

Unexplained blasting vulnerabilities in a historic town

D.F. Laefer

School of Architecture, Landscape and Civil Engineering, University College Dublin, Dublin, Ireland

B. Loughlin

Moylan Consulting, Dublin, Ireland

S. Hickey

Mangerton Construction Ltd. Killarney, Ireland

S. O'Farrell & G. O'Mahony

School of Architecture, Landscape and Civil Engineering, University College Dublin, Dublin, Ireland

ABSTRACT: Historic residential and commercial structures in an early twentieth century town in the south-eastern United States that is adjacent to a quarry have regularly and repeatedly been damaged, despite the diligent keeping of blasting levels below peak particle acceleration levels previously thought to be safe. This paper explores the possibility of the damage mechanism being blasting-induced partial, repetitive liquefaction and the investigation methodology that led to this conclusion. The detonation of high explosives has been used in research to induce liquefaction at designated field sites and in laboratories, but the possibility of this occurring as an unintentional side-effect of commercial blasting has not been explored. This paper evaluates data from an actual community to assess the likelihood that the recorded damage is being caused by the quarrying in the southeastern United States. Although the results are not definitive they are strongly indicative that induced liquefaction is endangering this historic town.

1 INTRODUCTION

In 1904 the town of Fountain, North Carolina in the United States was founded. Approximately 50 years later a granite quarry opened adjacent to the town, thereby providing a major source of employment (fig. 1). Another 50 years brought the expansion of the quarry's blasting line closer to the community, a community that was increasingly less dependent upon quarry economics for its livelihood. As such, tolerance for quarry-induced noise, vibration, and potential damage decreased.

As the quarry expanded towards the town, residents increasingly complained of blast-related damage. Yet extensive blast records clearly showed peak particle velocities well below established "safe" limits. Consequently an alternative phenomenon was suspected. The following is a summary of the investigation method, the findings to date, and an assessment of the probability of blast-induced liquefaction being the cause.

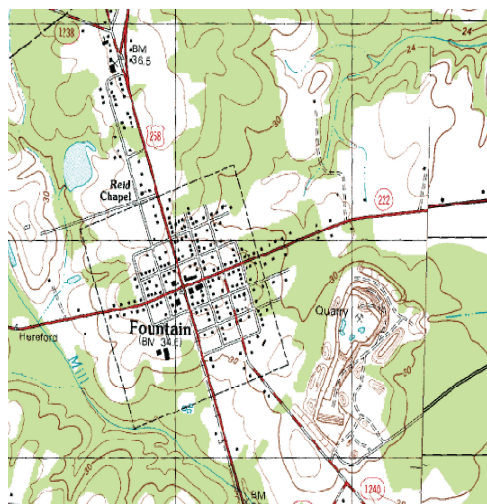


Figure 1. Aerial view of Fountain, NC, USA and quarry.

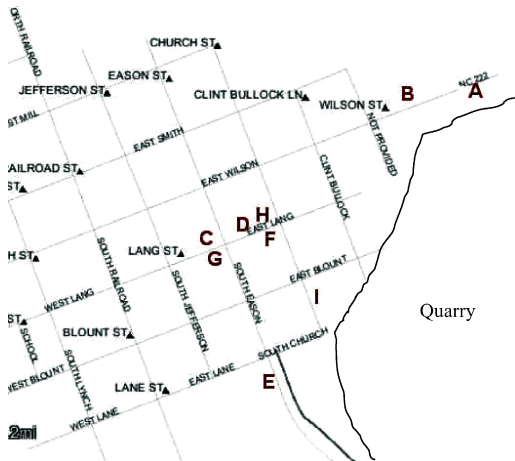


Figure 2. Map of Fountain, NC showing the location of the ten most damaged structures of the 26 inspected.

2 FIELD INVESTIGATION

The town of Fountain North Carolina has a population of approximately 520 people and a land area of approximately 1 square mile (2.59 square kilometers). Over the half century of the quarry's operation, granite extraction has evolved from removal of surface deposits, to blasting deeper into the rock mass and closer to the community. During the initial investigation, the two were as little as 290 m apart.

2.1 Study scope

In Spring 2004, residents commissioned an independent inspection of 26 properties, for interior and exterior architectural and structural damage. Additionally, a road damage survey was executed at that time, which was then updated in the Spring of 2006. The locations of structures experiencing the most severe damage are illustrated on the accompanying map (fig. 2).

