Foundation Blast Loading Accident History, Past Use, and Future Considerations

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Abstract

Protective construction criteria development has historically focused on components above the foundation due to their potential to rapidly fail and contribute to structural collapse and/or hazardous debris generation. For example, UFC 3-340-02 Structures to Resist the Effects of Accidental Explosions (reference (3)) does not include information on allowable soil bearing for load combinations that include blast load or conditions in which analysis is required for a footing or mat foundation for blast loading. However, the design and analysis of recent intentional detonation site (IDS) facilities has made it clear that expanded understanding of blast loading on foundations could be beneficial for siting as well as design and analysis of foundation systems. This study presents accident data compiled and analyzed for exposures that were noted to have led to foundation damage of surrounding structures. This information was used as an initial data point for a lower and upper bound of scaled standoff that could generate a risk to personnel from compromised foundations. It was considered that this approach alone may not contain a realistic upper bound value due to the understanding that a hardened facility would transfer more loading to the foundation prior to failure of the structure than many unhardened facilities built in the 1940s to 1970s, which span the timeframe of most of the collected accident data. For example, some of these accidents were unhardened buildings between K18 and K50, and some with no noted foundation damage may have resulted in foundation failure if the superstructure was hardened to mitigate hazards resulting from a quantity distance (QD) arc violation from an explosives operation location (EOL) or IDS. This initial data point was then compared to predicted crater sizes from UFC 3-340-01, Design and Analysis of Hardened Structures to Conventional Weapons Effects (reference (4)), and the blast effects computer (BEC) for a lower bound of consideration. This data was then used to conservatively check when the blast response at a given scaled standoff may exceed foundation design loading and when the allowable bearing pressure would be exceeded, even with added dynamic increase factors for shorter duration loading. Data is also presented on methods previously utilized for live-fire training facilities, many of which are experiencing extreme loading events monthly and are not experiencing differential settlement or other means of foundation failure.

Introduction

While protective construction has traditionally prioritized above-ground structures to prevent collapse and debris, new findings from intentional detonation sites highlight the need to better understand and design for blast effects on building foundations. Accident data repositories (references 5 and 6) were combed for relevant data to determine if previous accident reports supported the need or lack thereof for consideration of blast loading transference to foundation systems. This data was then used as a starting point for determining when blast loading on foundations could lead to structural failure and when foundations should be considered adequate without analysis. Additionally, methods of analysis and design of foundations for intentional detonations are included that have been used to successfully harden or analyze existing training and IDS facilities.

Accident Data Analysis

Available accident data and requested installation data was utilized to develop a matrix of HD 1.1 accidents that includes: NEW, scaled standoff, damage level, and notes on construction materials when the data was available. A large amount of this data was centered on the time span between 1942-1946 due to a scaleup of explosives production and handling during these years. However, the data set covers a century of HD 1.1 accidents. The complete data set, with 314 accidents, is shown in Appendix A. Of the 314 accidents, only 12 note foundation damage, outside of the crater of the detonation. The average scaled standoff distance of the donor was 13.6 $\frac{ft}{NEW^{(\frac{1}{3})}}$. Of these 12, only two had noted damage past a K30 scaled standoff, one of which is assumed based on a noted 2 ft. out-of-plumb recording of the top of the structure. The other was uneven foundation settlement of a home at K33.1. See Figures 1 and 2 below for reference.

