

Analysis and Experimental Evaluation of In-Fill Steel-Stud Wall Systems under Blast Loading

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Abstract: To be able to develop and advance blast-retrofit technologies, it is crucial first to be able to develop prediction methodologies and engineering design tools. Therefore, this paper will present the analytical modeling and experimental evaluation of steel-stud wall systems under blast loads. The results of the static full-scale wall tests, as well as the component tests, are used to evaluate the structural performance of the walls and provide recommendations for blast-retrofit systems. The analytical and experimental static results are used to develop the static resistance function for the wall systems, which is incorporated into a single degree of freedom dynamic model. The dynamic model will enable designers to predict the level of performance of the wall system under any explosion threat level. The analytical model conservatively predicted the measured field results with a maximum difference of 20%. The paper will discuss the performance of blast-retrofit wall systems under static and dynamic field tests simulating large vehicle bombs.

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Introduction

Much research has been conducted, and continued research efforts are being made, to improve the safety of infrastructure against natural hazards. Few research efforts have been made to mitigate the impact of manmade hazards, such as an explosion, on the civilian infrastructure that is most vulnerable to terrorist attacks. The recent rise in the incidence of terrorist bombings of high profile buildings has led to increased fervor in the development of a variety of blast-resistant construction systems, which may be applied to the exterior walls of buildings. The primary concern is the protection of building occupants from debris hazard and the prevention of progressive collapse.

Many commercial buildings today are constructed using reinforced concrete or steel frames with in-fill wall systems, such as concrete masonry unit walls or steel-stud walls. The in-fill systems currently constructed are designed to resist only natural loads such as wind, and to some extent, earthquake loads. For these infill wall systems, the design criterion is specified by a midpoint deflection and stress limit within the elastic response. In-fill steel-stud walls are nonstructural, and the chief hazard from these walls is debris, not structural collapse.

Steel-stud members have the desired combination of strength and ductility for blast resistance. The steel-stud walls can be con-

structed as retrofit walls placed inside existing exterior walls, or as exterior infill or curtain walls used in new construction. To design blast-resistant walls using steel studs, it is necessary to ensure a ductile wall performance under large deformations due to blast loads. Ductile performance requires that selected ductile components yield, but continue to carry loads and absorb significant energy through the plastic response. Thus, the potential premature failure modes must be prevented.

Steel-stud walls are constructed using cold-formed steel studs connected top and bottom to cold-formed steel tracks using self-drilling screws. Bottom tracks and top slip tracks, simulating a pin-and-roller system, are normally used in conventional steel-stud wall construction (AISI 2002), which fails under relatively low lateral pressure. It was shown by Muller (2002) that such connections are not sufficient to enable the wall system to plastically deform and utilize most of the stud ductility. Under lateral pressures slightly larger than what is needed to yield-buckle the compressive flange of the stud, a plastic hinge mechanism forms and the whole wall collapses due to system instabilities. Using deep rigid tracks improved the wall performance slightly, but not sufficient for pressure levels experienced during external explosions.

Oriented strand board or gypsum board is normally used as an external sheathing for the construction of steel-stud walls. Under large lateral pressure, such wall sheathing breaks due to excessive bending between any two adjacent studs. As a result, the studs lose the continuous lateral stability provided by the sheathing, and therefore the wall fails prematurely due to lateral-torsional buckling (Muller 2002). In addition, with the sheathing destroyed, the blast pressures and debris may enter the room and compromise the safety of the people inside.

Therefore, connections and external sheathing are properly designed to prevent undesirable premature failures. Stronger connections will transfer more load to the supporting structure, and therefore, the supporting structure should be designed more robustly to prevent progressive collapse. In this paper, the research effort to prevent anchorage failure is briefly presented first followed by the analytical model and full-scale wall and component

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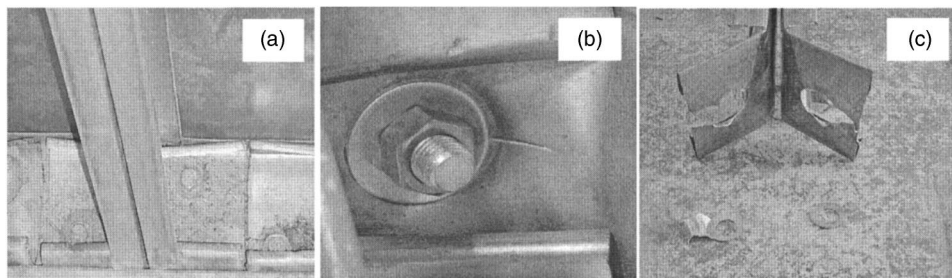


Fig. 1. Preliminary connection designs: (a) connection detail using clipped flanges and bent webs; (b) crack propagation; and (c) bearing failure

static tests to arrive at the static resistance function. The analytical prediction and design method are then summarized and used to design three full-scale walls for validation using dynamic field tests simulating large vehicle bombs.

