# STRUCTURAL CONTROL OF DYNAMIC BLAST LOADING USING FLUID VISCOUS DAMPERS

by

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## ABSTRACT

This paper evaluates the effectiveness of Fluid Viscous Dampers to reduce blast loading responses in steel buildings. The paper addresses the following issues: (1) development of a blast loading time history from a 3,000 pound explosive charge, (2) characteristics and historical applications of Fluid Viscous Dampers for blast and weapon effects, and (3) blast effects and performance comparisons of a conventional steel building frame with and without dampers, and a conventional concrete shear wall building. Simulation results indicate that Fluid Viscous Dampers provide a cost-effective way to greatly improve the performance of steel building frames under blast loading.

## INTRODUCTION

The purpose of this paper is to evaluate the effectiveness of Fluid Viscous Dampers (FVD) when used to control blast loading responses on lateral load resisting frames. High performance fluid dampers were originally developed for use on military systems subjected to high level shock inputs. Since 1990, this technology has been widely used by the structural engineering community to improve the seismic performance of structures. Recently, it has been suggested that the use of dampers for seismic energy dissipation may also improve a structure's resistance to blast, even though the time histories of seismic events are extremely dissimilar to blast pulses.

To evaluate this hypothesis, nonlinear dynamic force history analyses were conducted on three different types of structures: (1) Conventional Special Moment Resisting Frame (SMRF), (2) SMRF with FVD, and (3) Conventional concrete shear wall. The lateral load resisting frames of these structures were designed to conform to the 1994 Uniform Building Code, Zone 4 seismic design criteria. Nonlinear computer models with and without FVD were subjected to a dynamic blast loading from 3,000 pounds of TNT at 100, 40, and 20-foot standoff distances.

## BLAST LOADING TIME HISTORIES FOR A 3,000 LB CHARGE OF TRINITROTOLUENE (TNT)

The intent of this report is to study the relative performance of structures subjected to transient pulses caused by the detonation of explosives. Most explosives are developed and used primarily by the military and government agencies. Very little data is published in the public domain concerning blast pulse magnitudes and wave forms. The transient pulses presented here are for reference only. They were assembled entirely from an unclassified database of public domain material, and were appropriately scaled for use. In general, the frequency content from the time history of a detonation is at least an order of magnitude higher than the structural frequencies of a conventional building. Thus, it is not necessary to utilize high precision blast transients for determining structural response. Since only conventional buildings were to be studied, the extremely short explosive pulse durations also indicated that integrated pulse content was much more important than a highly precise wave form. For these reasons, all pulses were rendered generic by reducing them to an equivalent triangular wave form. The resultant time histories provide what is essentially a pressure impulse, which is then applied to the structure (See Figures 1, 2 and 3).

