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EXPERIMENTAL STUDY ON BLAST INDUCED ROOF LOADS IN BUILDING STRUCTURES

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ABSTRACT

Analysis of the response of different components of building structures to blast loading is an important step in the protective design of buildings. Included in such components are members of the roof and sides of building spanning perpendicular to the shockfront. Traditionally, in the analysis of the blast response of such elements, the blast loads are represented as equivalent uniform loading. The equivalent uniform loading converts the moving blast loading on the roof to a simplified spatially-uniform, time-varying load. However, earlier studies have shown that the results of response analysis based on the simplified loads are often quite inaccurate when compared to those obtained from a more precise analysis. Therefore, an experimental study was conducted to investigate the response of roof elements under blast loading and to compare the accuracy of two different methodologies based on the spatially-uniform and time-varying loading concept. The results show that the traditional equivalent uniform loads cannot accurately model the response of roof elements, while more detailed finite elements models yield more accurate and reliable results.

Keywords: - Blast wave, structural response, roof and side beams, experimental analysis, finite elements analysis.

1. INTRODUCTION

Accurate modelling and analysis of structures in extreme loading events such as blast and impact is very important in the practice of structural engineering today. Especially with widespread use of high performance computers and computational techniques, the need for using highly conservative simplified design models has been significantly reduced in the past decades. Today, more accurate simulation techniques and methodologies are being developed to replace the simplified conservative models. The growing cost of new construction and upgrade (or retrofits) of the aging structures and infrastructures urge engineers to use more advanced and accurate analysis methodologies for design.

One of the areas where the traditional simplified models are still being used in the analysis of structural elements is in blast resistant design. For members oriented perpendicular to the blast shockfront, in the roof or sides of the building structures, the traditional practice uses an equivalent uniform loading which is spatially-uniform but time-varying (DoA 1986, DOD 2008). This equivalent uniform loading ignores the propagation of the blast waves, and therefore is not compatible with the nature of the blast wave propagation phenomenon. In reality, as the blast wave traverses the roof element the magnitude of the incident pressure acting on the roof decreases while the duration of the blast wave increases.

The incident pressure caused by the blast is accompanied by the dynamic pressure, similar to drag pressures produced by wind gusts. These drag forces are negative pressures and counter the incident blast pressure (Glasstone and Dolan 1977, DoD 2008), therefore the overall pressure acting on the surface of the roof and side wall is the algebraic summation of the incident pressure and a dynamic pressure in accordance with Equation 1. At any specific time, only parts of the roof can be loaded, and the loaded length depends on the duration of the incident pressure, location of the shock front, the velocity of the blast wave, and the length of the roof element.

[1] $P(t) = P_{so}(t) + C_D.q_0(t)$

$P_{so}(t)$ is the side-on (incident) pressure, $q_0(t)$ is the dynamic pressure, and C_D is the negative drag coefficient.

The magnitude of the peak pressure of the simplified equivalent loading on roof elements is given by Equation 2. The parameters in the equation are time-varying and defined differently by different documents. C_E is the equivalent load coefficient, and index r refers to the reference point at which the blast pressures parameters are determined and used in the generation of peak equivalent uniform load.

[2] $P_e = C_E.P_{sor} + C_D.q_{0r}$

The UFC 3-340-02 document (DoD 2008) and ASCE (2010) use the front point of the roof (f in Figure 1) as the reference point, while TM 5-855 (DoA 1986) manual associated with the conventional weapons effects computer program (ConWep) uses the rear point (b in Figure 1). Also, the values of C_E and the variation of the equivalent load are calculated differently in these two references. A more detailed comparison of these two methodologies is presented by Nourzadeh et al. Nourzadeh et al. (2017), Nourzadeh (2017) and Nourzadeh et al. (2014).

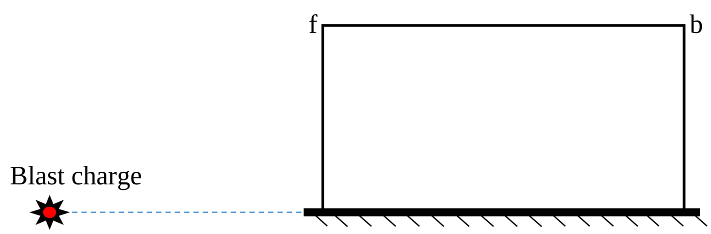

Figure 1: Definition of reference points in blast loading of roof beams

In previous research studies, it was shown that the two methodologies, namely ASCE/UFC3-340-02 and TM 5-855, generate significantly different equivalent blast time-histories, reaching more than 300% difference in peak pressure and more than 200% in impulse (Nourzadeh et al. 2017 and Nourzadeh 2017). Moreover, the response of the beams subjected to these equivalent uniform loads when compared to response from analysis using multi degree-of-freedom (MDOF) models in finite elements (FE) analysis showed significant discrepancies. The overestimation in maximum displacements of the beams under ASCE/UFC loading with respect to the FE analyses results reached nearly 900%, while this overestimation for TM 5-855 methodology was reported to be nearly 120% (Nourzadeh et al. 2017, Nourzadeh et al. 2015).

