock socket is required.

The wall is designed to be supported by four anchors during excavation. The top three anchors are R51N anchor bars at four feet, 13 feet and 23 feet below ground surface. The pre-stress for these anchors are 84 kips, 77 kips and 84 kips for the top, second and third anchors, respectively. The bond length of these anchors is shown to be 20 feet or 18 feet in soils. The bottom toe anchor of the wall is a one-inch diameter Grade 150 DSI bar, pre-stressed to 60 kips. The toe anchor is located two feet above the rock, and the bond length in rock is four feet. The method of grouting of the anchors is not specified.

8.1.2 As-Constructed Wall System

The as-constructed wall system consists of soldier piles at seven and one-half to eight feet centers. Measurement of the recovered piles indicated that HP12 x 74 Grade 50 steel sections were used for each solider pile. The pile size is also confirmed by Mignogna. Soldier piles #1, #2 and #3 were founded on severely decomposed rock, and the decomposed rock extended 6 feet below the toe of the soldier piles. Soldier pile 4 was founded on competent rock.

The as-constructed wall was supported by three anchors instead of four as provided on the design drawing. The elevations and depths of the anchors are summarized in the table below.

		Pile 1			Pile 2			Pile 3			Pile 4		
Ground Surface (Elevation-ft)		85			85			85			85		
Toe of Pile (Elevation–ft)		48			48			49			56		
Retained Height (ft)		37			37			36			29		
Number of Anchors		3			3			3			3		
Anchors Information	Location	Top	Mid	Bot	Top	Mid	Bot	Top	Mid	Bot	Top	Mid	Bot
	Elevation (ft)	77	66	54.5	77	66	54.5	77.5	67	55	77.5	67	57.5
	Depth below GS (ft)	8	19	30.5	8	19	30.5	7.5	18	30	7.5	18	27.5

	Bottom Anchor above toe of pile (ft)	NA	NA	6.5	NA	NA	6.5	NA	NA	6	NA	NA	1.5

Figure 9 Description of Anchoring System

Each anchor consists of self-drilling R51 anchors (two-inch outside diameter) manufactured by Belloli. No construction records of the anchors are available, and therefore the as-constructed total length, bond length and the pre-stress loads applied to the anchors cannot be determined or verified. The anchors were grouted.

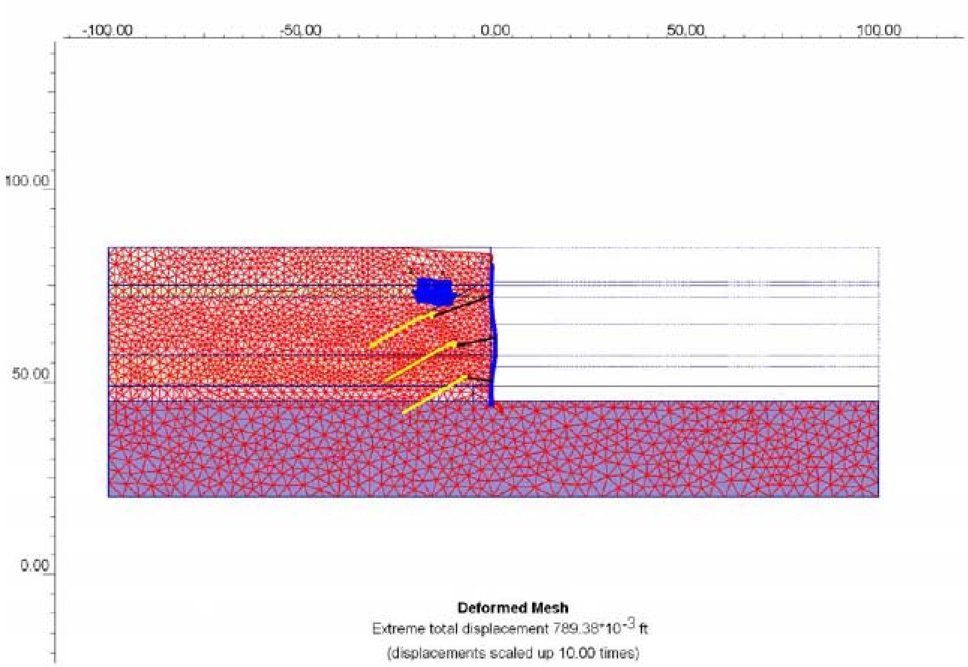

Figure 10 Total Displacement

8.1.3 Lateral Stability Check of the As-Constructed Wall System

The stability of the as-constructed wall system was checked using computer program FREW (Flexible Retaining Walls). The program performed an analysis that would check the lateral stability of the wall system assuming the wall was founded on competent rock. The soil was simulated as a series of springs with limiting active and passive pressure. The numbers associated with these “springs” were fed into the program and the outputs created were estimates for wall deflection, bending moment, shear force and anchor loads.

The Mohr Coulomb soil model is used in the FREW analysis with the soil properties given in the table below. Analysis is carried out based on the as-constructed wall system of solider pile 2.

Material	Total Unit Weight □$_s$	□’ (degrees)	c or S_u (ksf)	E’ (ksf)	Top of Stratum	Bottom of Stratum (EL.ft,

	(pcf)				(EL.ft, BMD)	BMD)
Fill	120	33	0	300	82	75
Clay	120	0	1	500	75	72
Till	130	38	0.1	1250	72	57
Decomposed Rock	130	42	0.2	3100	57	To toe of pile

Figure 11 Soil Properties

Since pile 2 is the deepest pile with the largest distance between the bottom anchor and the pile toe, that pile would provide the largest wall moment and forces in the anchors. The anchor lengths were assumed to be similar to the design drawing, using a bond length of 20 feet. Likewise, the pre-stress loads were assumed to be the same as the design drawing.

The existing building 784 Columbus Ave is supported on shallow footings. The footing load is applied to the FREW analyses as six ksf vertical pressure acting at elevation 71 ft, 10.5 feet from the wall. The following construction sequences are analyzed in the FREW analysis:

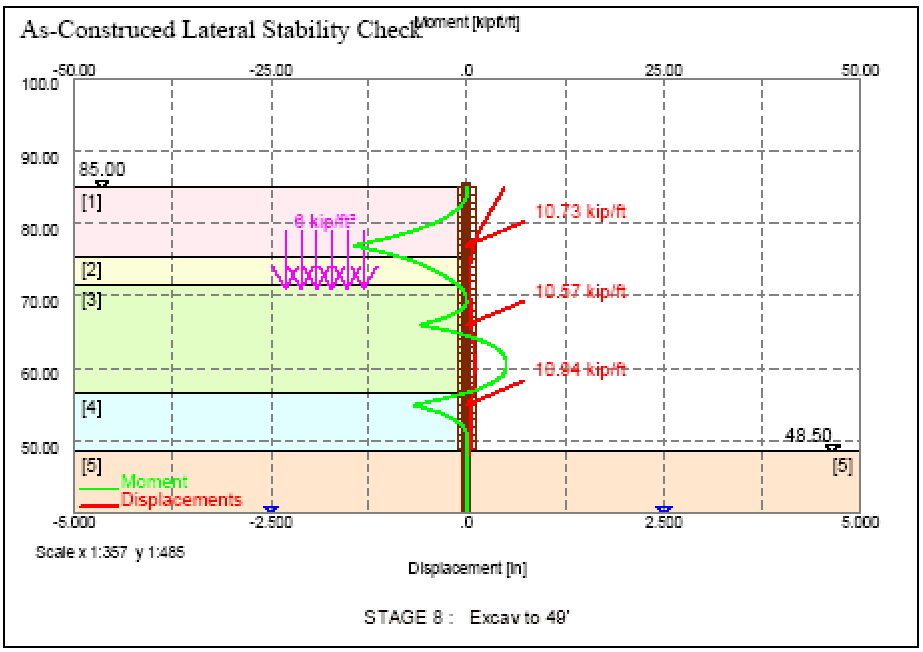

Figure 12 FREW Analysis –As-Constructed wall design check

(a) Set up initial stress, including the existing building footing load
(b) Install soldier pile and lagging wall
(c) Excavate to EL. 76 ft
(d) Install Anchor 1 at EL. 77 ft, preload to 84 kips
(e) Excavate to EL. 65 ft
(f) Install Anchor 2 at EL. 66 ft, preload to 77 kips
(g) Excavate to EL. 54 ft
(h) Install Anchor 3 at EL. 55 ft, preload to 84 kips
(i) Excavate to bottom of pile

.