Prevention of Premature Brittle Failures

The original concept for anchoring these steel studs involved cutting the flanges of the steel stud approximately 152 mm from the end of the stud, and then bending the stud at the web to form a “foot” at the top and bottom of the stud. A hole was then punched in this foot to allow anchorbolting of the stud to the floor and ceiling slab (Fig. 1). The bolt holes into the slab were either centered or staggered to prevent failure of the slab due to a tension crack joining the holes in the concrete, resulting in failure of the anchoring system, as occurred in earlier experiment. This method was used successfully in an Embassy Wall Retrofit Program (EWRP) research program, full-scale dynamic experiment EWRP-2, which is described in U.S. State Department Technical Information Bulletin (U.S. DOS 2001) and by Wesevich (2001). However, this method does not significantly increase the connection capacity from that of the conventional stud-to-track screw method, since the capacity of the connection is limited by the tension required to either fail the stud web in shear (a very localized failure), or failure by the foot pulling over the anchor nut or washer in bearing failure [Fig. 1(b)].

The key to utilizing steel studs in blast-resistant systems is designing the steel-stud connection so that premature brittle connection failures are prevented. The anchorage capacity is designed such that the stud itself fails due to yielding of the steel-stud cross section in tension, and eventual failure due to strain elongation limits of the yielded section. The ductile behavior allows for significant energy absorption during plastic elongation of the steel stud, while limiting the reaction forces required at the steel-stud connections to the floor and ceiling. The anchorage design study conducted by Shull (2002) focused on the development of an anchoring system to attach the steel stud to the floor or ceiling slab in order to develop the full tensile capacity of the cross section of the steel stud. The approach to designing the required anchorage involved static analyses and laboratory testing so that the connection capacity exceeds the tensile yield capacity of the stud. Details of the anchorage design are given by Shull (2002) and Salim and Townsend (2004).

The concept chosen for development uses a steel angle slightly narrower than the web width of the stud, with the vertical leg of the angle attached to the web of the stud, and an anchor bolt through a pass hole in the horizontal leg of the angle anchored into the floor and ceiling. This angled connection was designed by Shull (2002) to prevent connection failure in a variety of modes:

(1) shear failure of the angle-to-stud connecting bolts; (2) tension failure of the angle/stud in cross section; (3) block shear between bolt holes in steel stud; and (4) bearing failure in steel stud directly below bolt holes.

A 12.7 mm-thick steel angle was selected to prevent pullover of the anchor bolt as shown in Fig. 2. The anchor bolt connection into the slab was designed according to the concrete capacity design anchor design methodology as described in the ACI 318-02, Appendix D (ACI 2002) using appropriate factors for anchor spacing and edge effects.

To prevent premature failure of the sheathing, which could lead to system instabilities, ductile steel sheets (1.37 mm thick) were selected to span the spacing between the studs on the exterior face of the wall. Connection of the steel sheets to the steel studs at the edges is properly designed.

Once the connection details were determined and tested, the next step was to determine the load versus deflection response (*static resistance function*) of a transversely loaded steel-stud wall, which can be used for dynamic modeling. The dynamic modeling, the static modeling, and the static experimental program to develop the static resistance function are summarized next followed by the field verification using live explosives.