To further investigate the response of roof beams to a propagating blast wave, and to verify the FE analysis methodologies reported in the previous research studies, several experimental tests were carried out. The experimental tests were used to investigate the accuracy of the numerical analysis of response to a blast load traversing a roof element along the direction of blast wave propagation. The experimental tests were carried out at the facilities of Canadian Explosives Research Laboratory (CERL), Natural Resources Canada (NRCan) in Ottawa, Canada. The details of the experimental design and setup as well as the test results are presented in this paper.

2. TESTS SETUP

A blast table designed for use with small explosive charges was used for the testing. The explosive charge mass and table size considerations dictated the dimensions of the experimental specimens and the test setup. The overall dimensions of the blast table were approximately 2.36 m (6'-9") by 1.20 m (4'-0"), as shown in Figure 2. According to the regulations and protocols used in the field experimentation, the maximum amount of the explosive could not exceed 100 grams of C4 explosive (120 g of TNT-equivalent mass).

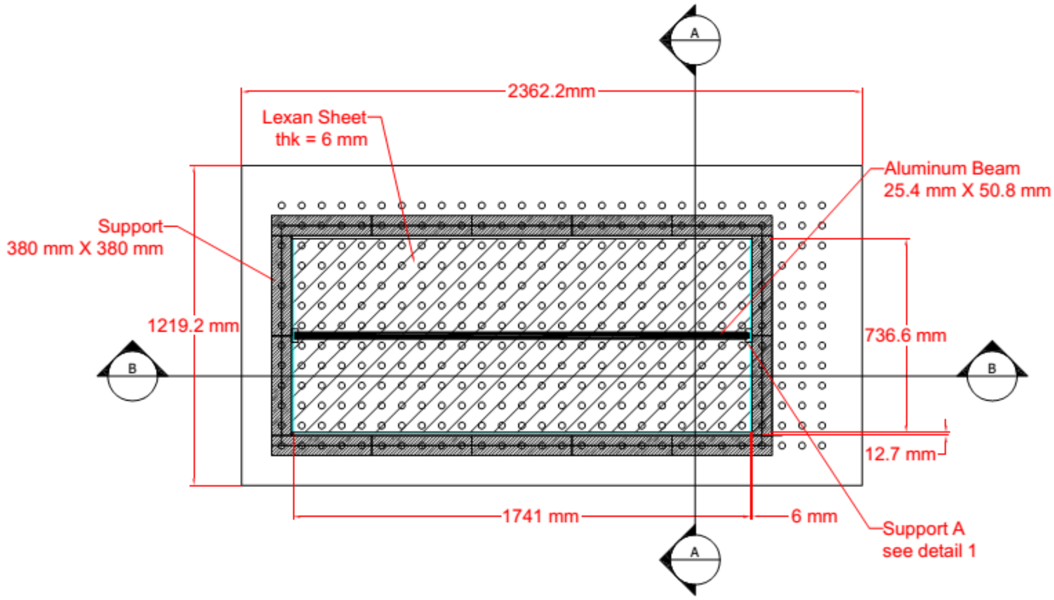

Figure 2: Plane view of blast table and the experimental setup

Since the explosions were going to be relatively small in scale, a small-scaled aluminum beam having a cross-section of 25.4×50.8 mm was used as the roof beam. The other properties of the aluminum roof beam, such as the support conditions and the span, were chosen to ensure that the beam response remained elastic throughout the test. This enabled multiple tests to be conducted using the same beam. A 6-mm thick Lexan sheet was used to increase the tributary area of the aluminium beam. The completed field setup is shown in Figure 3.