2.2 Building damage found

All but two properties displayed damage. Those located closer to the quarry generally sustained greater damage than those further away. Long structures oriented perpendicular to the quarry exhibited greater damage on the end closer to the quarry.

Typical damage included [1] plaster wall and ceiling cracks with especially pronounced openings and large horizontal travel (fig. 3a); [2] separation and rotation of the architectural features (fig. 3b); and [3] cracking of the masonry – mostly architectural, although some structural (fig. 3c). Such damage amongst the ten properties shown in Figure 2 is summarized in Table 1.

Table 1. Snapshot overview of damage.

Structure	Cracked window	Cracked interior plaster	Cracked exterior brick	Foundation damage	Pier rotation
A	0	3	3	2	1
B	3	1	0	0	0
C	0	3	3	3	3
D	0	3	2	1	0
E	0	3	3	3	3
F	0	3	3	3	0
G	0	3	2	2	1
H	0	3	2	2	1
I	3	3	2	1	1

Damage level: 0 = None; 1 = Low; 2 = Moderate; 3 = High.

The damage appeared extremely widespread and repetitive, irrespective of recent decoration or renovation, as well as regular maintenance, and a generally high level of property upkeep. During oral inquiry, residents attested to the newness of much of the damage. In the case of one home, all internal damage recorded had occurred within only two years, since the house had been jacked to correct existing deformations and subsequently painted. In that structure, relatively minute damage was found in the masonry foundation and skirt, with respect to the rest of the structure, which may have been attributable to the extremely soft, sandy, lime-based mortar, which has a high capacity for deformation without cracking and can also undergo a certain amount of self-annealing when cracked. Building displacement had caused extensive cracking in the structure's interior plaster walls, especially evident in upstairs bedrooms; where one crack near the fireplace measured approximately 1.22 m long with a width of 1.25–1.5 mm. Extensively cracked wall plaster included large areas of horizontal travel (fig. 3a).

Structures located extremely close to the quarry were most damaged. Of the two closest (E and B on fig. 2, respectively), figure 4 shows extreme rotation of a foundation pier of a home that was generally in poor repair, but figure 5 shows an apparently well-constructed and maintained church extension built in the 1960s; adjacent to Figure 5, daylight was visible through the separation of two load-bearing walls.

2.3 Other damage found

The community was plagued by ground level depressions. In the roads, these manifested themselves as potholes. In the lawns, they resembled large rodent burrows (100–200 mm deep and 200–400 mm in diameter), generally large enough to trap a foot. Nearly all properties and streets near the quarry were pocked with



(a) Cracked plaster wall



(b) Rotated porch pier



(c) Cracked masonry

Figure 3. Typical damage throughout the community.

these indentions (fig. 6). A survey of the potholes was conducted by one of the residents in 2004 and repeated by that individual in 2006 (fig. 7). During the initial investigation in a mere five by five block area, over 109 potholes or potholes clusters were recorded. Note that much of the damage reported in 2004 was repaired by the town, prior to the 2006 survey (fig. 7).

2.4 Subsurface conditions

During the site inspection in mid-spring, the ground-water table was seen to be just below the ground surface (which immediately discounted dewatering as the likely cause for the buildings' movements). Hand pushed cores of the first 1–2 m showed a mixture of sands overlaying several distinctive layers of clay. The quarry commissioned two deeper ones. These were conducted with a Standard Penetration Test (SPT) and only went to the depth of less than 7 m, thereby failing



Figure 4. Rotation of pier foundation, where gap has been filled with a large amount of detritus.



Figure 5. Major cracking in concrete masonry unit wall.



Figure 6. Typical pot hole.

to establish the position of the rock layer or to retrieve any of that material. One large, loose-sandy layer was identified in the range of 2.6–4 m from the ground level.