Theoretical Modeling

When a blast occurs near a structure, a very high pressure is applied in a very short duration. The structural response under severe loadings of this nature is significantly different from much slower loadings, such as wind loading. If the structure is not able to elastically absorb all of the blast energy, then permanent deformations will occur, and could result in complete failure. It is important to understand the behavior of a system under such extreme loadings to design against them. This portion of the paper focuses on the dynamic behavior and modeling of steel stud walls. A definition of “failure” will be given, and a single degree

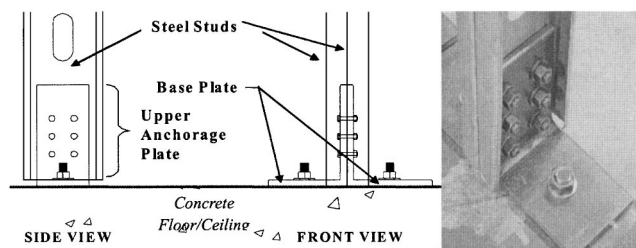


Fig. 2. Typical stud-to-floor/ceiling anchorage using a 12.7 mm thick steel angle

of freedom (SDOF) model will be introduced as well as a static resistance function, both of which are necessary to predict the response of walls under blast loads.

Blast Load

Dynamic modeling of a steel-stud wall system is used to predict the response of the wall under a certain short-duration blast loading. Any blast load can be defined using two parameters, the pressure of the blast and the impulse. When a blast occurs, a violent release of energy occurs producing a high-intensity shock front that expands outward from the surface of the explosive. As this shock front, also called a blast wave, travels away from the source, it loses strength and velocity, and increases in duration. As the blast wave expands in the air, the front impinges on any structure within its path, resulting in a pressure force being applied to the structure [U.S. Departments of the Army, Navy, and Air Force—(TM5-1300) (1990)]. The impulse of a blast can be defined as the area under the load-time curve.

A pressure applied to a structure over a short period of time has a lower impulse than the same pressure applied over a long period of time. A highly impulsive loading consists of a relatively high pressure applied quickly, while a static loading consists of a pressure that slowly rises to its peak value applied over a long period of time. If the duration of a blast pressure applied to a structure is very short compared to the natural frequency of the structure, the load can be considered as pure impulse (Biggs 1964).

Importance of Modeling

To protect people inside buildings from blasts, it is critical to know how the structure will behave under a certain pressure and impulse. If a building is not able to absorb the amount of energy created by a blast, the walls of the building will fail and the structure may collapse. It is usually impractical to design a building against blasts so that the structure remains undamaged. However, the objective of dynamic modeling and design is to minimize injury and death of the occupants. Modeling allows researchers to determine whether a building will survive a certain blast loading, and to predict the condition of the building afterward. So, given the two parameters of pressure and impulse, the dynamic response of a structure can be determined using engineering methods.

In this paper, wall failure is considered near collapse of the wall system. Typically, in design, failure is any behavior beyond the elastic region of response, and if a steel beam buckles in a building under normal loads, it is considered to be failure. In the field of blast design, the plastic region, or postbuckling region, is a critical part of the behavior and is the region where a system absorbs a large amount of energy in resisting extreme loads. This concept of energy absorption is of great importance in structural design for dynamic loads, since it accounts for a large portion of the blast-resistant capabilities of a wall system. Dynamic modeling of steel-stud walls requires knowledge of the loading, the pressure, and impulse, and accounts for both the elastic and inelastic response regions.

All structural dynamic systems contain a certain amount of damping and the effect can be significant if the load is oscillatory in nature. Structural damping during plastic response cannot be clearly defined or verified experimentally, and therefore its effects should not be considered in the plastic region of response (Kiger

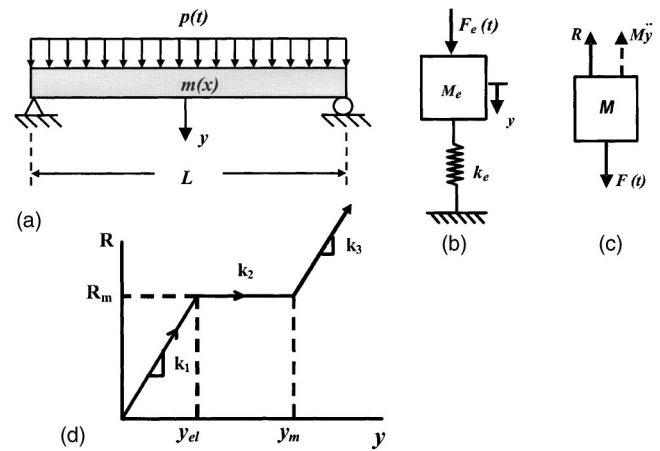


Fig. 3. Beam idealized as spring-mass system: (a) uniformly loaded simply supported beam; (b) single degree of freedom spring-mass system; (c) free body diagram of mass; and (d) resistance function

and Salim 1998). For the purposes of this paper, damping will not be a factor in the wall systems that are modeled.