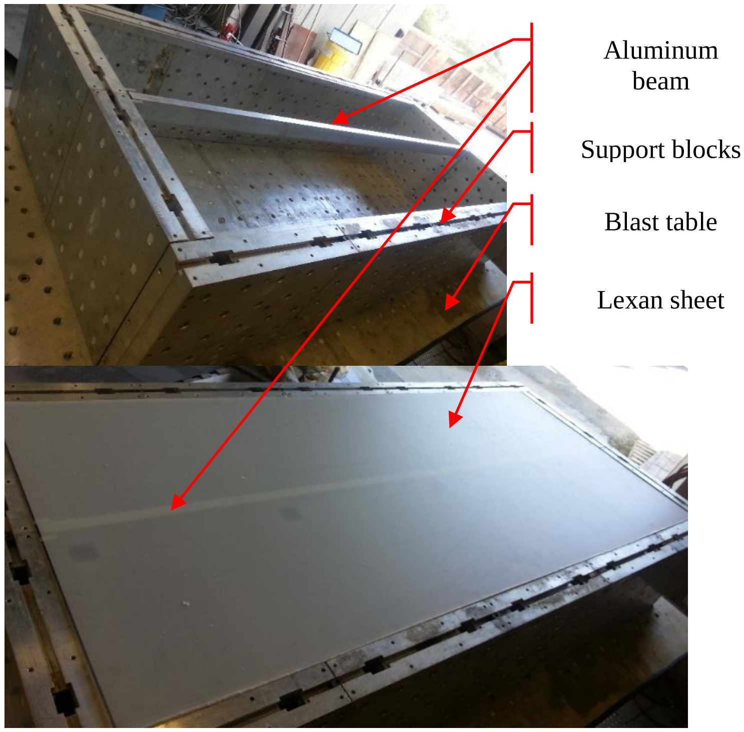

Figure 3: Experimental setup in the field

Instrumentation for the experimental tests included high-tension string potentiometers (string pots) attached to mid-span and quarter-span locations of the beam to capture the deflections of the beam. Also, Piezoelectric pressure gauges were installed at five locations along the span of the roof beam and used to capture the pressure profile of the traversing blast wave. Figure 4 shows the location of the blast pressure gauges (PG#) and string potentiometers (SG#). Figure 5 shows a photograph of the instrumentation installed on the blast table.