		FREW analysis	Allowable Capacity	Ultimate Capacity
Bending Moment (kips-ft)		121	165	275
Anchor Load (kips)	Steel Tensile Capacity	88	113	188
	Pullout Capacity		70	104

Figure 13 Summary of Results of FREW analysis

The FREW analysis result shows that while the as-constructed sheeting system has adequate bending moment capacity for the soldier pile and the steel tensile capacity of the anchors, the anchor loads are higher than the allowable pullout capacity. For anchors installed in dense soil using gravity grouting method, the typical design ultimate bond stress between ground and soil is between four to five ksf. Using five ksf ultimate bond stress, a four-inch diameter drill hole, 20 feet bond length, and a factor of safety of 1.5, the allowable pullout capacity is 70 kips.

Ultimate bond stress depends on myriad factors, including the method of drilling, the overbreak and stability of the drilled hole, the properties of the soil and the method of grouting and in-situ stress in the soil. As a rule of thumb, the ultimate bond stress of five ksf in the very dense till or decomposed rock should be conservative.

8.1.4 Sensitivity Analyses

Based on the FREW analysis of Pile 2, additional stages are added to the analysis to investigate the sensitivity of the wall system. Four sensitivity cases are analyzed and the description of each case is listed below.

Case 1) Adding 25% hydrostatic pressure behind the wall to simulate partial water pressure built up behind the wall.
Case 2) Check the stability of the wall system assuming top anchor fails.
Case 3) Check the stability of the wall system assuming middle anchor fails.
Case 4) Check the stability of the wall system assuming bottom anchor fails.

		Allowable Capacity	Ultimate Capacity	Case 1	Case 2	Case 3	Case 4
Bending Moment (kips-ft)		121	275	151	167	126	309
Anchor Load (kips)	Steel Tensile Capacity	113	188	90	100	92	279
	Pullout Capacity	70	104				

Figure 14 Sensitivity Analyses for Four Cases.

The results show that for cases 1, 2 and 3, the wall and the anchors would have adequate ultimate capacity. However, for case 4 (bottom anchor fails), the anchor load in middle anchor would have exceeded the ultimate capacity of the anchor, and the anchor would break or pullout, causing progressive failure of the wall.

8.1.5 Effect of Decomposed Rock below Toe of Wall

The effect of the presence of decomposed rock or weak soil mass below the toe of the wall is analyzed using the finite element program Plaxis. Plaxis is a two dimensional finite element soil-structure interaction program, which can handle staged construction sequences.

For excavation to the toe of the wall, the Plaxis analysis uses the same construction sequence, soil parameters and building surcharge load as the FREW analysis, assuming the pile is founded on rock. The deformed mesh of the excavation stage to the toe of the pile is shown below in Figure 15. The maximum anchor load is 105 kips, and the maximum bending moment in the wall is 143 kips-ft.

After the excavation reaches the bottom of the wall, two more analyses were added to investigate the effect of a weak rock mass below the toe. The first analysis simulates four more feet of excavation below the toe of the wall, and replaces the material below the toe of the wall with Decomposed Rock. The analysis shows the toe material becomes unstable and the analysis cannot converge. However, from the last step of the analysis, it shows the wall moves laterally outward and downward. The deformed mesh and the displacement vector at the bottom of the wall of the last step are shown below.

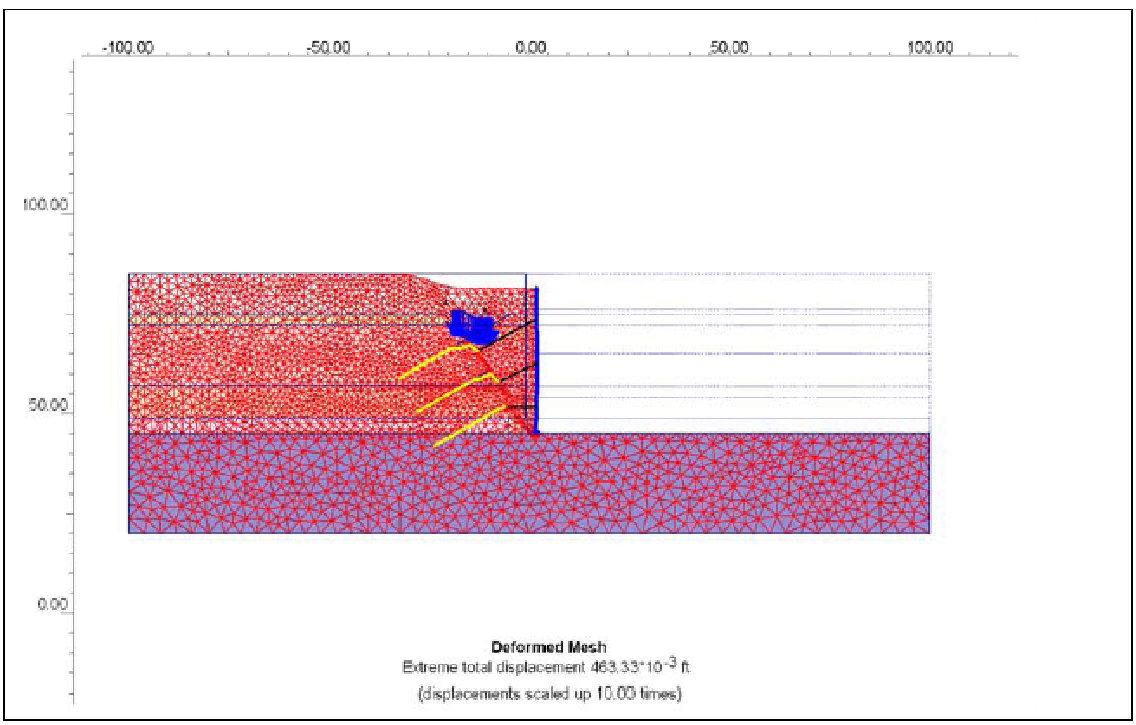

Figure 15 –Plaxis Analysis Deformed Mesh for 6 inch prescribed settlement of the wall