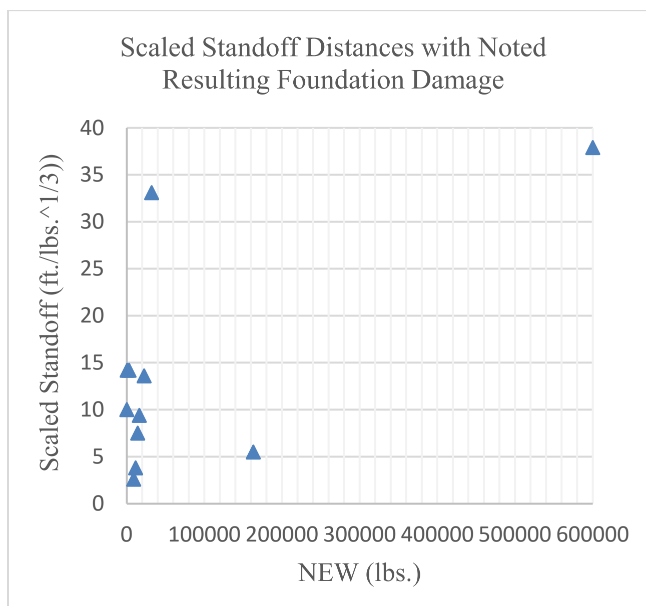

Figure 1. Scaled Standoff Distances with Noted Resulting Foundation Damage

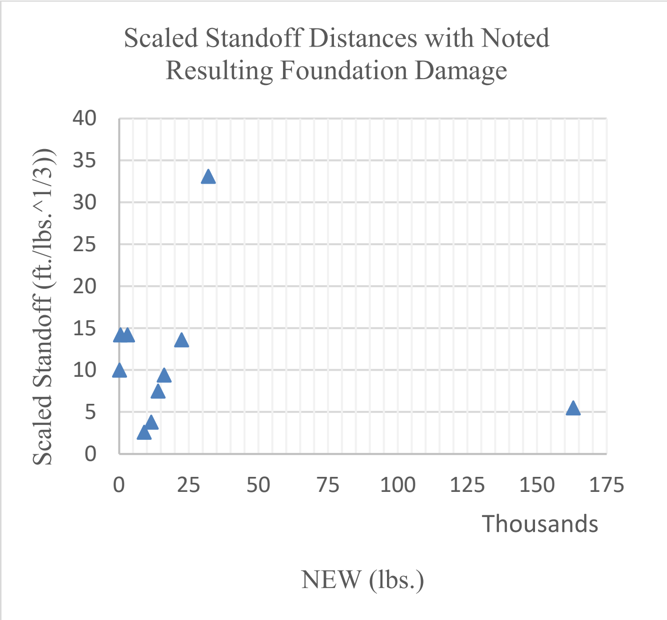

Figure 2. Scaled Standoff Distances with Noted Resulting Foundation Damage with Only Data Points of NEWs Below 500,000 lbs.

In contrast, it was also determined when foundation damage was specifically observed to not be present. This was done to avoid assuming that if a structure's walls were recorded to have failed, no foundation failure also occurred. 11 occurrences of this were found, with an average scaled standoff of 19.55 $\frac{ft}{NEW^{(\frac{1}{3})}}$. This data suggests that foundations are likely to be adequate for load transference from superstructure blast response beyond K24; however, this would not account for the primary superstructure construction of the donor facilities in the data set being wooden and brick facilities. In a situation where a facility is hardened with protective construction for an inhabited building distance (IBD) arc violation, the exposed site (ES) would likely be a reinforced concrete or reinforced concrete masonry facility and transfer more loading to the foundation. For this reason, additional analysis is suggested for facilities between maximum crater radius extents and K40 to determine if facilities within this range could fail from external loading, purely due to foundation response, even if the structure is hardened. Crater diameters from the accident data were compared to the predicted crater diameters calculated using the Buried Explosives Module (BEM) and UFC 3-340-01 methodology to obtain a lower-bound for analysis of the foundations. Please note that a continuously occupied facility should not be placed within the crater radius of a potential explosion site (PES), even with protective construction, due to potential for foundation undercutting to occur.

Cratering Analysis Tools vs. Accident Data

The accident data collected for a comparison of scaled standoff and noted foundation damage was then used to determine expected crater depths and diameters and compare those to the results of UFC 3-340-01 and the BEM cratering predictions. It should be noted that all accident data was assumed to have a soil depth of 0 ft., which can lead to skewed data of the Buried Explosives Module (BEM) and other analysis tools for cratering since the tool is primarily used for buried munitions. See Figures 3 and 4 below for comparative data. Figure 4 is representing the same data as Figure 3 with the only difference being that the scale along the x-axis has been altered for more clarity for smaller NEWs.

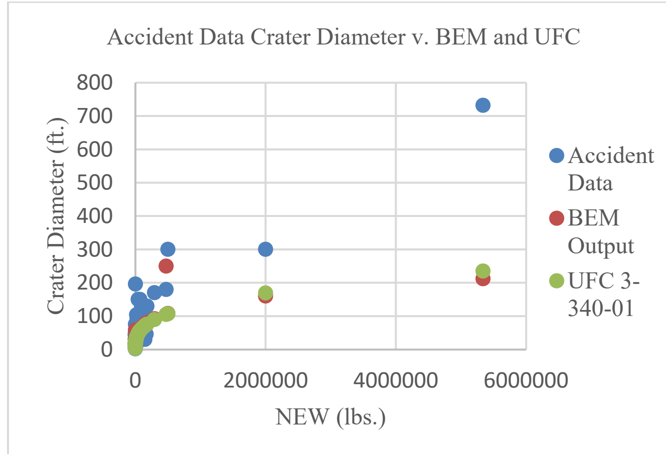

Figure 3. Accident Data Crater Diameter Compared to Buried Explosive Module and UFC 3-340-01 Method Output