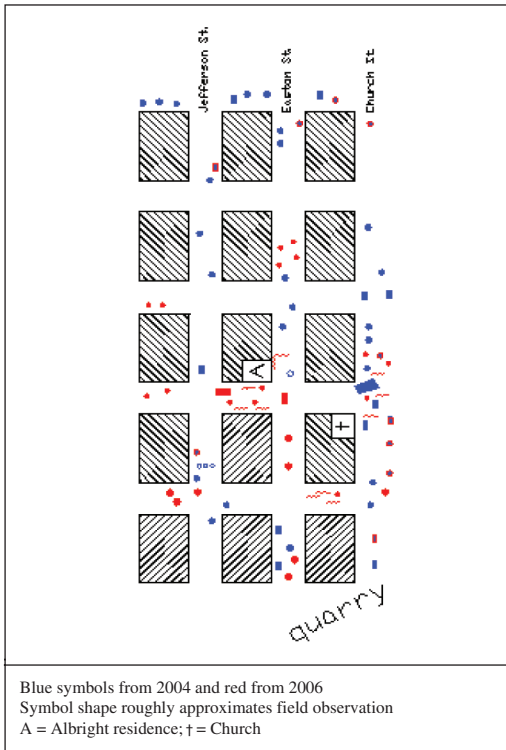


Figure 7. Pothole survey (2004 in red and 2006 in blue).

3 ANALYSIS

The analysis began with the most traditional approaches and later evolved to consider the less obvious.

3.1 Peak particle velocities as limit indicators

In 1971, the United States' Bureau of Mines concluded that the peak particle velocity was the best criterion with which to define damage-causing vibrations. They recommended a PPV of 50 mm/sec as the safe blasting limit. Chae (1978) reported that this was too conservative for some structures and not sufficiently safe enough for others, since response to ground vibration depends on structure type and form of construction, as well as both the stress history (age) and vibration-time history (peak value and duration) of the structure. Chae, therefore, recommended PPV values ranging from 13 to 100 mm/sec as limits.

A complete dynamic analysis of a structure by numerical analysis, phase diagram, Fourier series or response spectra using ground motion as input, would be ideal for determining structural response to ground vibration. However, such analysis is expensive, time-consuming, and requires an actual time history for

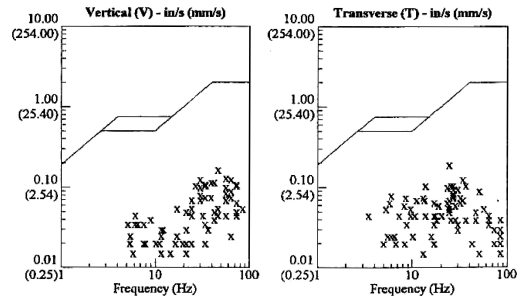


Figure 8. Blasting records showing peak particle velocities.

Table 2. Peak particle velocity limits for safe blasting (Amick 2000).

Category	Source	Category particle velocity mm/s (in/s)
Industrial	Wiss (1981)	100 (4)
Substantially constructed	Chae (1978)	100 (4)
Residential	Nichols, <i>et al.</i> , (1971) Wiss (1981)	50 (2)
Residential, new	Chae (1978)	50 (2)
Residential, poor condition	Chae (1978)	25 (1)
Residential, very poor condition	Chae (1978)	12.5 (0.5)
Visible previous damage	DIN 4150	4 (0.16)
Historic	Swiss Standard	3 (0.12)
Historic and ancient	DIN 4150	2 (0.08)
Occupied	U.S. Bureau of Mines	1.27 (0.5)

the subsurface conditions when subjected to blasting. While peak particle velocity is the best measure of a blast's damage potential, scaled distance is the best measure of a blast's vibration potential (Chae, 1978).

The State of North Carolina currently employs the U.S. Bureau of Mines limit on PPV of 12 mm/sec for plaster on lathe houses and 18 mm/sec for sheetrock construction, for frequencies less than 40 Hz. A vibration safety limit of 50 mm/sec is adopted for frequencies greater than 40 Hz. This standard is not, however, universally agreed upon, with many recommending even lower values – in the range of 3 or 4 mm/sec, especially for older or historic structures or those known to be in poor repair (Table 2).

A closer examination, however, of figure 8 clearly shows blasting levels not only in compliance with



Figure 9. Overturned building from liquefaction generated collapse, Nigita, Alaska, 1964 (www.ce.washington.edu).

North Carolina “good practice” limits but below the recommended European limits (Table 2). This created a quandary as there was clearly a pattern of damage (in terms of structures located closer to the quarry being more damaged than those further away) that indicated a mine-related cause but the direct vibrational shock and dewatering seem to be excluded.

3.2 Liquefaction

As direct vibration was excluded, another mechanism was sought that would explain why road and structure damage seemed to be correlated with proximity to the quarry. After much consideration, ground and road depressions were likened by the primary author to sand boils or sand blows, which are typically considered earthquake phenomena associated with liquefaction.