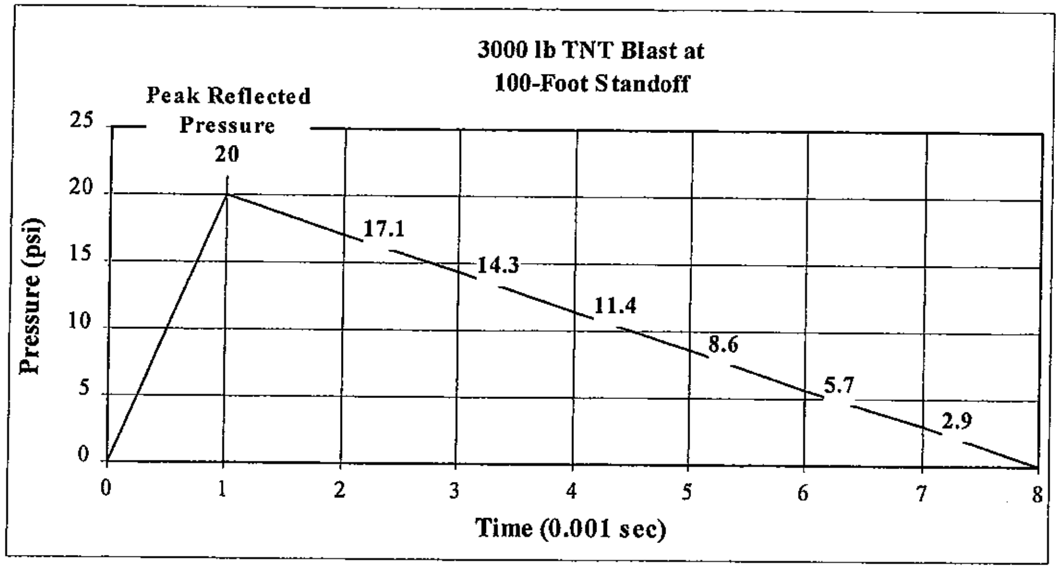



**FIGURE 1**

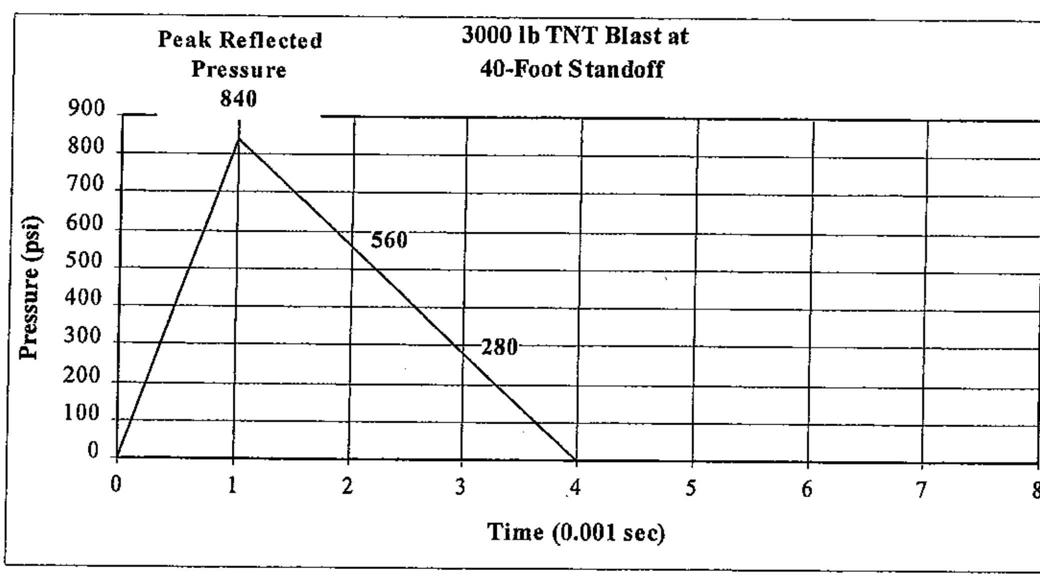



**FIGURE 2**

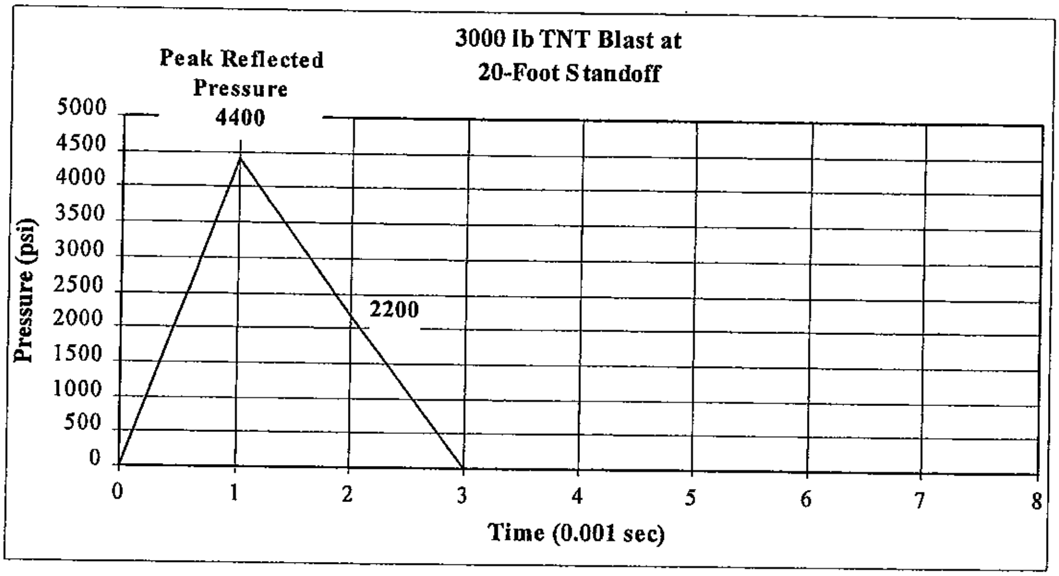



**FIGURE 3**

The time histories utilized are related to the detonation of a charge roughly equivalent in yield to 3,000 lb of TNT. The charge is assumed to be placed in a suitable container, which, in turn, is placed within an enclosed vehicle such as a truck or large van. No attempt was made to account for confinement of the charge within the vehicle. It must be remembered that the failure-inducing mechanisms of a conventional charge detonation include several very different components. These include:

1. Blast over-pressure
2. Ejecta
3. Cratering
4. Thermal Radiation
5. Induced ground motion.

The size and type of charge for the explosives, and their placement and range from the target will determine which of these components will dominate for a given event. For all cases studied here, blast over-pressure was considered to be the most significant failure-causing mechanism. However, as the range is decreased, it is expected that the damage caused by ejecta (objects and material driven from the vehicle and crater by the explosion) will become substantial. Cratering failures will eventually dominate, especially if the charge is placed in close proximity to one or more of the building columns, thus causing catastrophic failure.

## FLUID VISCOUS DAMPERS (FVD) FOR BLAST EFFECT

The fluid damper is well known throughout the military for its ability to arrest gun recoil, and numerous other military uses began in the post World War II era. Most applications were related to the protection of electronic systems on military platforms subject to attack by explosives. The platforms themselves had traditionally relied on strong and rigid design techniques for shock survivability. The resultant structures were said to be "shock hardened," and were truly massive and imposing to would-be enemies. Indeed, from the U.S. Navy's viewpoint, equipment is to be considered as flexibly mounted (i.e., base isolated) when a mounting frequency of less than 10 Hz is used (Clements [2]).

The technique of shock hardening could not easily be used on electronics systems and missiles, so fluid viscous damping devices began to see use for the shock protection of this equipment. Early examples include the Lockheed MK88 and the Unisys MK92 Fire Control System Antennas, the Raytheon SPS-49 Search Antenna, and the Raytheon MK29 Seasparrow Missile Launcher (Pusey [8]). Later application of this technology combined fluid dampers and spring elements, such as the Litton MK49 Ship's Navigator, Tomahawk, SM-2, and Seasparrow Missiles, and most large ballistic missiles, such as the Minuteman and Peacekeeper.

When the Cold War ended in 1990, much of the military's fluid damper technology became declassified and available to the public through former Defense Department suppliers. Today, more than 60 buildings and bridge structures in the United States utilize FVD to control earthquake response, taking full advantage of technology developed during the Cold War era (Constantinou and Symans [4]).

In general, weapons grade shock within the military begins at peak translational velocities in the range of 120 in/sec. This can be compared to the 55 in/sec peak recorded translational velocity of the 1994 Northridge, California earthquake. The upper limit of translational velocity that is considered "survivable" for military platforms is, of course, classified, but is generally considered to be in excess of 400 in/sec for structures designed to withstand near-miss nuclear detonations. Since fluid damper technology is now being used by both military and commercial projects, it is apparent that a commercial structure protected against earthquakes with fluid dampers should also be highly resistant to blast effects.