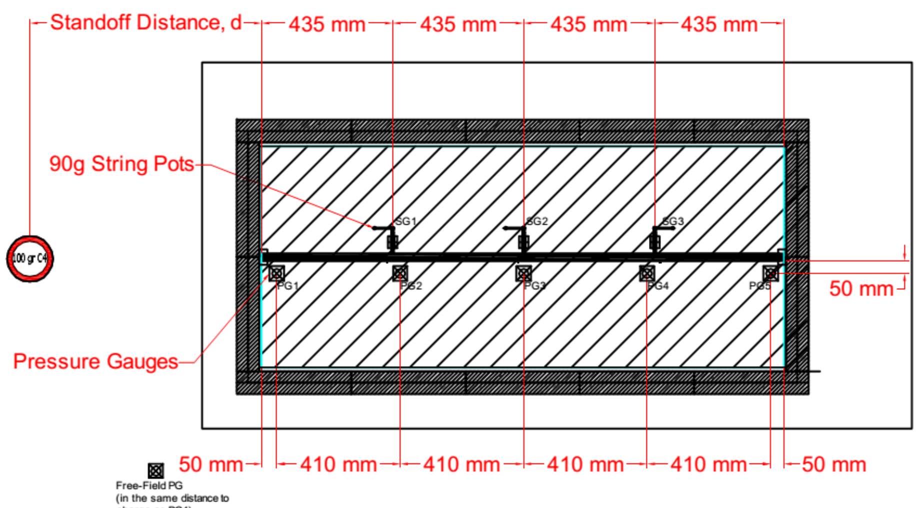

Figure 4: Plan of the measuring instruments used in the test

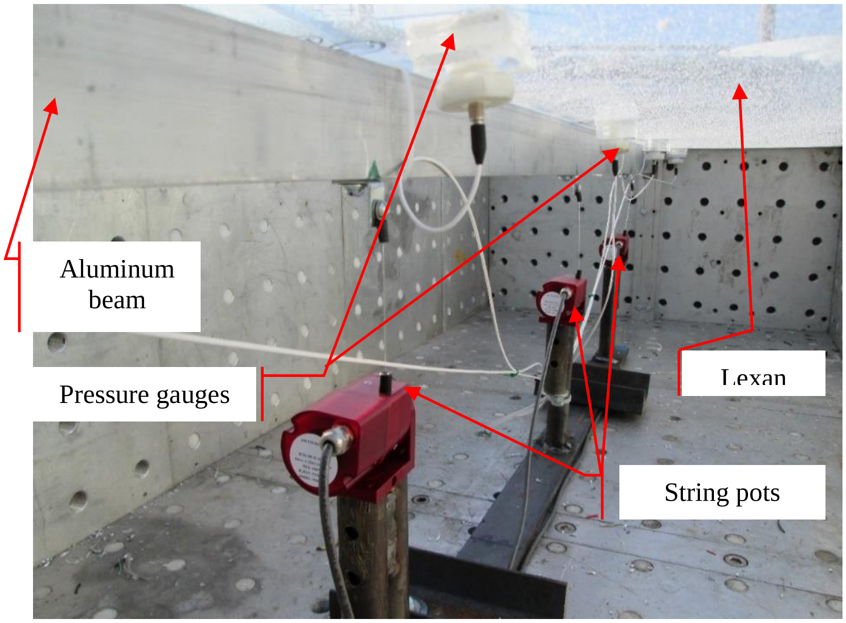

Figure 5: Inside view of the roof and measuring instruments used in the tests

The explosive charges were located at standoff distances ranging from 250 mm to 1000 mm from the front edge of the roof. The elevation of the charges was slightly lower than the roof elevation, so that a clear path to the roof was avoided to ensure an incident (not reflected) pressure distribution on the roof plate.

3. BLAST SCENARIOS

The 100-grams of C-4 explosive was detonated at 4 different standoff distances from the edge of the roof. The standoff distances used were 250, 500, 750 and 1000 mm from the roof edge. The pressure time-histories from the blast scenarios were calculated using ConWep and in accordance with Equation 1 and are compared to those measured during the tests in Figure 6. During the tests, some of the pressure gauges failed to function, therefore only pressures from the gauges that functioned are presented.

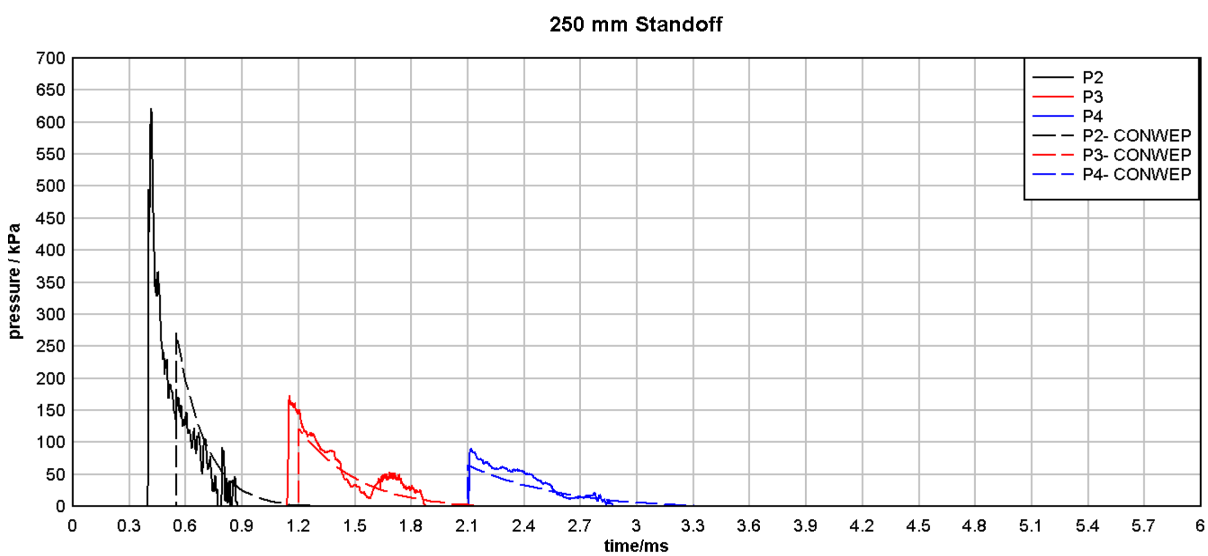

Figure 6: Comparison of measured pressures with those obtained from ConWep pressures and Equation [1]