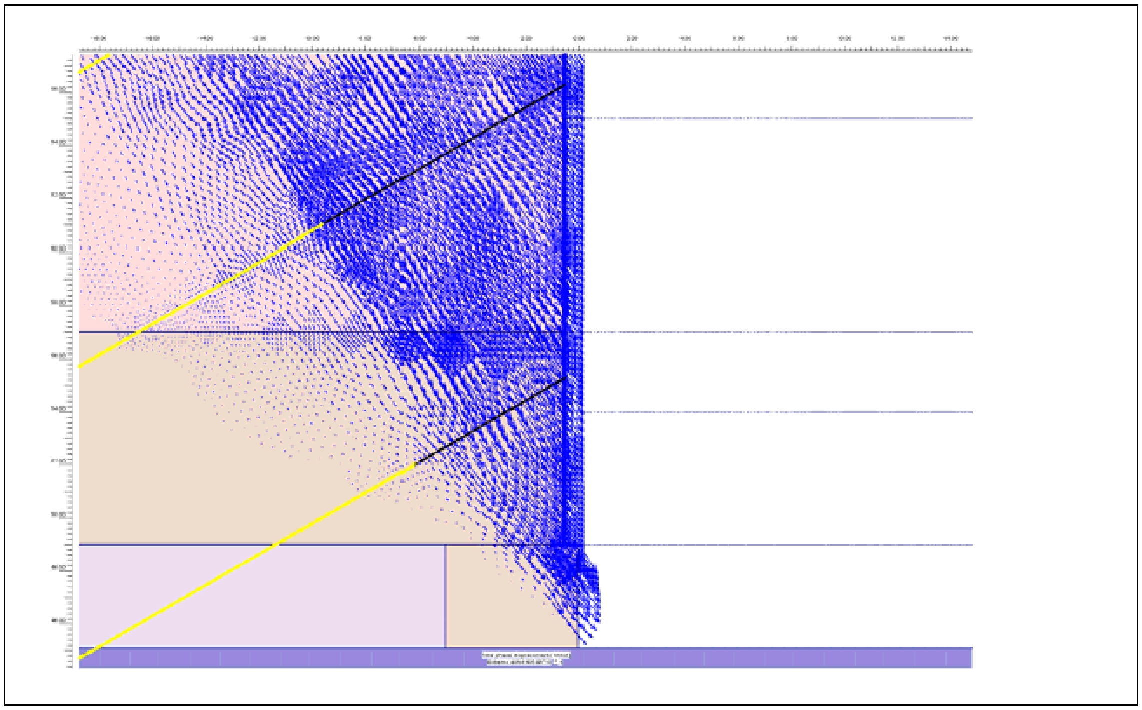

Figure 16 Plaxis Analysis – Vector Displacement Vector for four additional feet of excavation

The second analysis simulates four more feet of excavation below the toe of the wall, replaces the material below the toe of the wall with soft elastic material and prescribes a 6-inches settlement to the wall. The deformed mesh of the analysis is shown below. The load in the bottom anchor is 302 kips.

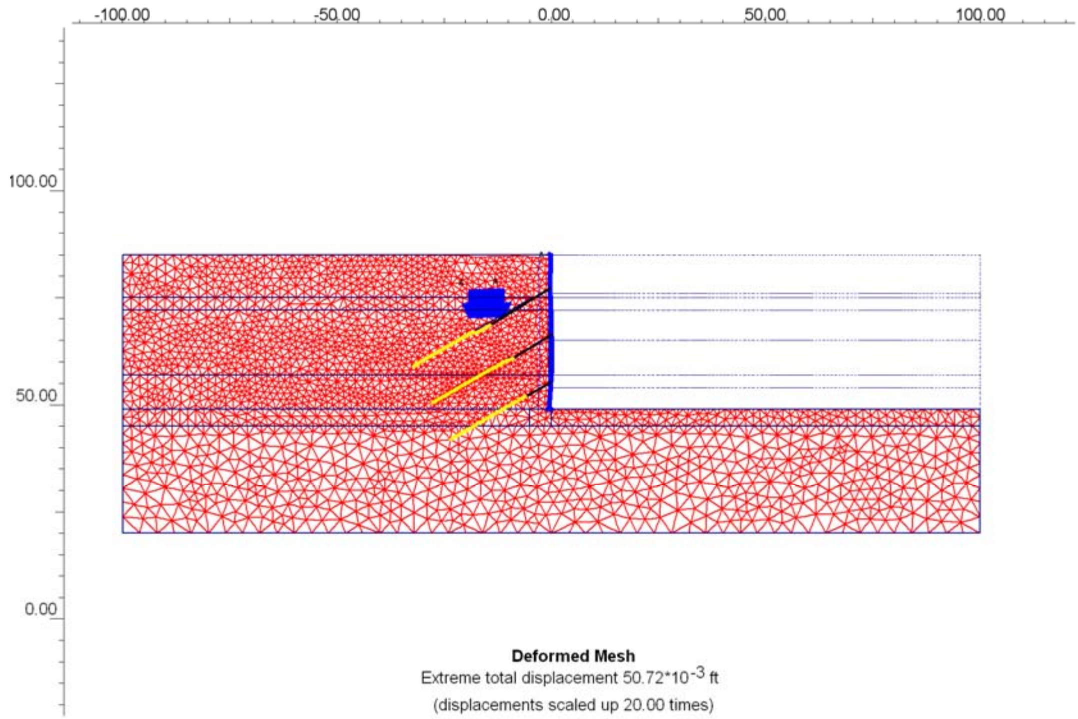

Figure 17 Plaxis Analysis –Deformed Mesh for excavation to bottom of pile

Both of these analyses show that the toe of the wall becomes unstable with the presence of the weak rock mass below the toe, causing downward and outward movement of the wall. This movement would have caused the bottom anchors to fail in tension or pullout, subsequently causing progressive failure of the middle and top anchors.

8.2 Blast Related Calculations

Typically, there are two types of blasting calculations. One type of calculation seeks to predict the vibrations at a given distance for certain blasting conditions e.g., type of rock or charge weight. The results of such calculations are then compared with limits established by regulatory authorities or deduced from engineering research. A second type of calculation seeks to establish a certain blasting protocol e.g., bench hole diameter and pattern, or charge weight, to obtain a certain fragmentation of the rock. For such calculations imposed vibration criteria are a given.

Neither calculation needed to have been conducted with regard to the blasting operations in question, and the blaster must have determined as much following the successful test blast. The fact that 784 Columbus did not even sustain cosmetic damage corroborates this judgment.

Definitive calculations regarding the effect of a blast on rock underlying the sheeting were not feasible for this analysis. Indeed such calculations are not generally available or required for most projects. The same holds true for calculations regarding the soil behind the sheeting and for the effect of vibrations on the sheeting system or rocks. Such calculations would require substantial additional supporting data, including the complete time history of each blast in the area (described in millisecond intervals). As previously mentioned, no such records are available.

Accordingly the following calculations are presented just to establish a possible order of magnitude of the particle velocities for the event. The calculations were prepared using data that was not collected with the intention of performing such calculations. The accuracy of the data is relatively low because of these facts.

8.2.1 Calculation of Peak Particle Velocity for Different Distances and Charge Weights

There are two sources of data for charge weight: the records of the contractor, Mayrich and the records of the instrumentation and monitoring firm, Metric. The Metric data are superior in that they include more information, such as number of holes and velocity measurements but they are only available for part of the blasting program. There are no records for the closest blasting carried out.