## DESCRIPTION OF STRUCTURES

Two steel moment resisting frame buildings with heights of 1 and 5-stories are used in the analysis. The typical floor-to-floor height is 14'-0", with the exception of the first floor height of 16'-4". The footprint in all cases is 105 ft x 130 ft. The floor diaphragms are composed of cast-in-place concrete over metal deck. Each building is designed to conform to the Special Moment Resisting Frame (SMRF) requirements of the 1994 Uniform Building Code for seismic Zone 4 and S2 soil conditions.

This study was conducted to examine the stability of a SMRF only. The exterior wall and diaphragm must be designed to withstand or absorb the blast loading. For example, the State Department's Office of Foreign Building Operations (FBO) requires that fenestrations be limited to 15% of each structural bay, and blast windows should be blast hardened (Gurvin and Remson [6]). High strength glass may be laminated glass, polycarbonate, or plastic interlayer (ASCE [1]). Exterior column and wall elements may be blast hardened by conforming to the UBC Zone 4 Seismic requirements, or by the use of Fiber Reinforced Plastic (Crawford [5]).

## 1-STORY MODEL

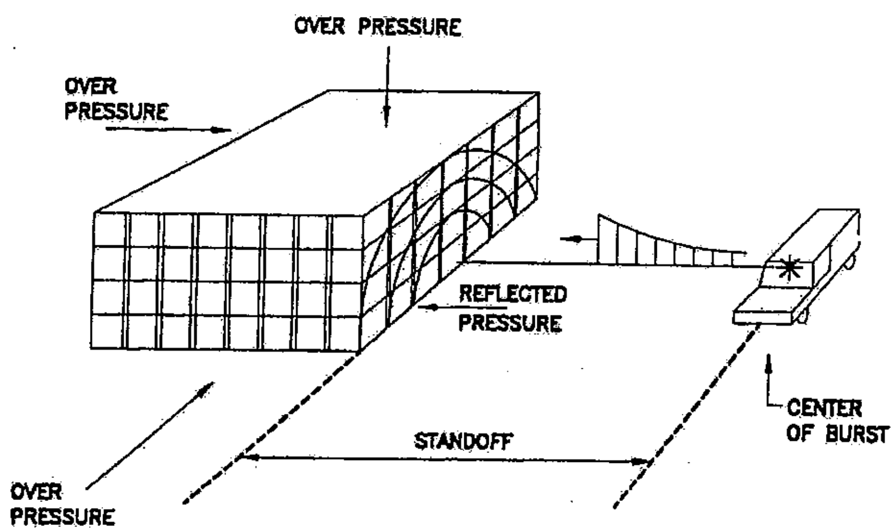
Two-dimensional models were constructed using Drain 2DX code (Prakash, Powell and Campbell [9]). Steel beams and columns are modeled as plastic hinge beam-column elements. A bilinear response is assumed with 5% plastic hardening. Yield strength is increased by approximately 20%, considering material over-strength and dynamic strength increase (at a fast strain rate, a larger load is required to produce yielding than at a lower rate (ASCE [1])). The Fluid Viscous Dampers are modeled as discrete damping elements mounted on chevron driver braces. Linear fluid damping elements were used, where damper output force is directly proportional to the velocity across the damper mounting points. Approximately 20% of equivalent critical damping by modal analysis is provided. An additional 5% of critical damping is assumed for global structural damping. In subsequent paragraphs, the SMRF with dampers is called a Damped Frame, and the SMRF without dampers is called a Bare Frame.

## 3,000 LB TNT BLAST AT 100-FOOT STANDOFF

A Bare Frame is subjected to a 3,000 lb TNT blast at a 100-foot standoff distance. The blast wave propagates by compressing the air molecules with supersonic velocity, and is reflected by the building, amplifying the over-pressure (Hinman [7]) (See Figure 4).

The Dynamic Time History Analysis indicates that the structural response parameter is insignificant for this loading, and no yielding occurs. Tabulated results at roof level are:

| MAXIMUM VALUES | BARE FRAME |
|---|---|
| Displacement (in) (drift) | 0.18 (0.0009) |
| Velocity (in/sec) | 2.39 |
| Acceleration (in/sec$^2$) | 602 |
| Base Shear (g) | 0.07 |




**Blast Loads on a Building (Hinman, 1998)**

FIGURE 4

This verifies that a minimum set back distance of 30 m (98 feet) for U.S. Embassy building and related structures may be adequate to protect lateral force resisting elements (Gurvin and Remson [6]). The majority of the blast energy is conserved by the kinetic energy in the structure since the duration of impulse is very short in comparison to the natural period of the structure. Shock spectra for dynamic amplification factors have been developed by Clough and Penzien [3]. These show that the larger the difference between impulse duration and the period of the structure, the smaller the displacement amplification (See Figure 5).