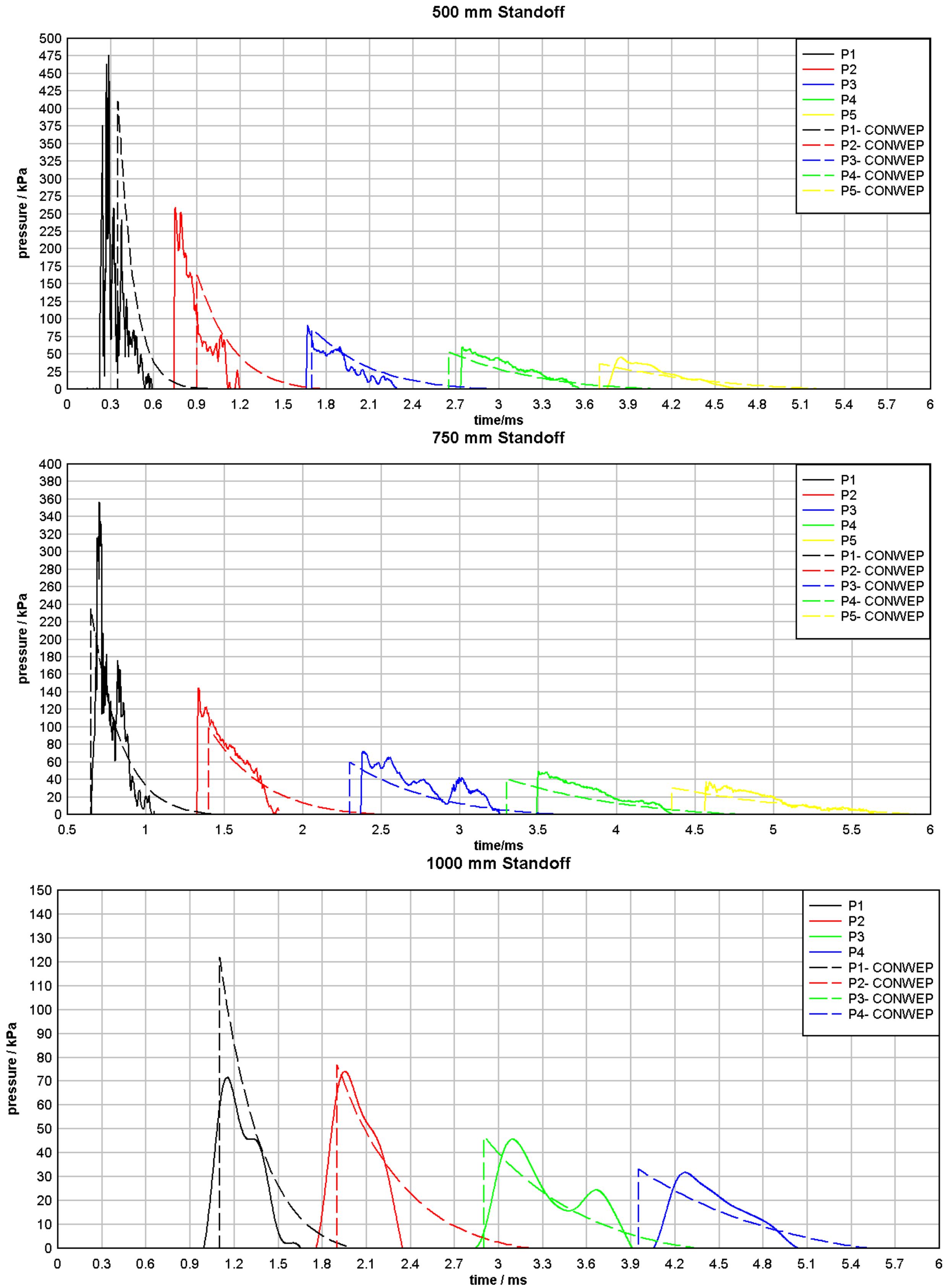

Figure 6 (Cont'd.) Comparison of measured pressures with those obtained from ConWep pressures and Equation [1]

As can be observed from Figure 6, the arrival times measured during experiments match those obtained from ConWep reasonably well, while the peak pressures are slightly less consistent. In most cases, the experimental peak pressures are higher than the ConWep values. The higher differences occur particularly at the closer ranges, that is, close to the front edge of the roof (PG1 gauge in Figure 4). This is because the expected turbulence from blast wave reflection and diffraction around the front face of the structure. Also, the negative phase of the experimental blast has been truncated for comparison with the ConWep results.

4. NUMERICAL SIMULATION OF THE EXPERIMENTS

The objective of the experimental tests was to compare the test results with the results obtained from the numerical models. For such comparison, the experimental test cases were modeled using single degree of freedom (SDOF) models with the equivalent uniform loads, as well as the FE models with travelling blast loads. The characteristics of the analytical model used in the simulations are presented in Table 1. The SDOF and FE models are generated based on the same methodology as that suggested by different references (Biggs 1964, DoA 1986 and DoD 2008), and analyzed for the four tested scenarios. The equivalent uniform loads are calculated based on both UFC/ASCE and TM 5-855 methodologies discussed previously.