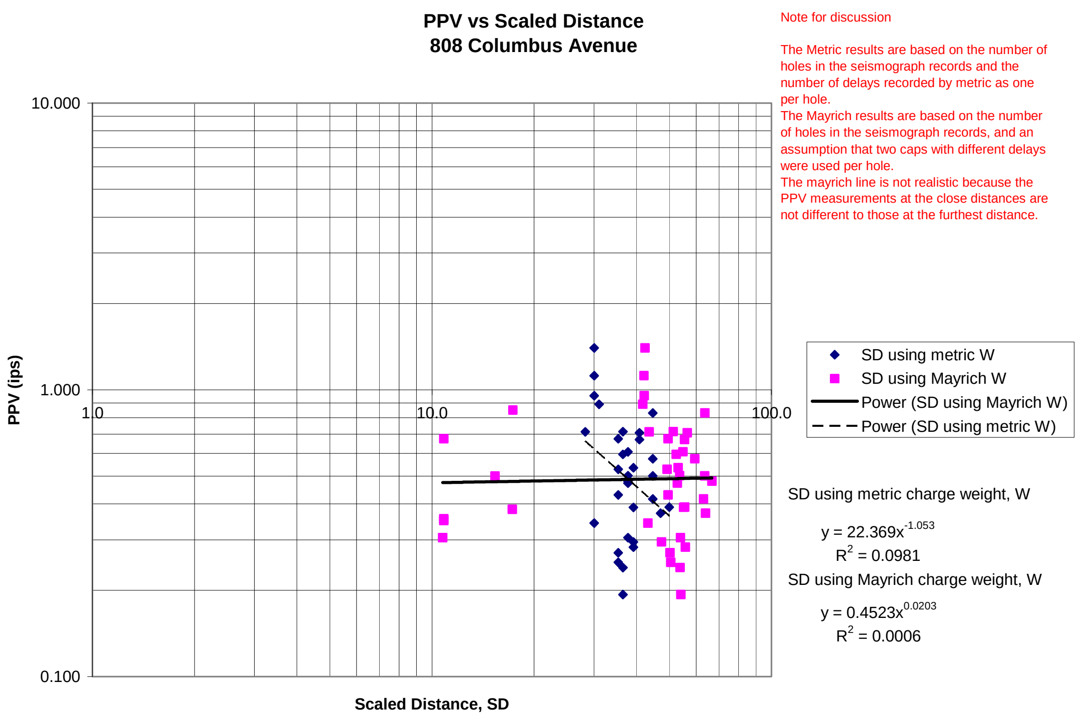

Figure 18 PPV vs Scaled Distance.

The PPV vs. Scaled distance (SD) plot using the Metric data for charge weight, distance and vector sum velocity gives the following relationship:

PPV = 22.369SD-1.053
R2 = 0.0981

This is in reasonable conformance with the empirical formula

PPV=K.SD-1.6

Where K is a ground related constant. The recommended K values have a very large range and recommendations are much higher than 22 but this was determined by analysis of the measurements and so it should be used for further calculation

This is a worst case condition because it assumes that there is only one delay per hole whereas the indications are that the contractor may have used two delays per hole

PPV (ips) for various distances and charge weights for the Metric formula						
	Charge weight, W (lbs)					
Distance (ft)	2	3	4	5	6	7
7	4.2	5.1	6.0	6.7	7.4	8.0
10	2.9	3.5	3.5	3.5	5.1	5.5
13	2.2	2.7	3.1	3.5	3.9	4.2
16	1.7	2.2	2.5	2.8	3.1	3.4
20	1.4	1.7	2.0	2.2	2.5	2.7

Figure 19 PPV for Various Distances and Charges

The highest measured PPV of 0.9 to 1.5 ips occurred when the blasting was between approximately 85ft and 45 ft from the instrument in 784 Columbus Avenue. The charge weights at this time were generally higher than normal between 8 lbs to 13 lbs per hole (or per delay if 1 delay per hole assumed). Therefore, the PPV of 2.7 ips for 7lb charge weight 20 ft from the building seems reasonable.

This gives much higher PPV values for the assumed charge weights 7ft from the sheeting. However, the theoretical basis for blast analysis in close proximity is underdeveloped and it is suggested that the conventional square relation should be changed to a cube relation for scaled distance. However, there are already too many unknowns to meaningfully advance the analysis further.

9 PROBABLE PHYSICAL CAUSES OF FAILURE

There are two fundamental probable causes of failure of the sheeting system: failure of the ground on which the sheeting was relying for support and failure of the sheeting components themselves.

9.1 Loss of Capacity of Material Underlying the Sheeting

9.1.1 Loss of Capacity Due to Effects of Blasting

The soldier piles should be founded on ground that is able to withstand the applied loads from the piles during installation of the piles themselves, during installation of the lagging, and for the duration of the construction of the permanent works. The foundation material must not deform to the point where the sheeting components are overstressed or displaced to a point where the retaining wall no longer performs as intended by the design. Blasting can cause of loss of bearing capacity of the ground by a) inducing settlement, b) excessive fragmentation, c) sliding of rock blocks and/or d) removal of confinement.

Settlement. Settlement is normally associated with submerged relatively fine grained well-sorted soil and so is not considered to be a likely consequence of blasting in this location.

Fragmentation. Fragmentation of rock in totally confined conditions generally occurs in an area limited to about 20-30 times the radius of the blast hole. (Note that if the rock mass is not totally confined the amount of fragmentation and other damage from the blast becomes more unpredictable because the degree of blast damage) The degree of blast damage to the rock mass is strongly influenced by the charge weight per delay, the blasting sequence, the round configuration, the line drilling, and the rock properties. Fragmentation is normally greater in brittle rocks than highly fractured, porous or weak rocks but in some cases their inherent weakness renders them vulnerable to excessive damage from the blast. Therefore, it is good practice to avoid blasting in very poor quality rock because of the unpredictable nature of the ground response and the economy of excavation by mechanical means.

There is some likelihood but no definitive evidence that there was a degree of excessive fragmentation at the base of the wall due to close proximity blasting. Indirect evidence is the absence of line drill holes in the shattered and decomposed ground and a remnant face that was behind the adjacent line drilling. Furthermore, the analysis in 11.3 indicates that close proximity blasting in these weak rocks could cause failure at the ppv values predicted from the scaled distance analysis. However, there is no established reference for predicting blast damage in the type of ground encountered at the base of the failed wall. As a result, any conclusion that the blasting caused fragmentation at the base of the failed wall here would be speculative.

Sliding of Rock Blocks. Induced sliding occurs along discontinuities such as bedding planes when the vibration across the plane causes uplift of the block, temporarily reducing the friction along the plane causing the block to slide. The faulted ground in this location is so shattered and decomposed that there are no dominant discrete planes. As a result, sliding due to blasting is unlikely to have been a probable cause.

9.2 Removal of Confinement

When rock is excavated it exposes a vertical wall that is normally stable in strong rock. However in weak decomposed and shattered rocks at the location of the sheeting system that collapsed, the ground does not have sufficient strength to stand vertically when exposed. The exposed rock tends to weaken and ultimately slump away from the footings causing a loss of bearing capacity. These conditions were found here and it is our opinion that such a loss induced the collapse.

9.3 Failure of Sheeting Components: Increased Load

9.3.1 Due to Vibrations

Some increase in load on the piling system due to inertia forces induced by the blast might have been possible, but the calculations show that there was significant available capacity in the installed system. The increase in load should have been over 40% to initiate collapse. Vibrations due to blasting could not have accounted for such an increase. Furthermore, collapse due to inertial forces would have developed immediately following the blasting.

9.3.2 Due to water accumulation

Water infiltration was present on the site, and a broken pipe was found in the incident area although it is not clear whether the breakage preceded the incident or not. In addition, two days before the incident there was a considerable rain with flash flood conditions throughout the region. Nevertheless, the setting of the sheeting boards was such that water could not have accumulated behind them. More importantly, calculations considering an increase of the hydrostatic pressure of 25% show that a sheeting system similar to the one in question would not have collapsed due to the presence of water.

Photo 11 – Broken drainage pipe at collapse area

9.4 Failure of Sheeting Components: Insufficient Capacity

9.4.1 Improper Design

The design was adequate, and the calculations performed for verification were in fact based on the field installation that had not properly followed the submitted drawings. They still show adequate capacity in the system to prevent failure.

9.4.2 Improper Installation

As shown in 6.3.3 the installation of wales did not follow the drawings submitted to the department. Calculations show that even the substandard installation had the capacity of retaining the soil albeit with a smaller factor of safety**.** As shown in the geometric analysis, the lack of anchor set close to the toe (pile 2) created a weakness in the system that amplified the soldier pile vertical movement**.**

According to the design, the soldier pile was supposed to be founded on competent rock, not decomposed rock**.**

9.4.3 Loss of Anchorage

Calculations were performed for failure of tendons at various locations. Loss of anchorage at the top of the system would not have resulted in collapse whereas loss of anchorage at the bottom had the clear capability to fail the system. However, post-collapse the adequacy of the anchorage could not be established, and there are no records of controlled inspection that could be analyzed with respect to anchorage strength.