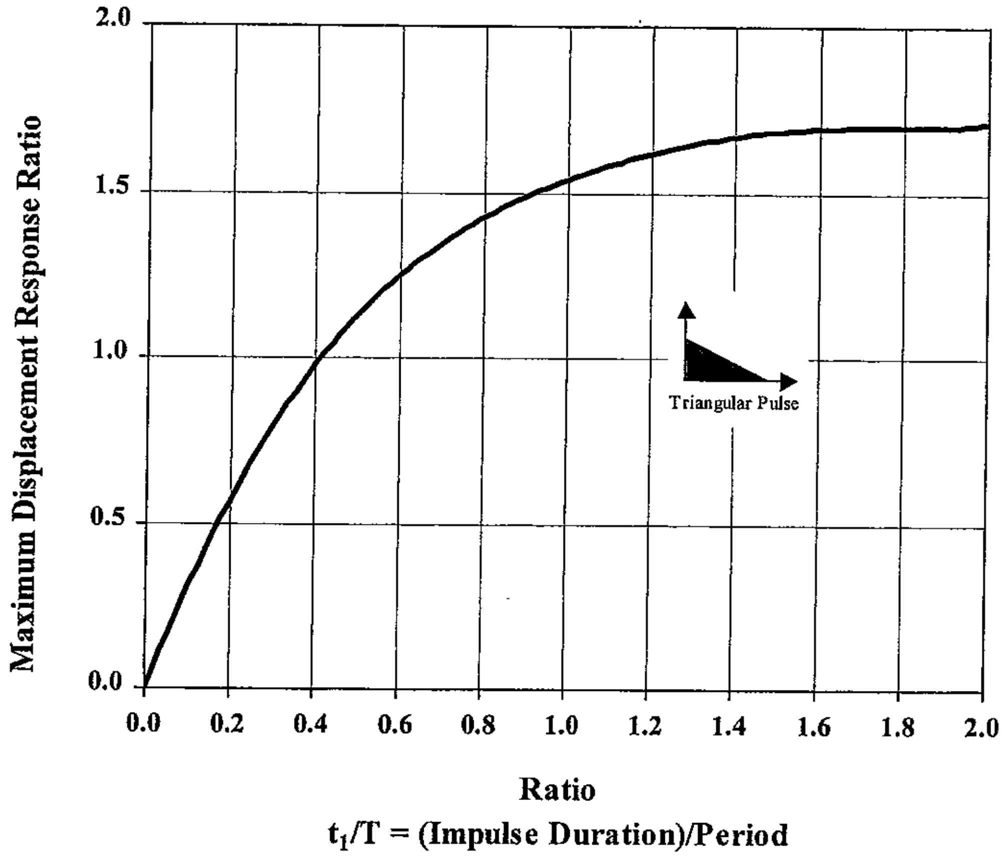



**FIGURE 5**
**SHOCK SPECTRA (CLOUGH & PENZIEN, 1993)**

### 3,000 LB TNT BLAST AT 40-FOOT STANDOFF

The Bare and Damped Frames are subjected to a 3,000 lb TNT blast at a 40-foot standoff distance. Minor yielding is observed for the Bare Frame. Although there is no significant difference between the Bare and Damped Frames in structural performance, approximately 30% reduction in maximum displacement is observed for the elastically responding Damped Frame. Tabulated results at roof level are:

| MAXIMUM VALUES | BARE FRAME | DAMPED FRAME |
|---|---|---|
| Displacement (in) (drift) | 2.6 (0.013) | 2.0 (.010) |
| Velocity (in/sec) | 35 | 35 |
| Acceleration (in/sec$^2$) | 1.76 E+4 | 1.76 E+4 |
| Base Shear (g) | 0.95 | 0.85 |
| Plastic Rotation % | 0.37 | - |

Figure 6 shows roof level displacement vs. time. Displacement and velocity decay at a faster rate for the damped frame. Maximum velocity and acceleration response occurs immediately after the blast. Figure 7 shows the Fluid Viscous Damper response for the damped frame.