Table 1: Characteristics of the analytical model for numerical simulation of the experiments

Parameter	Value
Length of beam (L)	1.7 m
Tributary width (L_{tr})	0.3683 m
End conditions	simply-supported
Modulus of Elasticity (E)	69.0 GPa (Aluminum)
Yield stress (F_y)	275.0 MPa (Aluminum)
Beam dimensions	25 mm× 50 mm (Rectangular)
Moment of inertia (I)	260.42×10^3 mm^4
Max. yield moment (M_y)	2.86 kN.m
Thickness of Lexan sheet (t_L)	6 mm
Mass per unit length (m)	6.027 kg/m

For assigning a realistic damping ratio to the numerical model, the responses of the beams in the field tests were examined using logarithmic decrement of the peaks in two consecutive free-vibration cycles ($u_{max,n}$ and $u_{max,n+1}$), the damping ratios (ξ) were determined for the different experimental cases using Equation 3 (Humar 2012). The calculated damping ratios are 7.9%, 8.6%, 5.0% and 4.0% in cases 1 to 4 (standoff of 1000 mm to 250 mm), respectively.

[3] $$\ln(u_{max,n}/u_{max,n+1}) = 2\pi\xi/(1-\xi^2)^{1/2}$$

Based on the parameters discussed above, a 2D dynamic analysis in OpenSEES software was conducted using a multi degree of freedom (MDOF) model of the roof beam. The sections are built using elastic elements, while the length of the elements is discretized into small sub-elements with lengths of 10 mm. The blast time-histories based on ConWep were applied to the end nodes of each element based on its range to blast charges. Direct integration of the equations of motion was carried out using Newmark's average acceleration method as in the SDOF analysis. The selected analysis time step was varied between 0.01 msec to 0.1 msec, depending on the number of iterations required to achieve convergence. The deflections at the mid-spans of the beams obtained from the SDOF analyses with equivalent uniform blast loading from the UFC/ASCE and TM5-855 methodologies and the MDOF analyses with travelling blast loading calculated in accordance with Equation 1 and with the experimental blast pressures are compared in Table 2 and Figure7 and Figure 8.

Table 2: Comparison of the maximum mid-span deflections obtained from numerical analyses and experiments

Test no.	Experimental results	MDOF model with travelling blast loads from ConWep	MDOF results with pressures from experiments	SDOF model based on UFC/ASCE	SDOF model based on TM 5-855
1	11.97	19.67	- *	78.02	17.77
2	9.04	14.62	11.09	81.21	16.34
3	7.94	12.55	8.87	37.86	14.52
4	5.67	10.13	- *	24.00	13.43

* some pressure gauges malfunctioned, so carrying out this analysis was not possible.

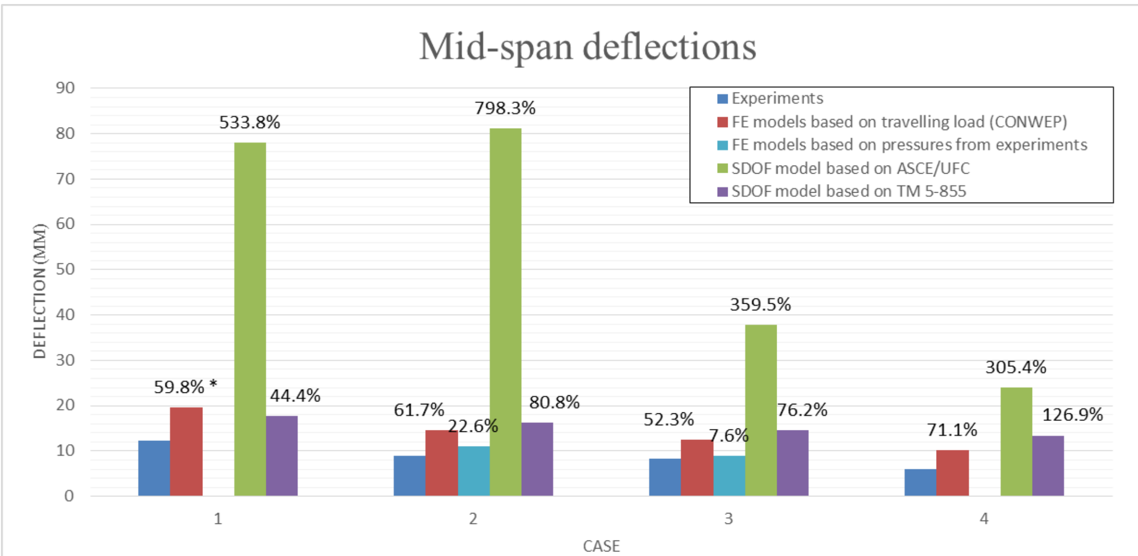

Figure 7. Results summary for the different cases
(* the labels show the difference with respect to experimental results)