Nevertheless, tests on 15 anchors installed elsewhere on the site show that the tendons had been tensioned—meaning the anchorage bonds were sufficiently strong. While this does not conclusively prove that the tendons in the area of the collapse were properly tensioned as well, the adequacy of the remaining tensioning, and the fact that the loss of one anchor and subsequent failure of a soldier pile would not necessarily engage additional anchors in the collapse, there is no basis to assume that anchor weakness led to the failure.

10 ANALYSIS AND DISCUSSION OF PROBABLE CAUSE OF FAILURE

10.1 Refutation of Design Defects and Vibrations as Possible Causes

The analyses carried out in Section 9 demonstrated that the as-constructed sheeting system -- if founded on competent rock -- would have had adequate capacity to sustain the load, although it was not installed as shown on the design drawing. Furthermore, the analyses also concluded that an increase in load from blasting vibrations might have brought the installed system beyond the code prescribed coefficients of safety, but would not have caused the collapse. Specifically improper installation or loss of capacity of the anchors was checked by calculations and tests and is not considered likely to have occurred.

10.2 Observations and calculations indicate problems with underlying rock

Section 9 shows that the weak part of the installation was the material underlying the soldier piles. The design required installation of the soldier piles on competent rock. Sheeting that was installed in competent rock, although exposed to the same similar construction operations and loads, did not fail.

Observations made during sheeting reconstruction to replace the failed wall revealed that the sheeting had been set on faulted and weathered rock, that is rock with properties significantly different than indicated.

10.3 Removal of Foundation Material

The sheeting system had been in place for some time before the collapse. Thus, events that occurred on the day of the collapse brought the system to failure: specifically, the removal of confinement of particularly weak foundation material. This explanation coincides with all observations and analyses. As long as the weathered rock was confined, the potential downward movement of the piles was not significant and unlikely to collapse the sheeting. The removal of the confinement in front of the weathered rock alone was sufficient to collapse the system. Had this removal been effectuated by mechanical means the result would not have been substantially different.

10.4 The Effect of Blasting

The investigation was not able to establish with a satisfactory level of confidence the details of blasting on 7/25/07 *vis á vis* collapse area with regard to the pattern of blasting and quantity of explosives. Also, the seismic monitoring required by FDNY had not been performed that day. This lack of information makes it difficult to prove with acceptable confidence that the blasting did or did not have an effect on the incident.

We could not find cosmetic cracks in nearby structures clearly attributable to the blast. What *is* certain is that blasting did not affect the adjoining building. We are almost certain that the blasting did not affect the building at 784 even in a cosmetic manner.

It is possible that the blasting had further weakened the soft rock. While the investigation did not find signs of over excavation due to blasting the gas pressure and shock produced by blasting in close proximity to piles 2 and 3 did have the potential to further weaken the already-soft mass of rock. Vibration from blasting performed six or seven feet away from the sheeting could also have weakened the weathered rock.

We can though state that had the rock been competent the blasting would not have had any effect. Also since there was a three hour gap between the last blast and the actual collapse one can say that at most blasting had a secondary effect.

10.5 Most Likely Scenario on July 25th, 2007

The debris examination showed that the piles were standing, albeit inclined and in a removed location. This correlates well with the displacement diagrams obtained in computer runs considering settlement and soft material at the base of the piles. Displacement of fractured rock under vertical (pile) load when lateral restraints are removed is well documented in technical literature.

The investigators agree that the most likely scenario of the incident was a collapse induced by the removal of the confinement that held together the weathered mass of rock supporting the sheeting. The portion of sheeting that collapsed was set on a faulted and weathered rock mass, instead of competent rock as indicated in the design drawings. This removal might have coincided with some blast initiated disturbance of the rock's precarious standing.

The soldier piles in that area had a vertical load of approximately 130 kips from the reactions of the inclined anchors. The weathered rock without lateral support from the confinement of the rock in front, which may be concomitantly weakened by some effects of blasting, could not hold the load and started to move towards the excavation. As a result the anchors were overstressed and lost their ability to support the sheeting system

10.6 Construction activities that allowed the failure

There were several deficiencies during construction. Some of these deficiencies, such as the as-constructed system not following the design drawings closely, did not cause the collapse incident. The major deficiency during construction was the placement of soldier piles in completely weathered rock. Based on review of the design documents and observations of the constructed piles, there were indications of the presence of the weathered rock and these are listed below:

- The drilling for the pile found soft rock and the socket for pile #3 was over-drilled because the piles were not driven and we can see the extension of the top of the pile with timber.
- The drilling for the blast holes encountered soft drilling in the same area.
- Test borings performed before construction had revealed a location of poor quality rock. This was interpreted as such and put on a geological section by Alperstein. (figure 24)

Although the presence of the weathered rock mass below the piles may be obvious, the parties responsible for monitoring the installation did not take any measures to improve or mitigate the condition.

III. Conclusions

Calculations and field observations show that the cause of collapse was probably not water pressure, inadequate design calculations, insufficient wall anchor capacity, or vibration-induced overload. Although the sheeting system included fewer anchors than required in the drawings, the sheeting system had sufficient carrying capacity.

The investigation found that the portion of sheeting system that collapsed was set over a faulted and weathered rock formation. Calculations show failure would have occurred for sheeting set upon weak rock. Additional calculations show that settlement of the material underlying the soldier pile forces the bottom anchor of the system to fail. This in turn, initiates a progressive collapse of the sheeting system in the area of the weak rock. This failure deformed mode obtained in the analyses correlates well with the actual debris layout observed.

On the date of the collapse, rock adjoining the failed sheeting was removed. The removed rock had been providing lateral restraint to the weathered rock mass underneath the piles. The removal seriously weakened the weathered rock mass, allowed rock displacement and induced the collapse. If the rock under the sheeting had been competent, as foreseen in the design, or the weak rock had been stabilized as soon as it was identified, the incident would not have occurred. The investigators agree that there were indications of the poor condition of the rock in that region prior to excavation but no remedial action was taken.

Because of lack of detailed records a solid opinion on the exact influence of blasting on the incident cannot be given. While the investigation did not find signs of over excavation due to blasting, the gas pressure and shock produced by blasting in close proximity to piles 2 and 3 did have the potential to further weaken the already-soft mass of rock. What *is* certain is that blasting did not affect the adjoining building, even in a cosmetic manner.

The investigators agree that the most likely scenario of the July 25th incident was a collapse induced by the removal of the confinement that held together the weathered mass of rock supporting the sheeting. The portion of sheeting that collapsed was set on weathered rock mass, instead of competent rock as indicated in the design drawings. This removal might have coincided with some blast initiated disturbance of the rock's precarious standing.

In light of the foregoing, it is the considered opinion of the investigators that the lack of proper observations and stabilization measures by the responsible supervising engineer throughout pile installation and further excavation allowed this failure to occur.