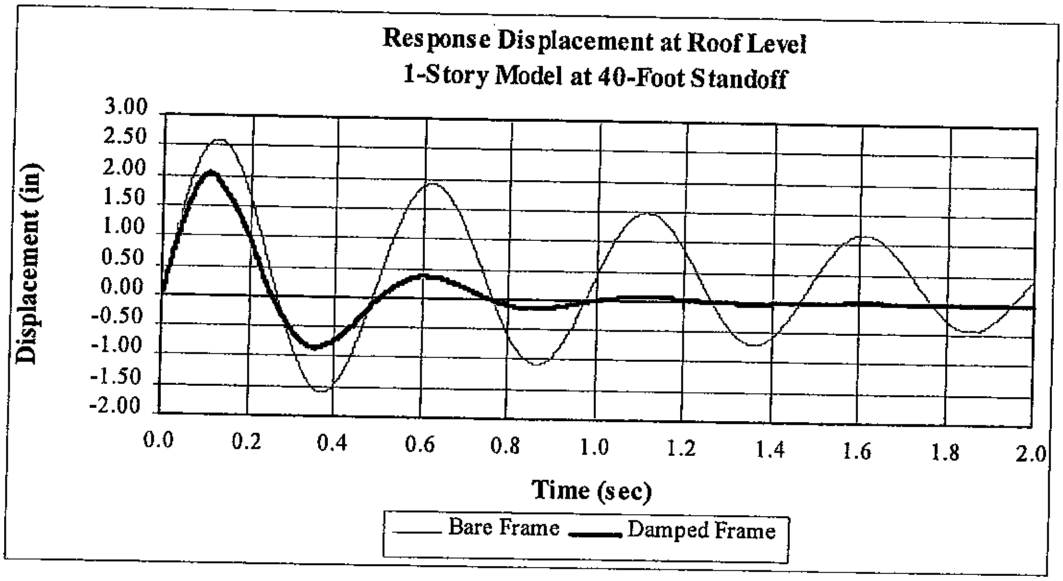



**FIGURE 6**

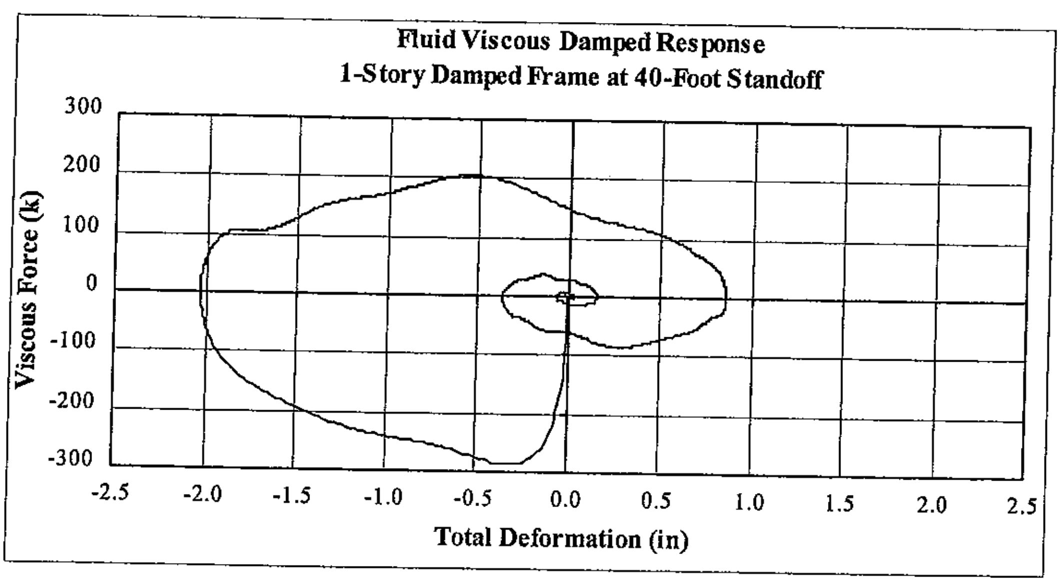



**FIGURE 7**

**3,000 LB TNT BLAST AT 20-FOOT STANDOFF**

The Bare Frame, Damped Frames, and Shear Wall models are subjected to a 3,000 lb TNT blast at a 20-foot standoff distance. A concrete wall 78 feet long and 24 inches thick is used for the shear wall model. The Shear Wall is modeled as a linear elastic panel element, and a 50% cracked section is assumed.

The Bare Frame experiences a collapse mechanism due to excessive plastic hinge rotation (5% or more). Viscous Dampers prevent the collapse of the Damped Frame by reducing drift and plastic hinge rotation. The permanent displacement is 5.44 in. Permanent displacement is caused by the following: (1) significant blast energy caused large plastic hinges in frames, (2) elastic strain energy in frames is not significant enough to bring back frames to their vertical position. The Shear Wall produces extremely high base shear, since the dynamic amplification factor is much higher than that for SMRF's. This shear produces brittle shear failure unless additional Shear Wall is provided or the existing walls are reinforced with extensive steel reinforcement. Tabulated results at roof level are:

| MAXIMUM VALUES | BARE FRAME | DAMPED FRAME | SHEAR WALL |
|---|---|---|---|
| Displacement (in) (drift) | 13.2 (0.068)* | 8.0 (0.04) | 0.83 (0.004) |
| Velocity (in/sec) | 112 | 112 | 113 |
| Acceleration (in/sec$^2$) | 7.53 E+4 | 7.53 E+4 | 7.44 E+4 |
| Base Shear (g) | 1.25 | 1.11 | 36.5* |
| Plastic Rotation % | 6* | 3 | -- |

* Failure

Figure 8 shows roof level displacement vs. time, and reveals that maximum displacement occurs at approximately 0.2 seconds. Figure 9 shows the Fluid Viscous Damper response for the damped frame.