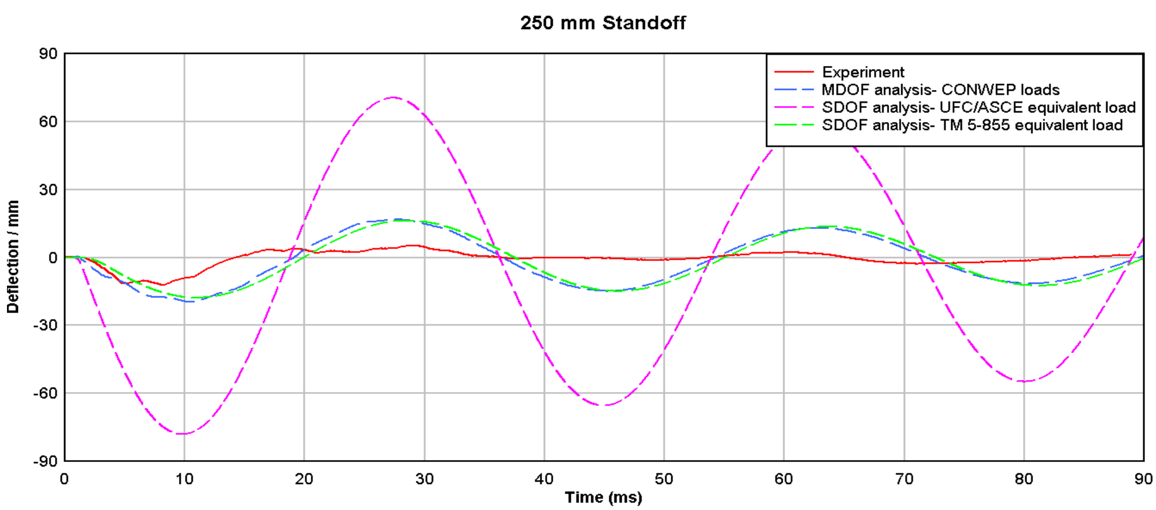

Figure 8. Comparison of mid-span deflections for different cases

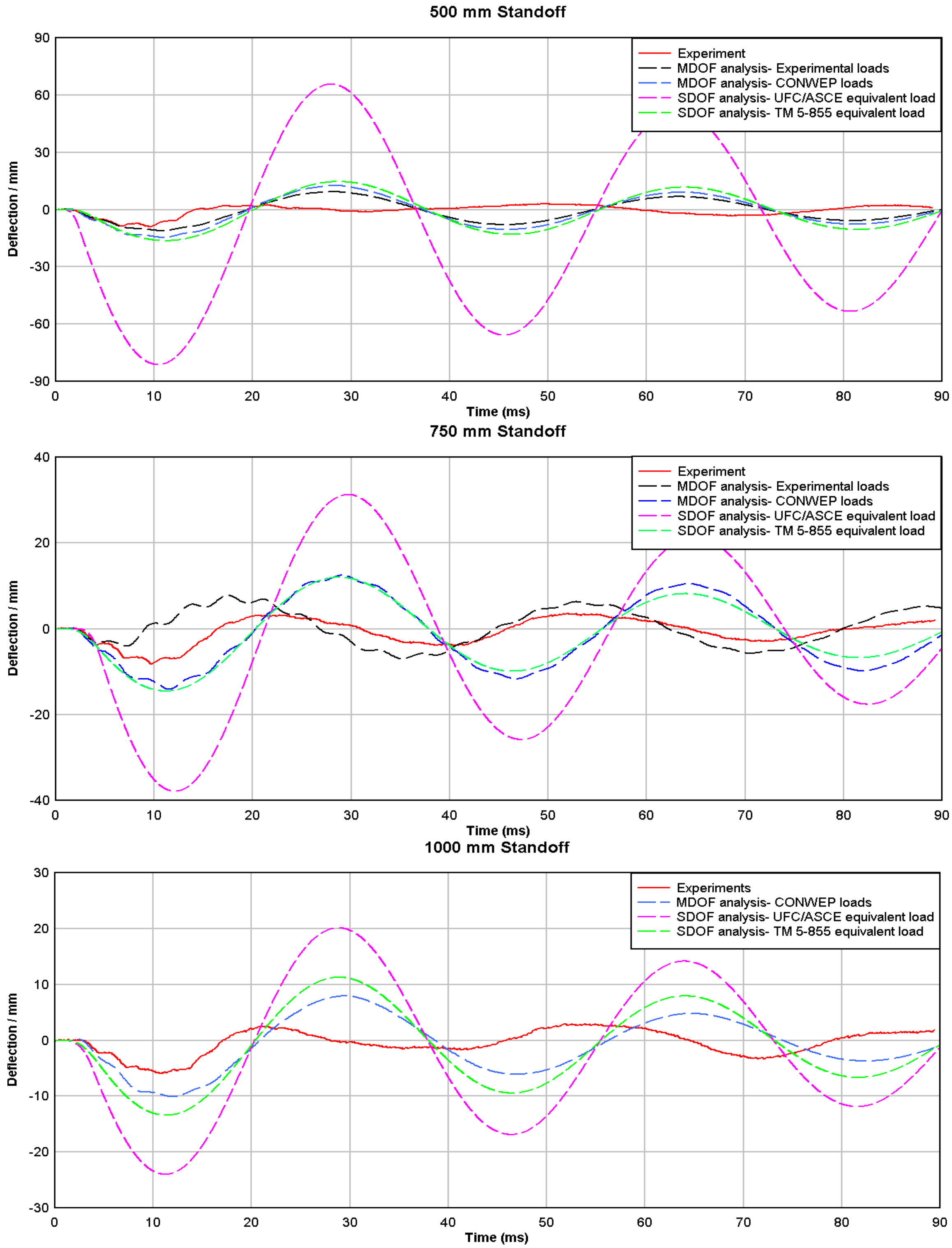

Figure 8. (Cont'd.) Comparison of mid-span deflections for different cases

5. DISCUSSION AND CONCLUSION

Several observations can be made and conclusions drawn from the results presented in the preceding paragraphs:

1. The difference between the pressure time-histories obtained from ConWep and those obtained from the test are more pronounced for the shorter standoff distances (250 mm), and for the first pressure gauge (pressure time-history P1 recorded at gauge PG1). One of the reasons is the high turbulences near the edge of the roof, which can affect the recorded pressures significantly.

2. On comparing the peak responses of the structure measured in the experiments and those obtained from numerical simulations, it is observed that generally the results from FE (MDOF) models provide a closer match with the results of the experiments. It appears that the differences between the results of the MDOF models and the experiments are mostly because of the difference in the loading, because when the recorded pressures are used instead of ConWep pressures, the accuracy of the numerical simulation improves significantly (errors going from 61.7% to 22.6% and 52.3% to 7.6% for standoff distances of 500 and 750 mm, respectively). It is therefore reasonable to conclude that the FE model with travelling blast loads, used in this research, can capture the response of the roof beams under blast loading.

3. Between the two equivalent uniform load methodologies, TM 5-855 provides the equivalent uniform loads that yield more accurate results. The use of equivalent loads determined from the ASCE/UFC method results in large overestimation of the response. Although the scale of the experiments dictated the use of the equivalent load coefficients at the margins of their applicable range, the results and trends observed are similar to the ones obtained in the previous stages of this research. Thus, it is suggested that the equivalent loads specified in TM 855 be used to determine the response in cases where for some reason, the more accurate MDOF models with travelling loads cannot be employed.