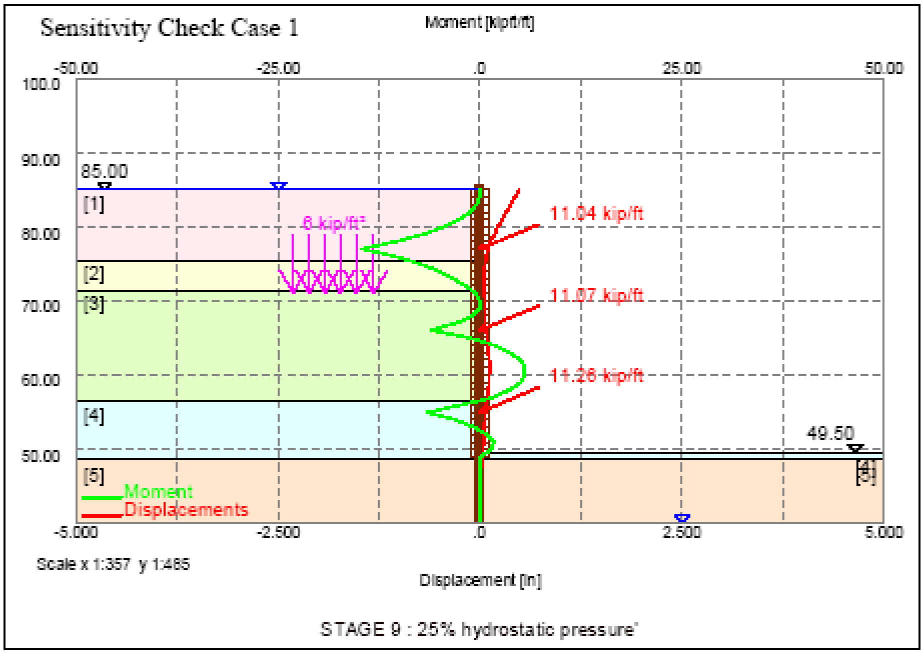

Figure 20 Sensitivity Check Case 1

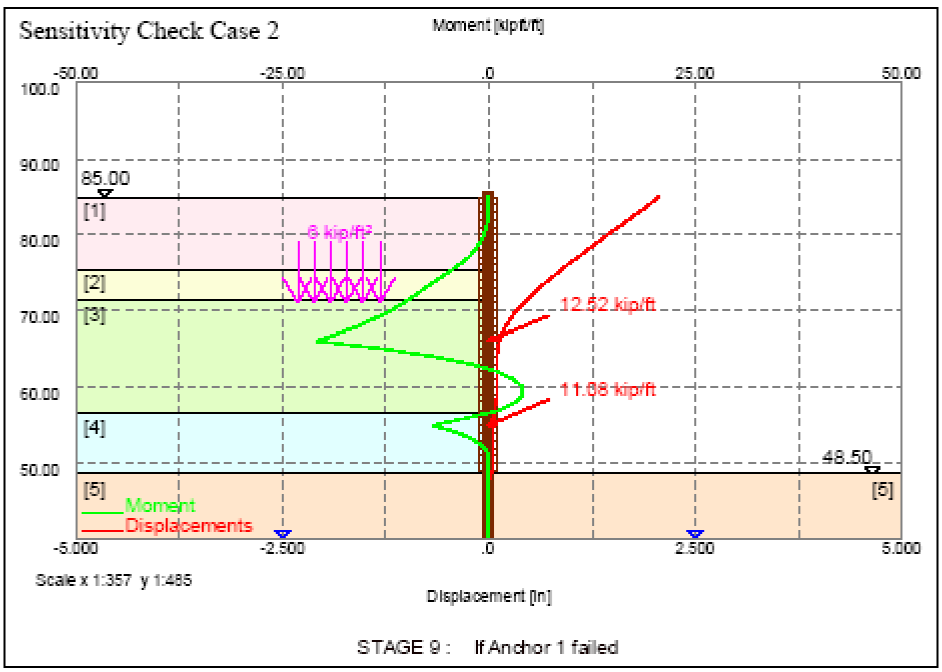

Figure 21 Sensitivity Check Case 2

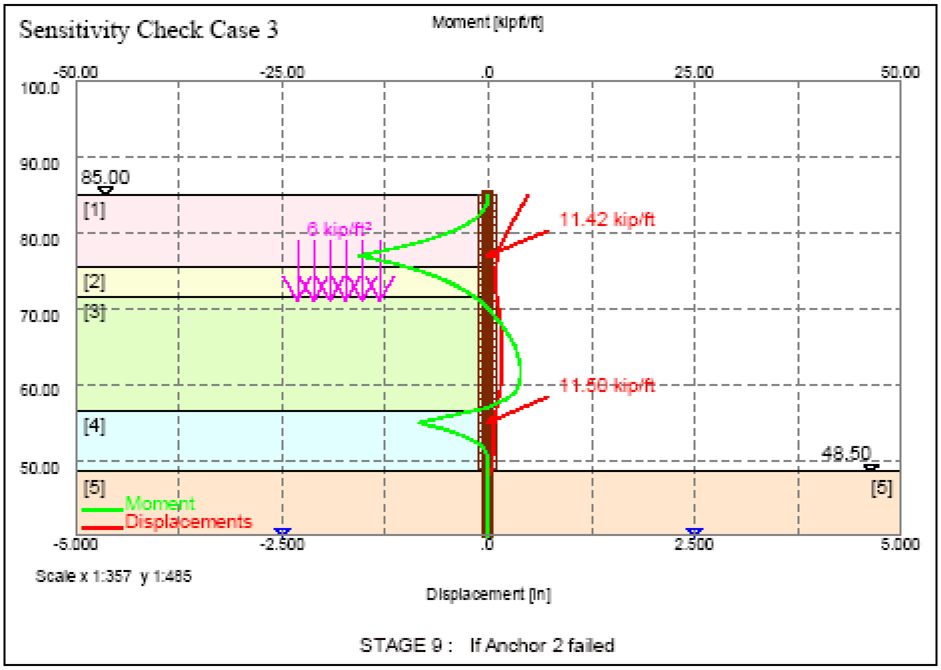

Figure 22 Sensitivity Check Case 3

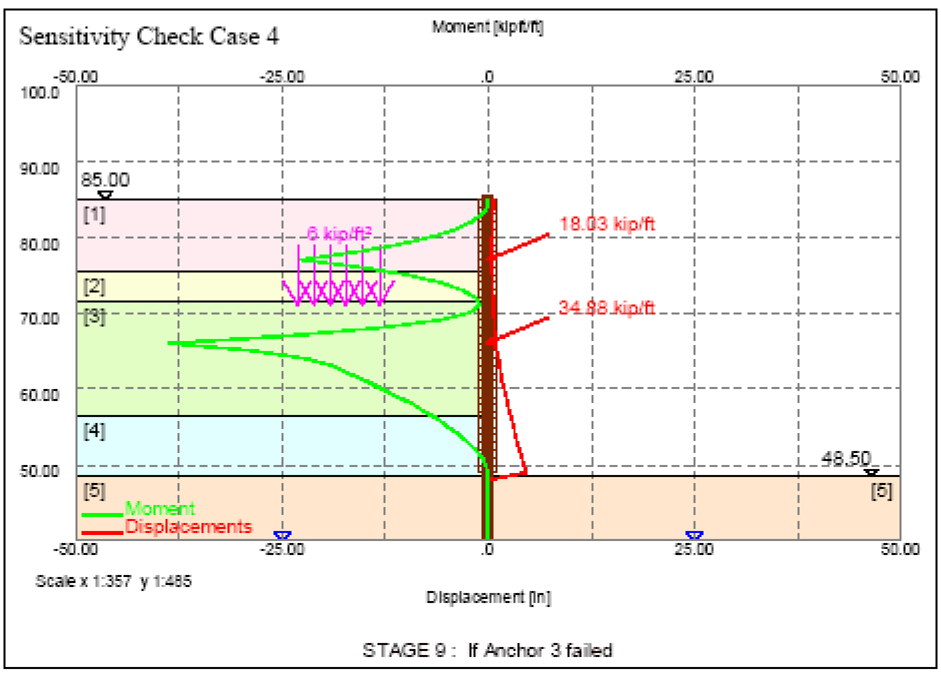

Figure 23 Sensitivity Check Case 4

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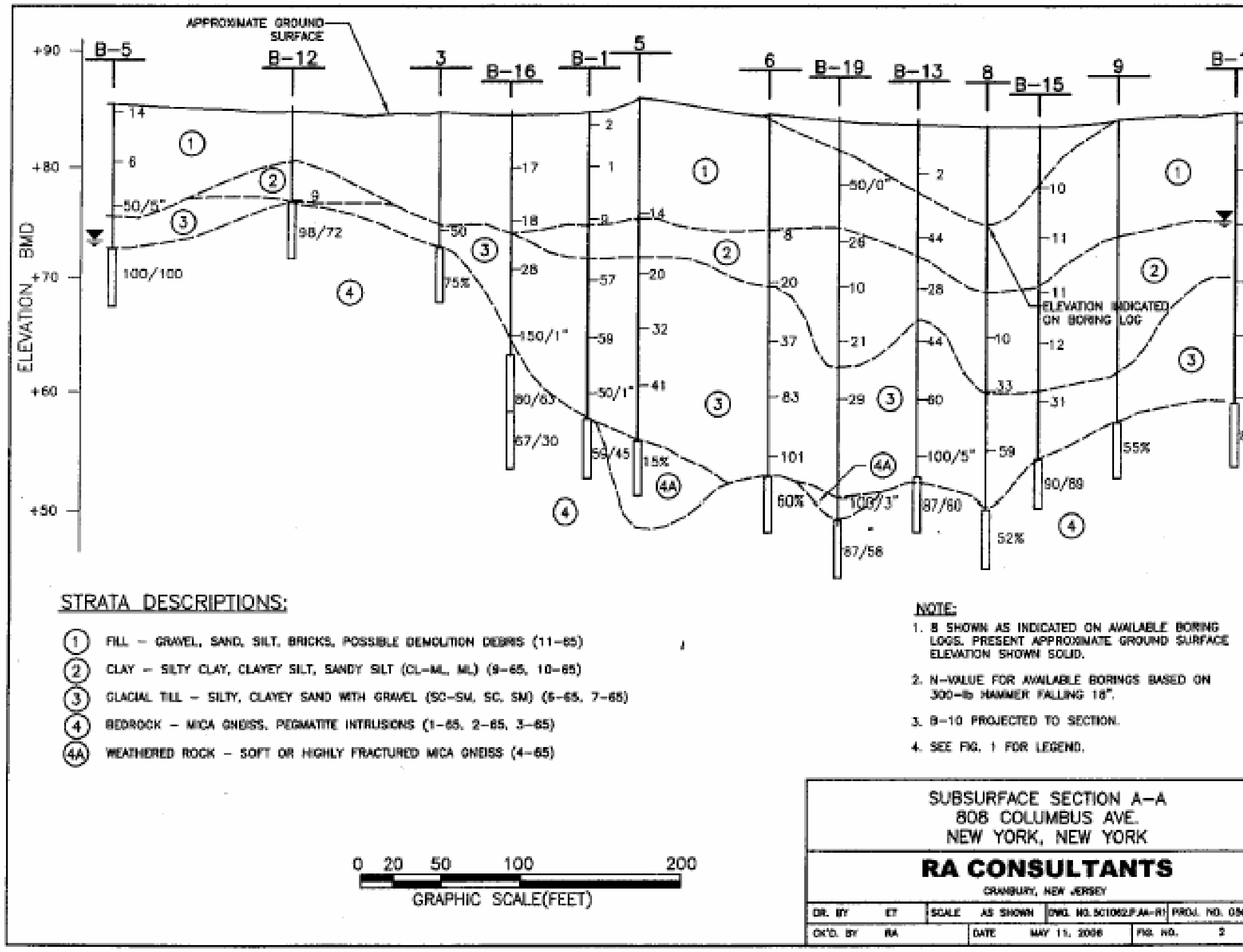

Figure 24 Section thru Site -RA Consultants (Alperstein)

11 APPENDICES

11.1 Sources Of Information

The following table presents the sources of information and the method of acquisition on which this memo is based.

Source of Information	**Method of Acquisition**
Geological records of in situ conditions	Observations, mapping and measurements by Dr Snee
Position and condition of structural elements in rock	Observations and measurements by Dr Snee and DoB
Blasting records	Provided by DoB/FDNY
Blast design	Interviews with Mayrich and NYFD
Instrumentation and monitoring	Provided by DoB, Shapiro and discussions with Instrumentation contractor representative
Means and methods of blasting	Provided by Mayrich and NYFD
Condition Structural Elements	Observed by ARUP and DOB Submitted drawing by Mayrich

Bibliography

a. Hartman Engineering, Design of Sheet Piling Structures Course Notes
b. US Army Corps of Engineers Manual EM 1110-2-3800 Systematic Drilling and Blasting for Surface Excavations.
c. Wiss, J.F. (1981*) Construction Vibrations: State of the Art*. Journal of the Construction Division, ASCE, vol 107
d. Dowding , C. H. *Construction Vibrations*, 2nd ed. 2000
e. Siskind, D.E Structure Response and Damage produced by Ground Vibrations from Surface Blasting. RI 8507, USBM (1980)

11.2 Blasting in poor quality rock

The rock mass properties are the most critical group of variables affecting the efficiency and effect of a blast. It is a matter of fact that rock properties are variable and so the selection of explosive, blast design and delay pattern must consider this variability. The following are the most significant ground properties that should be considered.

Rock Strength
Soft rocks can absorb blasting energy, make fragmentation more difficult and result in greater overbreak.

Density and porosity
Denser materials respond best to explosives and less dense and more porous rocks absorb energy that make controlled fragmentation more difficult. Porous and permeable zones can result in

unstable conditions because adjacent excavation walls can be lifted by gases migrating from the detonation. Conversely, permeable and porous zones can cushion and dissipate seismic energy.

Joint frequency