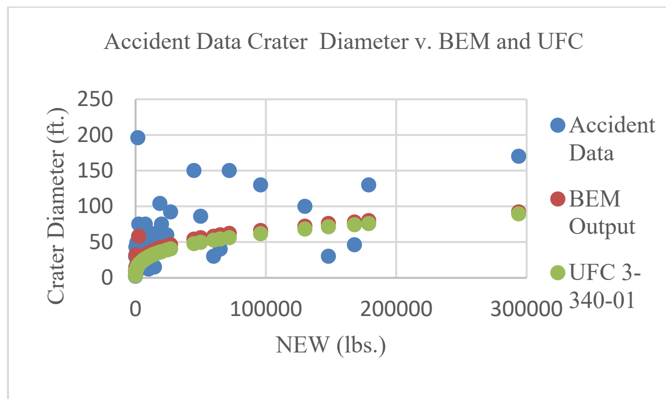

Figure 4. Accident Data Crater Diameter Compared to Buried Explosive Module and UFC 3-340-01 Method Output with Only Data Points of NEWs Below 300,000 lbs.

It was observed from the data in Figures 3 and 4 that the predicted crater diameters, using the methods of the BEM and UFC 3-340-01, track below the accident data crater diameters on average by 7% and 24%, respectively. Some values as far below as 89% and 92% below the recorded value. It is assumed that some of the extreme error between values may have been caused by soil liquefaction, which is noted in literature to have the potential to expand crater diameters but is not specifically noted as occurring in the accident data reports. The highest error value was shown to be a statistical outlier and removed from the data set to avoid skewing results; however, the next highest errors for underprediction were 71% and 79%. BEM is shown in Figure 4 to provide more conservative results than UFC 3-340-01 and still tracks below accident data. Therefore, of the two, BEM would be more reasonable method for prediction and is shown in Appendix A results for clarity and self-consistency.

It was assumed that for this reason, it would be beneficial to evaluate the accident data for cratering as a potential minimum scaled standoff for location of adjacent explosives operations locations (EOLs). The average observed radius of noted cratering was at a scaled standoff of 2.47 $\frac{ft}{NEW^{(\frac{1}{3})}}$. The maximum extent observed was 6.46 $\frac{ft}{NEW^{(\frac{1}{3})}}$, excluding the statistical outlier value. Only two of 57 accidents, exceeded K5. When explosives safety separation distance (ESSD) arcs cannot be met, protective construction is often used to mitigate the hazard by providing an acceptable level of protection to personnel and explosives in the acceptor facility. For this reason, situations often arise where facilities are hardened against a given dynamic loading to provide an equivalent level of protection to occupants and explosives in the adjacent facility. Understanding the maximum potential extents of cratering can aid in understanding the threats to building foundations at various scaled standoff distances.

Based on the comparative results above, it was assumed that a reasonable range of analysis and design for blast loading on foundations may be K5-K18 but could be higher due to much of the accident data not containing hardened buildings. This would not be the case in most future occurrences due to Defense and Safety Regulation (DESR) 6055.09 (Section V2.E5.1.2.1 reference (1)) requirements to harden buildings within K18, from related potential explosion sites (PES) to provide an equivalent level of protection to a K18 scaled standoff distance and the gradual movement away from grandfathering of explosives safety site plans (ESSPs). This superstructure hardening could result in more load transferring to the foundation than what would have been seen in the historical data set. For this reason, an expanded range of K5-K40 was used to determine if basic analysis methods could be used to rule out a need for the region from K18-K40 to be considered for blast loading on foundations, using residual capacity, increased dynamic soil bearing capacity, and dynamic increase factors, etc.

Foundation Blast Loading Analysis and Design

Several initial underlying assumptions were utilized to investigate an appropriate range of scaled standoffs for which blast loading on foundations may need to be considered, based on both accident data and initial analysis methods.

- Facilities are between K5-K40
- Facilities of concrete construction.
- Shallow foundation systems, with continuous footings.
- Footings bearing on sand foundation.
- Soil vertical and rotational springs used for wall support conditions based on footing dimensions.
- Internation Building Code (IBC) minimum allowable bearing pressure for unknown soil conditions assumed (1,500 psf)
- A two times multiplier applied to allowable bearing pressure for blast loading combinations (3,000 psf), due to short duration. Similar to 33% increase allowed for seismic and wind, but higher due to milliseconds duration verse seconds duration.