4. In all the experiments discussed here, it was observed that the maximum response of the beams occurred when the shockfront had cleared the whole beam span. This is contrary to what has been suggested in the TM 5-855 methodology, which defines a point on the roof at which the shockfront must be located to produce the maximum internal efforts in the beam.

6. ACKNOWLEDGEMENT

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7. REFERENCES

ASCE. 2010. *Design of Blast Resistant Buildings in Petrochemical Facilities*, ASCE Task Committee on Blast Resistant Design, Reston, Virginia, USA.

Biggs, J.M. 1964. *Introduction to structural dynamics*, McGraw-Hill, New York, USA.

DoA (US Department of the Army). 1986. *Fundamentals of Protective Design for Conventional Weapons*, TM 5-855-1, USA.

DoD (US Department of Defense). 2008, *Structures to Resist the Effects of Accidental Explosions*, UFC 3-340-02 (TM 5-1300, NAVFAC P-397, AFR 88-22), USA.

Humar, J.L. 2012. Dynamics of Structures, 3rd ed., CRC Press/ Balkema, Taylor and Francis Publication, London, UK.

Nourzadeh, D., Humar, J.L. and Braimah, A. 2014. *Blast Induced Roof Loads in Building Structures- Comparison with Empirical Formulations*, CSCE Annual Conference: 4th International Structural Specialty Conference, Halifax, Canada.

Nourzadeh, D. 2017. Response of Building Structure and its Components to Blast Loading, Ph.D. Dissertation in Civil and Environmental Engineering Department, Carleton University.

Nourzadeh, D., Humar, J.L. and Braimah, A. 2017. *Response of roof beams in buildings subject to blast loading: Analytical treatment*, Journal of Engineering Structures (138) pp. 50-62, Elsevier, In Press, DOI. 10.1016/j.engstruct.2017.02.009.