Closely jointed rock requires less energy to fragment and pre-splitting is particularly difficult in highly fractured rock. Overbreak is extremely difficult to control because the explosive gases migrate along the many fractures and loosen the mass or cause heave as with the porous or permeable condition.

Joint condition
Clay and other soft minerals on rock joints attenuate the seismic waves.

Faults
Blasting near faults will often break to the fault itself which is the same effect as a free-face on a blast so the preferred free face should be parallel to the fault. If the fault is perpendicular to the free-face there can be excessive gas migration beyond the blast and damage to adjacent walls.

Gasses can escape along permeable fault zones, causing loss in the blasting energy. Also, these features tend to have a higher porosity so they can cushion the effects of the blast. The blast that produces the desired effect in a strong massive rock may be unsuitable in weak faulted strata.

Weak zones
The advice in the US Army Corps of Engineers Manual EM 1110-2-3800 advises that special precautions are always required where weak zones are indicated in the excavation because "The resistance to sliding and slope failure along these surfaces may be divided into two components, an interlock strength and a residual strength. The residual strength may not be sufficient to preserve the slope so that during blasting, all precautions should be taken to avoid lowering the interlock strength by excessive vibration." The manual further advises that "Excavation walls containing weak zones may need to be redesigned so that the potentially unstable material may be removed" and "A carefully conceived blasting pattern will avoid development of unstable conditions … ." Further, "Undetected zones of weakness such as shears are serious problems in blasting because explosive energy always seeks the path of least resistance. If these zones of weakness can be identified and logged, steps can be taken to adjust the blast design accordingly. The driller is in the best position to identify weak zones by the increase in penetration rate during drilling the shot-holes."

Weathering
Some weathered rocks are so decomposed that they can be treated as soil and excavated without blasting. In these materials blasting has little to no benefit.

Line drilling
Line drilling consists of drilling a row of uncharged closely spaced holes along the final excavation limits to provide a plane of weakness to which the final row of blast-holes can break and also reflect a portion of the blast's shock wave. Line drilling involves small holes in the range of 2 to 3 inches diameter spaced two to four diameters apart. To further protect the final perimeter, the blast holes adjacent to the line drilling are spaced more closely and loaded more

lightly than the rest of the blast. These holes are sometimes referred to as relievers.