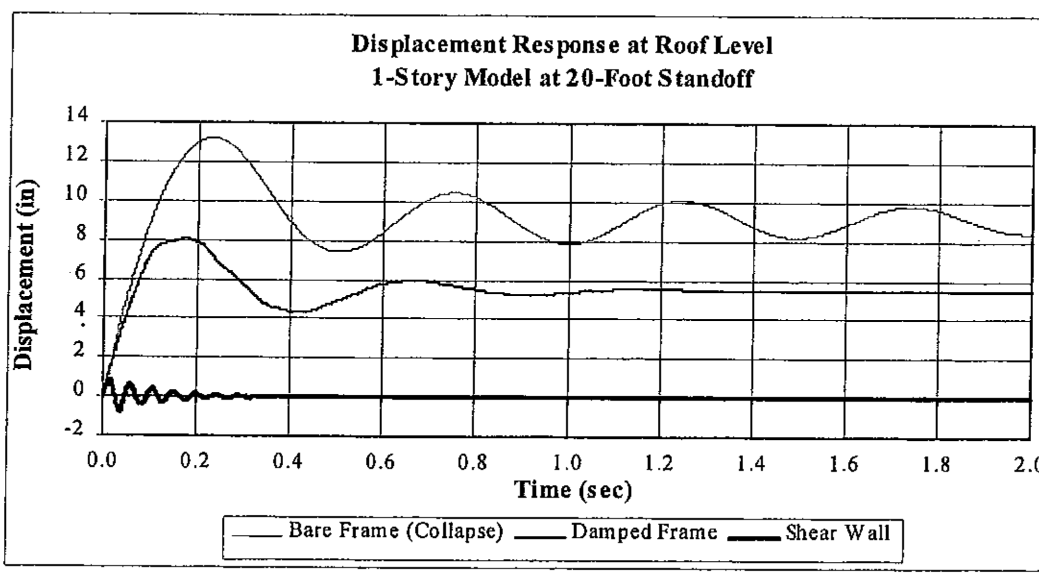



**FIGURE 8**

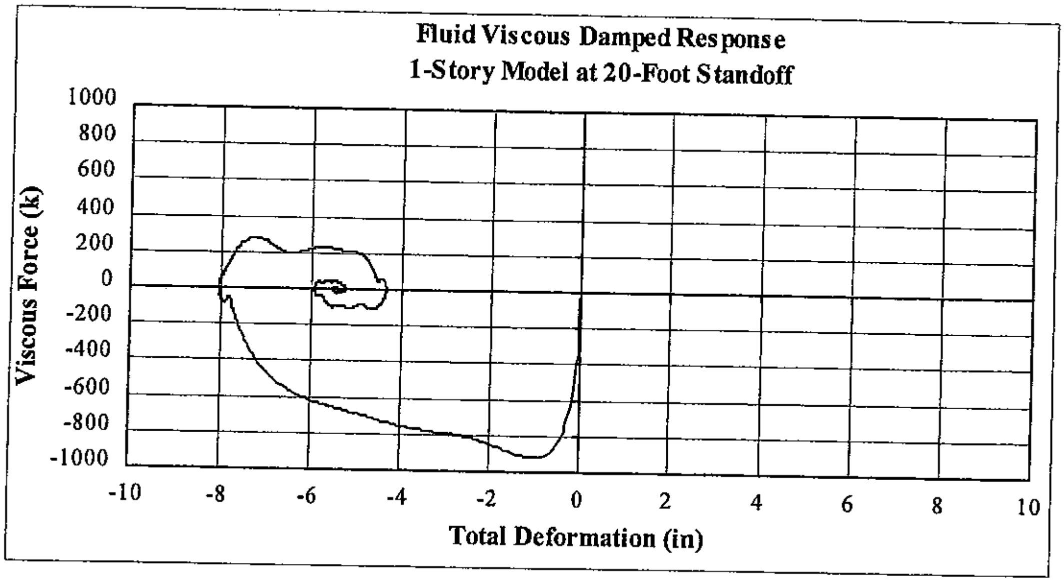



**FIGURE 9**

## 5-STORY MODEL

Two-dimensional models are constructed using Drain 2DX. The same procedures are used as in the 1-story model. Again, approximately 20% of equivalent critical damping by modal analysis is provided at each floor level. The Fluid Viscous Dampers are modeled as discrete damping elements, with linear damping. An additional 5% of critical damping is assumed for global structural damping.

## 3,000 LB TNT BLAST AT 100-FOOT STANDOFF

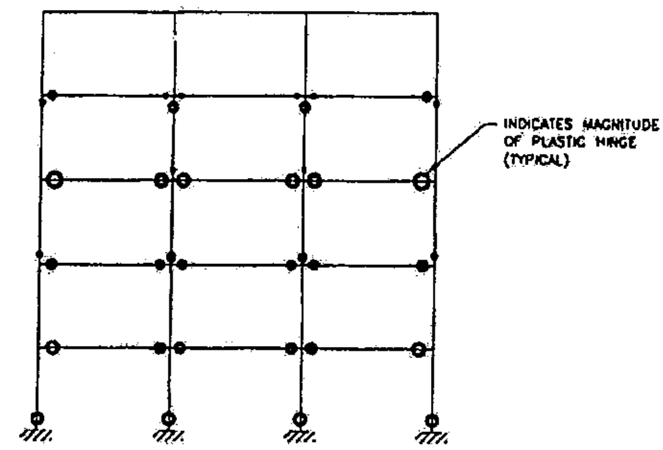
The Bare Frame is subjected to a 3,000 lb TNT blast at a 100-foot standoff distance. Pressure decrements at upper floors are also considered. Nonlinear Time History analysis indicates that the structural response parameter is insignificant from this distance, and no yielding of members is observed. Tabulated results at roof level are:

| MAXIMUM VALUES | BARE FRAME |
|---|---|
| Displacement (in) (drift) | 2.2 (0.0032 ) |
| Velocity (in/sec) | 8.6 |
| Acceleration (in/sec) | 159.9 |
| Base Shear (g) | 0.08 |

## 3,000 LB TNT BLAST AT 40-FOOT STANDOFF

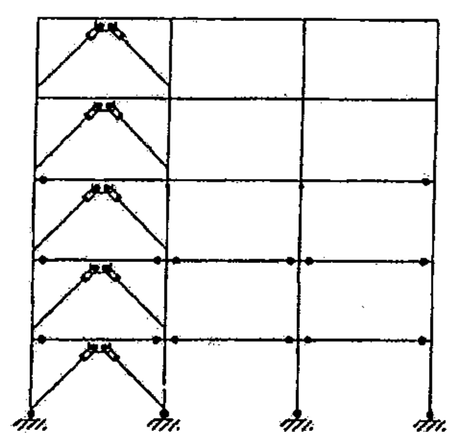
The Bare and Damped Frames are subjected to a 3,000 lb TNT blast at a 40-foot standoff distance. Yielding and permanent displacement is observed for both frames. Magnitude and quantity of plastic hinge rotation and displacement are significantly reduced for the Damped Frame. The permanent displacement is 13.36 in. (0.015 roof drift) for the Bare Frame and 3.5 in. (0.004 roof drift) for the Damped Frame.

Figures 10 and 11 show plastic hinge distribution for the Bare and Damped Frames.




**Bare Frame at 40-Foot Standoff**
**2.8% Maximum Plastic Hinge Rotation**

FIGURE 10



**Damped Frame at 40-Foot Standoff**
**0.79% Maximum Plastic Hinge Rotation**

FIGURE 11

Figure 12 shows roof level displacement vs. time. Displacement and velocity decayed at a faster rate for the Damped Frame. Maximum velocity and acceleration responses occur immediately after the blast. Figure 13 shows the Fluid Viscous Damper response for the damped frame.