While facilities within K5-K40 of existing and EOLs could be concrete masonry or reinforced concrete construction, it was assumed for the purpose of this study that they are all reinforced concrete to simplify analyses and focus in on the main area of concern, which is increased load transference to the foundation from increased component robustness in comparison to historical data.

A mock ES, concrete EOL facility was assumed to have an internal height of 15 ft., width of 25 ft., and length of 50 ft. These dimensions were maintained for all scaled standoffs evaluated. ACI 318-14 was used as a basis of pre-existing design to determine residual capacity of concrete foundations. A conservatively low region of wind and seismic was assumed as a basis to avoid overestimating residual capacity for blast loading based on higher wind and seismic. The underlying assumption is that personnel would not be present and explosive operations occurring during a tornado or other 100 mph wind event, as operations are shutdown in extreme weather events. Therefore, controlling load combination is assumed to be dead load plus blast load with a factor of 1 on both as an extreme loading event. Soil bearing capacity was assumed to be two times the IBC minimum for unknown soil conditions, based on soil bearing capacity increasing for rapid dynamic loading. Foundations were assumed to be continuous shallow spread footings, with internal slab-on-grades. Only the continuous footings were evaluated for blast response due to load path. It was found that the Load and Resistance Factor Design (LRFD) combination of 1.4 times dead load controlled for the foundation design as opposed to load combinations containing wind and seismic, prior to analyzing the structure for blast loading. This is reasonable a blast hardened building as this often increases mass. A service loading demand of 4.72 kips axial and 2.28 kip-ft moment were found for foundation reaction and then used for a preliminary footing size, using equation 1 below. Where σ is the bearing pressure below the footing, P is the load applied, A is area, M is moment, and S is the section modulus of the footing.

$$\sigma = \frac{P}{A} \pm \frac{M}{S} \tag{1}$$

The additional load applied from the footing itself was added to P in Equation 1. Based on this information, a baseline footing of a 12-inch thickness and minimum footing width of 5’-6” was assumed to get bearing pressure below 1500 psf for conventional loading combinations. Minimum flexural reinforcement was found to be adequate for the baseline footing of #5@12” C/C on each face and in each direction. This data was then used to check if the capacity of the soil and/or footing would be exceeded under a K39 accidental detonation for a PES with 2,000 lbs. It is understood altering NEW would change the impulse loading and duration, which impacts the strain rate, response, and other factors. However, this is only a preliminary check for cause for concern.

Residual capacity of the footing is found to come from three sources. First, the soil bearing capacity being able to take higher loading for shorter duration loading events. Second, the minimum reinforcement for temperature and shrinkage and flexural reinforcement often exceeding the flexural demand. Lastly, the load combinations for loads other than blast loading having more built in conservatism for dead load than the dead loads plus blast load combination.

RAM Elements Software was used to apply blast loading as a static loading on all faces of the structure to allow for global movement to be recognized as increasing foundation uplift or bearing pressure around the perimeter footing. This loading was initially applied using a dynamic load factor (DLF) of 1.95 for the peak triangular loading DLF) from UFC 3-340-02. This loading was then refined based on the period of the components being analyzed to reduce over conservatism. The periods calculated using this method were compared to UFC 3-340-02 SDOF periods as a sanity check, with the structure broken up to match what is shown in Figure 5 below. The assumption for the SDOFs, were four-side-fixed for the roof and three-side-fixed for the front wall. This comparison is shown in Table 1 below.

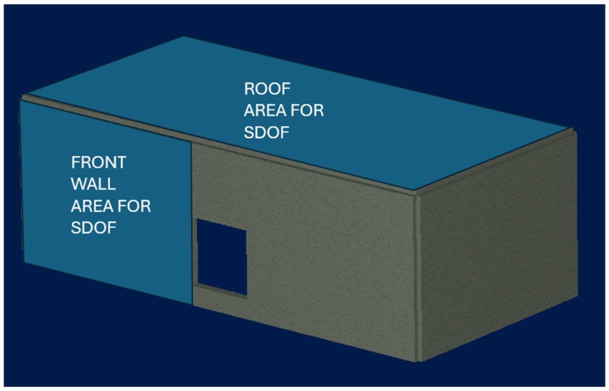

Figure 5. Front Wall and Roof Area for Blast Response Comparison

Table 1. Comparison of SDOF and DLF Method Component Periods

Component Period Comparison			
Method	**Component**	**Period T (ms)**	**Δ (%)**
DLF	Front Wall	44.37	-
SDOF	Front Wall	35.56	25%
DLF	Roof	82.93	-
SDOF	Roof	68.68	21%

The DLF method is best suited for elastic response, but is known to be conservative for inelastic response, due to the neglecting of the reduction in stiffness post yield. Neglecting this aspect for load transfer to the foundation is conservative. Periods for the DLF method and SDOF method were found to be within 25% of each other for the front wall and roof components. This is a reasonable agreement of data, when considering that the DLF method considers both global and relative movement, as opposed to the SDOF, which only considers relative component deformation.