11.3 Order of Magnitude of Particle Velocity

The following is an attempt to evaluate the order of magnitude of the Peak Particle Velocity at a point in the underlying rock when blast vibrations are below the damage threshold for the building. The specific question is, could one have high PPVs at the rock underlying the sheeting even when the building does not display any crack after a round of blasting?

Rock Type	Dynamic Breaking Strain μin. in.	Propagation Velocity fps	Particle Velocity at Failure ips
Granite	360	18,500	80
Sandstone	550	5,000	33
Marlstone	310	13,000	48
Chalk	300	7,500	27
Salt	310	14,500	54

Figure 25 Strain and Particle Velocity at Failure for Five Rocks (from EM 1110-2-3800)

From the basic formula $U_{2=} U_1 * (R_1/ R_2)^n$ using n=2

R_1 –10 feet distance blast hole to collapsed sheeting
R_2- 25 feet distance blast hole to building
U_1 - particle velocity at rock failure
U_2 - particle velocity at building

$U_2 = 50(10/25)^2 = 8$
$U_2=20(10/25)^2=3.2$
$U_2=5(10/25)^2=0.8$ for very weak materials failure might have been possible without cracks appearing .

Result – An acceptable PPV of 0.8 could be sensed at the building when a blast 10 feet from the sheeting goes off and generates a 5 inch per second PPV at the rock under the sheeting.

R1 –5 feet distance blast hole to collapsed sheeting
R_2- 20 feet distance blast hole to building
U_1 - particle velocity at failure
U_2 - particle velocity at building

$U_2 = 50(5/20)^2 = 3.125$
$U_2=20(5/20)^2=1.25$
$U_2=5(5/20)^2=.312$

The vibrations sensed at the building would be even smaller when the blast is only 5 feet away from the sheeting. This result though does not take in consideration that the effect of blast in such proximity (5 feet) are not well studied and the general formula might not hold.

11.4 Geometrical Analysis

A typical problem in anchored sheeting is space inadvertently left between piling and rock. See Hartman Engineering, *Design of Sheet Piling Structures*. Such inadvertent gap between pile and rock allows a vertical movement of the pile to take place as the tendon is stressed and the pile attempts to resist the vertical forces. The unchecked capacity of the tendon to rotate around the point of anchorage makes the pile move in the horizontal direction as well. In the case of pile 2 where the last anchor was about six feet from the rock a one inch out of plane movement would have accompanied every two inches of vertical movement.

The size of the movement without failure of the anchor is determined by the distance between the toe of the soldier pile and the top of the bottom anchor. Using only geometrical analysis it can be shown that pile 2, when forced to move vertically one foot, for example due to rock breakage or sliding, would have displaced without failure of anchors. This vertical movement would have been accompanied by a horizontal movement of about five inches

11.5 Proposals for improving control of blasting operations when used for rock excavation.

Duties regarding control and handling of explosives

There should be a designated responsible party for the following activities:

- Control and handling explosives and explosive devices
- Trial blast
- Blast design
- Blast execution
- Administration of non-compliances

Design

A blast design should be submitted and it should specify the following as a minimum

- Designate responsible parties for each function (blasting, vibration monitoring, adjoining structure inspection)
- Explosive and explosive device type
- Diameter, depth, spacing and pattern of blast holes
- Proposed and maximum weight of explosive per hole.
- Maximum number of holes per round
- Type, make and configuration of delays
- Minimum distance of blasting to closest structure
- Expected level of vibrations at 50ft and 100 ft. from blast
- Generic type of rock to be blasted.
- Reference to reports and other documents used to develop the design – e.g. geotechnical reports
- Reference to permit and other stipulations
- The design should list the structures (buildings, infrastructure, construction in progress) in the area, describe structure type and state the expected impact. Any third party utility within the zone of influence must be identified if at risk from blasting – e.g. NYCTA

- The impact of the blast on the temporary works should be considered for the site and adjoining sites.

Control

- Health and safety plan including transportation and handling of explosives, security of the site, prevention of fly-rock
- Test blast should verify the design assumptions or redone
- The test blast should be documented
- A minimum of two seismographs should monitor blasting
- Current calibration certificates should be attached to active seismographs
- Maximum allowable vibration and noise levels should be observed
- Level of allowable vibrations that need to result in stopping of blasting operations for examination
- Procedure for resumption of blasting if maximum allowable vibration levels has resulted in a stop-work order
- Level and type of measurements that should be reported in real time to authorities should be stated

Reporting

Daily reports should include the following as a minimum:

- Seismograph readings
- Location of blasting in relationship to seismographs
- Number of holes
- Hole depth
- Hole diameter
- Hole spacing
- Hole pattern
- Total weight of explosives
- Type of blasting cap
- Delay series
- Maximum holes per delay
- Maximum charge weight per delay
- Stemming height

11.6 Debris of Piles

Figure 26 PILE 1

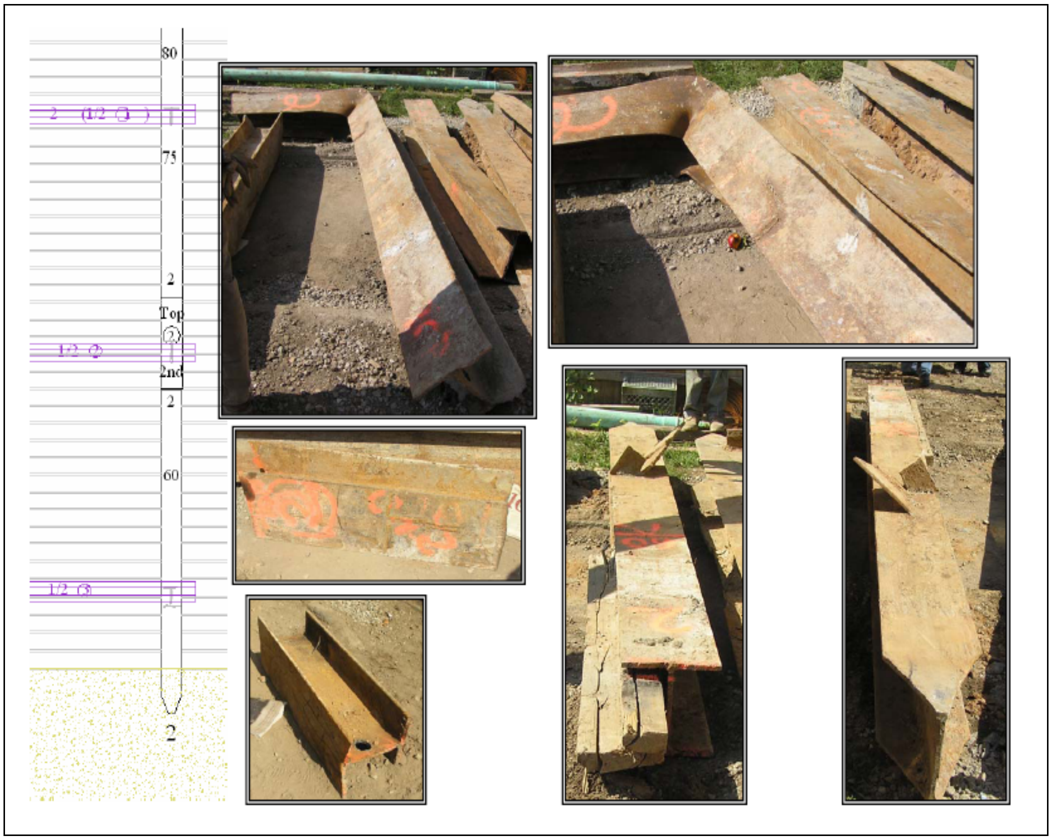

Figure 27 PILE 2

Figure 28 PILE 3

Figure 29 PILE 4