Liquefaction is a reduction in strength of a saturated soil, due to increased pore water pressure, volumetric compression and densification, caused by vibration or rapid loading. During this action, loose sands have a tendency to densify. Depending on the rate of loading, there may be insufficient time to allow pore water to escape, resulting in the pore water pressure (PWP) increasing and pushing the soil particles apart, thereby reducing the soil’s strength. Under extreme conditions the PWP can increase sufficiently to liquefy the soil (Seed and Idriss, 1971), possible results of which are shown in figure 9.

In addition to destabilizing structures, liquefaction can cause sand boils (sand blows), which occur, when pore water pressure steadily increases in a sand layer underneath a hardened surface (figs 10 and 11). During liquefaction, the pore water pressure increases, eventually reaching a magnitude sufficient to break through the solid surface and appear much like a bubble in boiling water.

Liquefaction is generally considered an earthquake phenomenon, occurring in sands, which can result in significant damage of buildings and infrastructure. However, selective laboratory and field studies over the past 30 years have proven liquefaction can be induced in sands, silts, and clays by repetitive, near

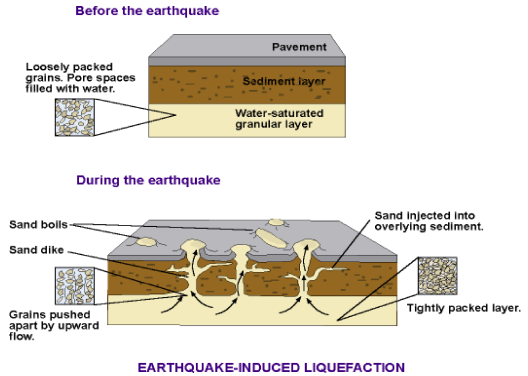


Figure 10. Sand boil description (<http://wapi.isu.edu>).



Figure 11. Documented sand boil in Mentasta Lodge, Alaska (courtesy of Shannon and Wilson).

field blasting. Recent examples of where explosives were intentionally used to induce liquefaction include Boulanger and Idriss 2004, Al-Qasimi et al. 2005, and Ashford et al. 2004.

3.3 Blast-induced liquefaction

Blasting can induce ground vibrations, which can compact and densify soil particles, thus, reducing their volume. As previously discussed, increasing pore water pressure, typically resulting from volumetric changes in a soil, is the catalyst to liquefaction and subsequently loss of strength in a soil mass. In assessing the potential for liquefaction to be induced by blasting, pore water pressure and peak particle velocities are key factors. If the peak particle velocities are not high enough to increase the pore water pressures, then the soil cannot liquefy. Charlie and associates (1985) stated that strains less than 0.01% were elastic and therefore not associated with residual pore water pressure increase. In assuming wave propagation velocity through water of 1493.5 m/sec, it was concluded that a peak particle velocity of 149 mm/sec may

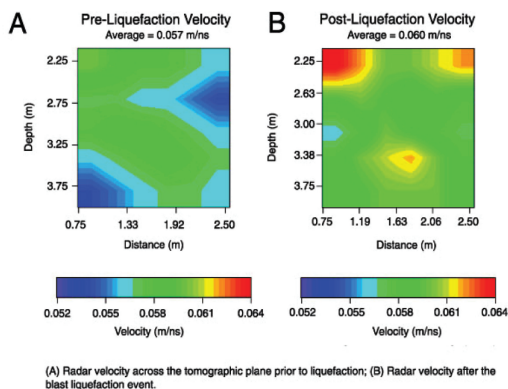


Figure 12. Density results as a function of velocity of Treasure Island Experiment (Rollins et al. 2005).

be significant enough to cause inelastic deformations and, thus, pore water pressure increase (Hryciw, 1986).

Experiments at the National Geotechnical Experimentation Site at Treasure Island, California 1995 highlighted interesting facts on blast induced liquefaction and the effects the phenomenon has on the soil mass involved (Rollins et al. 2005). Ground Penetrating Radar (GPR) surveys were used to examine the behaviour of hydraulically emplaced soils before and after blast induced liquefaction (fig. 12).