The rear wall and side walls did not respond to the loading in a comparable manner to an inbound loading SDOF analysis. The side walls acted more as shear walls in response, with sway occurring, and the rear wall deflected in a more comparable manner to a cantilever wall due to the front wall and roof movement rotating the rear wall away from the applied blast loading. This rotation resulted in more downward force from blast loading on the rear wall, than the front wall. Peak reactions from load combinations that include blast loading were found to be greater than those that do not for K39. However, the residual capacity of the footing was shown to be adequate due to the following considerations:

- Minimum flexural reinforcement exceeded demand.
- Minimum footing depth added shear capacity to the section, which was not fully used in conventional demand.
- Blast loading bearing pressure being allowed to reach double the allowable bearing pressure due to rapid dynamic loading and unloading.
- Load combinations for loads other than blast load use a higher assumption for dead load (1.4DL etc.).

This process was then repeated for K38, which showed that the peak reactions continue to exceed non-blast loading load combinations, but the foundation has enough residual capacity to meet demand. This process was iteratively followed with decreasing scaled standoff, until analysis showed potential failure with a 2,000 lb. NEW. A hemi-spherical surface burst was assumed in all cases, as this is most common for accidental detonations.

Shallow spread footing analysis showed no potential failure occurring at K23 and larger scaled standoffs for 2,000 lbs. using these methods. These results track closely to the observed scaled standoffs of noted foundation damage from accident data, shown in Figure 1, which only had two data points with noted foundation damage past K23. One of these only noted out of plumb building at top of two feet. This could have potentially been caused by superstructure damage only. The other was a home foundation with differential settlement. The eleven data points with specifically noted foundations that did not have observed damage were averaged at K19.55, which also agrees with this data well. At K23, 99% of two times allowable bearing pressure was reached.

A literature review was next performed to determine if any additional data supported or refuted the need for consideration of blast loading on foundations within K23.

Literature Review

Existing Department of Defense criteria on blast design and analysis of reinforced concrete structures does not cover foundation design and analysis as the focus has been given to response of the superstructure. Existing studies in academia have been largely focused on deep pile foundation systems and ground shock transference through the soil to buried structures. For this reason, no data external to this paper was found to support or negate the need for consideration of blast loading on shallow foundations for hardened facilities.

Summary/Conclusions

Based on the findings of this study, accident data and conservative analysis methods suggest foundation blast loading may need to be considered within K23 for blast hardened buildings. Additionally, intentional detonation operations should have foundations evaluated for blast response to ensure the structure remains elastic. Further analyses and research should go into investigating the potential range of scaled standoff distances for concern and evaluation.

References

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2. DA PAM 385-64, "Ammunition and Explosives Safety Standards," RAR, 10 October 2013
3. UFC 3-340-02. "Structures to Resist the Effects of Accidental Explosions," Unified Facilities Criteria, Change 2, 01 September 2014.
4. UFC 3-340-01. "Design and Analysis of Hardened Structures to Conventional Weapons Effects," Unified Facilities Criteria. 1 June 2002.
5. HNDTR-75-23-ED-SR, "Overpressure Effects on Structures," 01 April 1979.
6. Dr. Isley's Explosion Log

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Acknowledgments

Thank you to Brad Durant of the Expeditionary Warfare Center (EXWC) for starting many of the conversations around this topic and coordinating efforts. Also, thank you to Daniel Richey and Sarah Puchner for data collection and input used.

Definitions/Abbreviations

EOL	Explosives Operation Location
QD	Quantity Distance
BEC	Blast Effects Computer
IDS	Intentional Detonation Site
ES	Exposed Site
PES	Potential Explosion Site
LRFD	Load and Resistance Factor Design
C/C	Center-to-center
NEW	Net Explosives Weight
DLF	Dynamic Load Factor

Appendix

Please see attached 11x17 Appendix Table for reference.