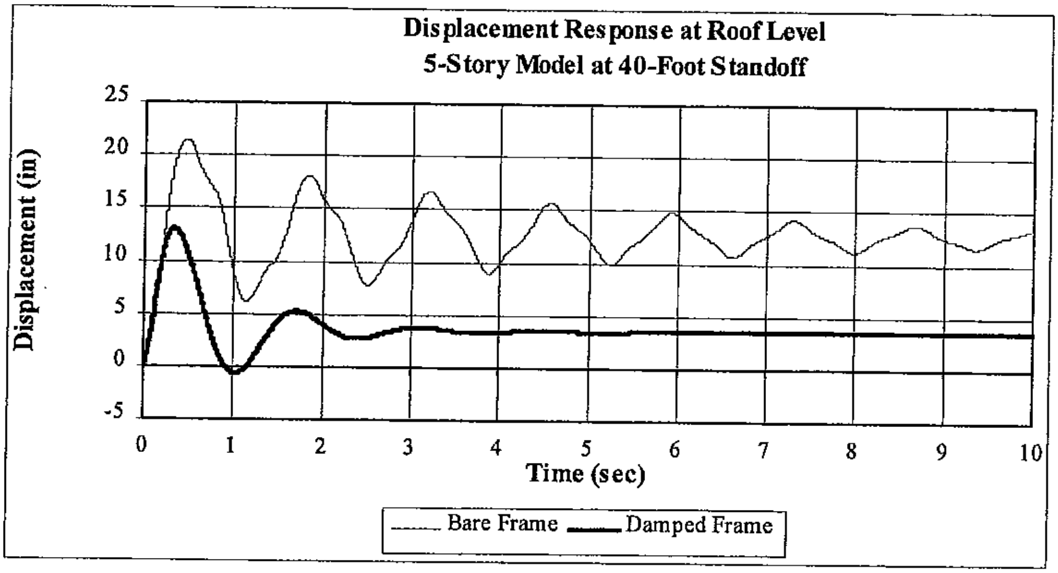



**FIGURE 12**

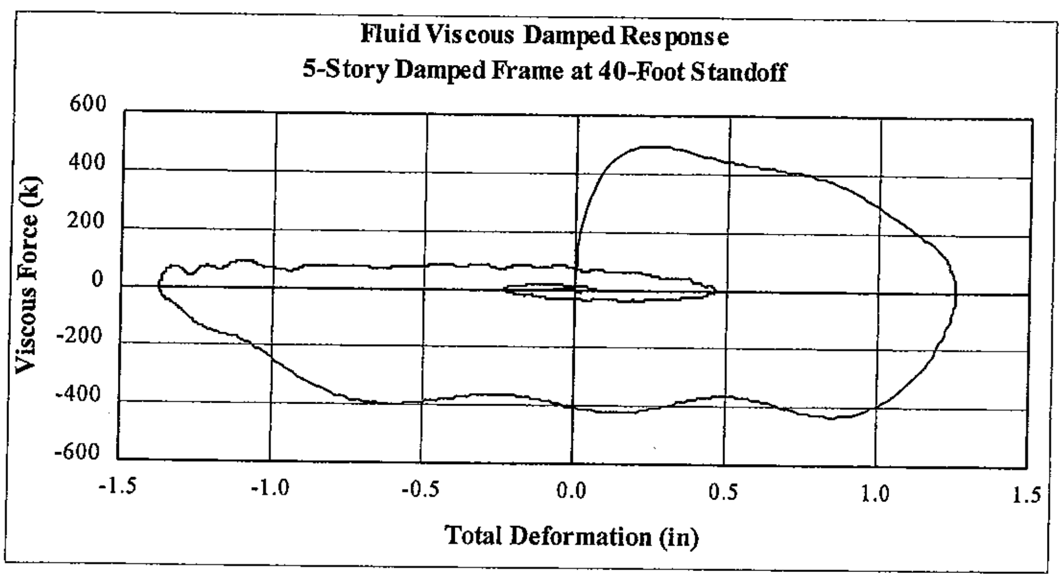
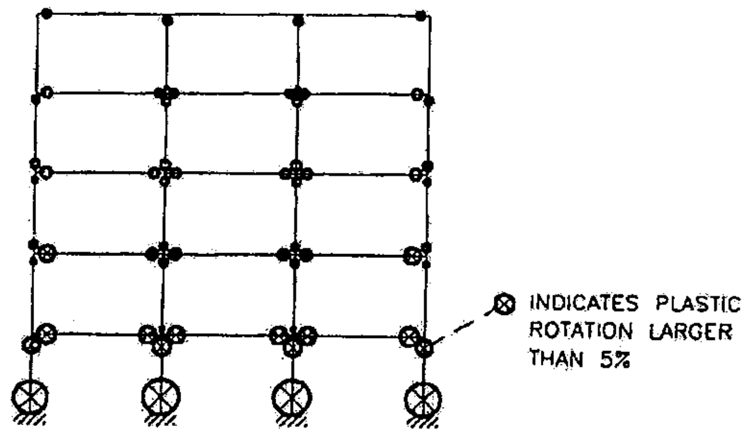



**FIGURE 13**

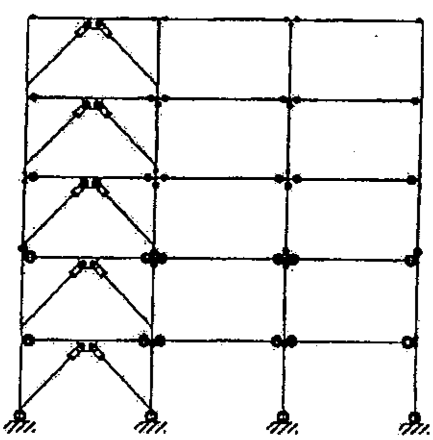
## 3,000 LB TNT BLAST AT 20-FOOT STANDOFF

The Bare Frame, Damped Frame, and Shear Wall models are subjected to a 3,000 lb TNT blast at a 20-foot standoff distance. A 5-story 78' long x 24" thick concrete shear wall is used for the Shear Wall model. The Bare Frame experiences the collapse mechanism at second and third story levels due to excessive plastic hinge rotation (See Figure 14). Failures of connections occur (plastic rotation of 5% or more) at 0.15 second and 9.5 inch roof displacement. The Shear Wall produces an extremely high base shear of 24.5 g, constituting structural failure. Viscous Dampers prevent collapse in the Damped Frame by reducing plastic hinge rotation and drift. Maximum rotation in the Damped Frame is 5.0%, which is considered significant inelastic demand in frame connections (See Figure 15). However, recent research indicates that some types of SMRF connections can be stable for this large demand (SSDA [10]). The permanent displacement at roof is 22.2 in. (0.026), and maximum displacement is 34 in. (0.04) for the Damped Frame. Figure 16 shows roof level displacement vs. time. Figure 17 shows Fluid Viscous Damper response for the damped frame.