PVC cased boreholes, 9 m deep were drilled adjacent to a blast zone consisting of 8 0.5 kg explosive charges placed in a circular pattern around the pile group, approximately 3.5 m below the surface. Two antennas were incrementally lowered down the closely spaced boreholes and, along paths of known distance, high frequency GPR signals were transmitted. The distances along each path was measured with picosecond precision. Tomographic analysis determined the velocity structure of the soil, and as GPR velocity is very sensitive to changes in volumetric water content the velocity tomograms could be used to calculate a change in the voids ratio post blasting and hence determine changes in the soil density (and if settlement had occurred).

Prior to liquefaction the average GPR velocity recorded was 0.570 m/ns, which translates to an average voids ratio of 0.738. It was noted that blasting clearly liquefied the soil within the tomographic plane, elevating pore pressures and resulting in sand boils and settlement. Average GPR velocity post liquefaction was 0.597 m/ns which translated to an average voids ratio of 0.664 indicating a 10% decrease. This change in density resulted in a change in volume, which resulted in settlement of 17 cm for the 4 m thick liquefied layer. This experiment clearly shows that sub-surface blasting within a saturated soil mass will result in liquefaction and consequential settlement.

3.4 Likelihood of liquefaction

Thus, what remained to assess was the vulnerability of the in situ material to liquefaction. Liquefaction has been shown to occur in sands, silts and clays. A significant amount of research has been undertaken to parameterize the properties, which determine whether a soil is susceptible to liquefaction. Liquefaction susceptibility of silts and clays has been based on criteria obtained from observations of liquefaction at various earthquake sites in China (Perlea et al. 1999). Boulanger and Idriss (2006) concluded that a distinction can be made between soils that exhibit sand-like behaviour (commonly referred to as liquefaction) and those that exhibit clay-like behaviour (cyclic softening) based on soil properties such as Plasticity Index (PI), Liquid Limit (LL) and Water Content (Wc).

As further summarized by Seed and Idriss (1971) liquefaction potential depends on a soil's characteristics, its initial state of stress and vibrational characteristics. Significant factors include soil type, relative density (voids ratio), initial confining pressure, and ground vibration intensity. Uniformly graded materials are more susceptible to liquefaction than well-graded materials, while fine and/or looser sands tend to liquefy more readily than others. Liquefaction potential is reduced by increased confining pressure. Ultimately, however, liquefaction cannot occur without sufficient ground vibration.

Using the proposed relationships by Drake and Little (1983) and Wu and Hao (2005), whether the blasting from the quarry caused the soils to liquefy remains inconclusive. The analysis used to assess the liquefaction potential of the soils at the chosen sites in Fountain, N.C. was based on equations derived from controlled experiments carried out in homogenous soils on a much smaller scale than the analysis for the sites in Fountain. As such, the equations used in the analysis depended on variables that were difficult to determine or for which insufficient data was available and, thus, the analysis is heavily dependant on assumptions. Additionally, the ambiguity was exacerbated by a high level of uncertainty regarding the soil characteristics, primarily the absence of a gradation distribution for each layer and the lack of Cone Penetration Test (CPT) data, the latter of which has the highest correlation of known vulnerabilities, in terms of readily available soil property.

Furthermore, small modifications in certain parameters, such as the attenuation coefficient, results in a substantial difference in the predicted peak particle velocity transmitted. Furthermore, given the complexity and scale of the site compared to those from which the equations were derived, the equations may require minor modification to be able to be applied directly. In the end, the analysis generated inconclusive results, requiring better site investigation.

However, irrespective of whether the soil meets the criteria of liquefiable, it is well-known that substantial damage to a structure can occur from settlement and subsidence from repetitive energy input, as illustrated in the watershed paper by Lacy and Gould (1985), where pile driving was found to cause sufficient ground densification to cause settlement-induced damage far below the peak particle velocity levels previously employed as a safe limit.

4 CONCLUSIONS

As the analysis relies on assumptions made based on limited information, further study is warranted into the soil conditions (e.g. attenuation coefficient), where a more detailed analysis would be a more reliable indicator of the potential of the blasting to induce liquefaction. The output from a cone penetration test would also be very useful in determining the liquefaction potential of the soils. The facts do however remain that there is an underlying geotechnical phenomenon that is causing great damage to a historic community and that the damage is largely proportional to the distance from the quarry's blast line. Since vibration measurements in the form of peak particle velocity have shown the blasting levels to be below what was previously thought to be safe, additional investigation is needed to acquire more precise soil characterization data and/or more sophisticated methods are required to analyse a wider scope of the vibrational characteristics generated by the blasting to better understand energy transmission means.

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