**Bare Frame at 20-Foot Standoff**
**14.3% Maximum Plastic Hinge Rotation (Failure)**

FIGURE 14



**Damped Frame at 20-Foot Standoff**
**5.0% Maximum Plastic Hinge Rotation**

FIGURE 15

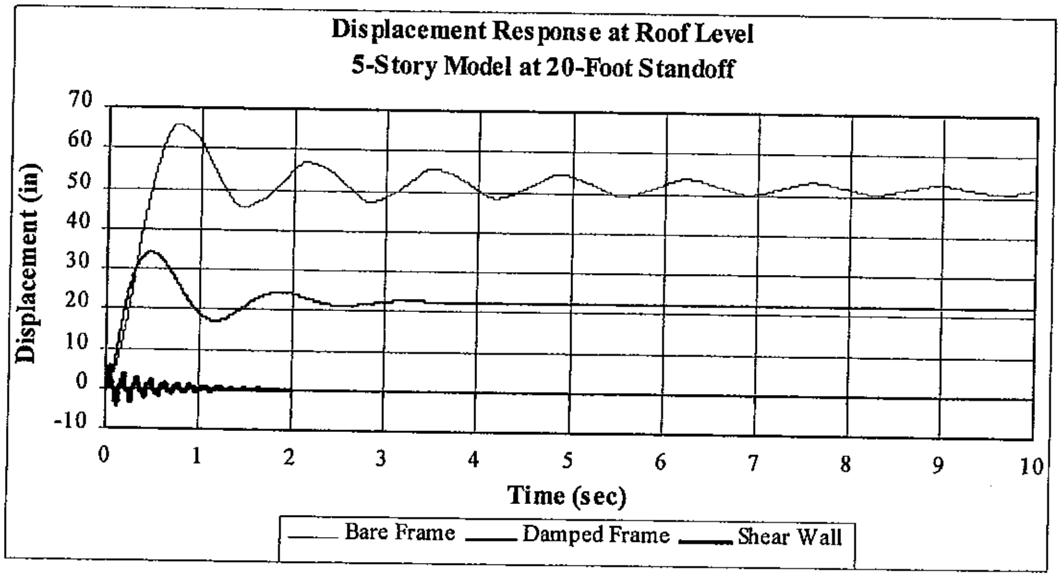



FIGURE 16

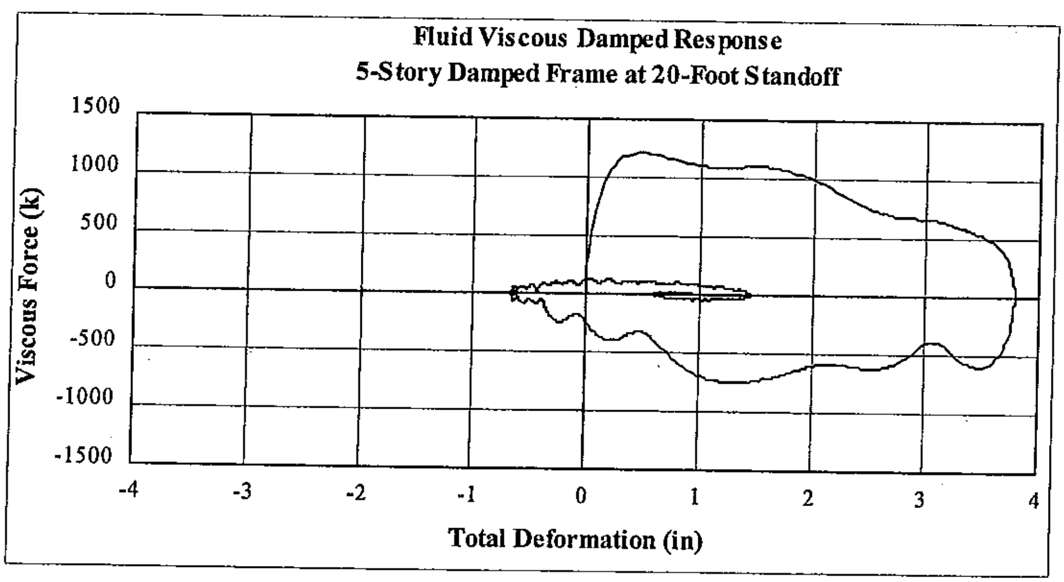



FIGURE 17

## CONCLUSIONS

The results of this study provide the following conclusions:

1. 3,000 lb TNT blast at 100-foot standoff distance does not cause significant structural responses to the candidate buildings.

2. 3,000 lb TNT blast at 20-foot standoff distance may cause failures in Bare Frames.

3. Fluid Viscous Dampers reduce inelastic demand and story drift, and can prevent failures in moment frames.

4. Added fluid damping does not affect maximum velocity and acceleration responses in the structure.

5. Concrete shear walls cause high strain energy demand and shear forces in the structure.

6. The SMRF with fluid dampers is a very effective system to control large blast loading.

This study indicates that a SMRF with Fluid Viscous Dampers is a very effective system to increase structural performance during large blast loading. The exterior concrete skin should be connected to the diaphragm by out-of-plane connections only, rather than providing in-plane shear transfer. This out-of-plane connection should be flexible enough to reduce energy transfer to diaphragm and frames. Non-ductile moment frames can be simply retrofitted with fluid dampers, since the dampers reduce or eliminate inelastic demand in the frames.

Using shear walls to control the blast impulse may produce the so-called "chasing tail" syndrome in structural design, i.e., adding shear wall increases the frequency and causes higher dynamic strain energy. The above 1-story example produces 36.5g base shear at a 20-foot standoff distance. This shear force requires substantial additional shear wall or steel reinforcement.

In summary, Fluid Viscous Dampers absorb significant amounts of energy throughout the duration of the structural response to a blast input. After the blast occurs, a majority of the blast energy is conserved by kinetic energy, therefore the amount of damping does not affect maximum acceleration and velocity. Maximum velocity and acceleration occur shortly after the blast impulse. Maximum displacement occurs at a somewhat later stage in the time history. Therefore, damping energy reduces the strain energy contribution and reduces maximum displacement. Large blast impulse loadings, from standoffs of 20 feet to 40 feet, overcome kinetic energy and cause inelastic response in the structure. Fluid Viscous Dampers are a very effective tool in reducing this inelastic demand by adding large amounts of viscous damping energy dissipation.

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