Dynamic Modeling

Dynamic modeling in this paper refers to predicting the behavior of a steel-stud wall system under a blast load. There are two primary methods for dynamic modeling: Numerical procedures and rigorous solutions. Numerical analysis is the most general and straightforward approach and is based on physical phenomena (Biggs 1964).

Real world structures can be idealized and represented by a combination of springs and masses. For example, a beam that is subjected to a uniform pressure, such as a dynamic load, can be represented by a simple spring-mass system, shown in Fig. 3(b). In order for modeling to be accurate, the idealized system must represent the actual structure. Therefore, it is important to select the proper system parameters, these being the spring constant k_e and the mass M_e . The spring constant k_e is simply the resistance of the system and can be found from the properties of the beam, since it is the ratio of force to deflection. For complicated systems, the force-displacement relation cannot be defined by a single k_e , and thus the *static resistance function* is normally utilized and will be described later.

To define an equivalent one-degree system, like the one shown in Fig. 3(b), it is necessary to evaluate certain parameters, these being M_e , k_e , and F_e . The equivalent system is chosen so that the deflection, y , of the concentrated mass is the same as that for some significant point on the structure, such as the midspan of a beam. The constants of the equivalent system are evaluated on the basis of an *assumed* deformed shape of the actual structure resulting from the *static* application of the *dynamic* loads. It is convenient to introduce certain transformation factors to convert the real system into the equivalent system. The total load, mass, and resistance of the real structure are then multiplied by the corresponding transformation factors to obtain the parameters for the equivalent one-degree system. Details of this procedure can be found in Biggs (1964).

Single Degree of Freedom Model

When an explosion takes place, pressures are placed on the outside of a building structure, often resulting in permanent deflec-

tions which push the walls toward the inside of a building. When modeling, this inward movement of a wall system is simplified in order to more easily predict the behavior of a system. This simplified system is commonly referred to as a SDOF model. As discussed in Biggs (1964), with this model only one type of motion is possible; or in other words, the motion of the system at any instant can be defined by a single coordinate system.

The first step in dynamic modeling is to isolate the mass as a free body diagram [Fig. 3(c)], and then write an equation of motion by applying the concept of dynamic equilibrium [Eq. (1)]. For Fig. 3, the equation of motion is

$$M_e \ddot{y} + R - F_e(t) = 0 \quad (1)$$

where $M_e \ddot{y}$ =force of inertia; R =resistance of the spring force; during elastic response $R=k_e y$; $F_e(t)$ =applied external force as a function of time t ; which is a function of time; y =displacement; and $\ddot{y}$ =acceleration.

This differential equation can be solved to determine the variation of displacement with time once the specific parameters are defined. After the system is represented as an SDOF model and the loading function and initial conditions are defined, the numerical integration can then be performed. This is a procedure by which the differential equation of motion, Eq. (1), is solved step by step, starting at time zero when the displacement and velocity are known, and the displacement can be extrapolated from one time step to the next.

When modeling an ideal system, a SDOF model is developed, dynamic equilibrium is applied, and then an equation of motion can be determined. Before one can perform numerical integration, the loading function must be known as well as the initial conditions, which include the mass and the resistance of the system R . The resistance of a system is the internal force that tends to restore the system to its unloaded static position (Biggs 1964). As mentioned previously, the resistance of a linear elastic system is modeled using the spring constant k , which is the ratio of force to deflection. In a linear system, the spring constant is represented by a single number. In a more practical system, where the resistance of a system varies with deflection, the resistance function can be represented as a bilinear or trilinear function.

Consider the resistance function of Fig. 3(d). As the displacement increases from zero, the resistance also increases linearly with a slope of k_1 , the spring constant, until the elastic limit displacement y_{el} is reached. As the displacement continues to increase, the resistance remains constant at R_m , which is the plastic resistance, and has a slope of $k_2=0$. The resistance then increases at a slope of k_3 during the tension membrane region of behavior. The displacement will continue at this resistance of k_3 until the structure reaches its ductility limit and failure of the system occurs. If a maximum displacement occurs before the ductility limit is reached, the system is said to rebound, and will continue to rebound at a slope assumed to be the same as the slope in the elastic region of response. So, in predicting the dynamic behavior of steel-stud walls using a SDOF model, a resistance function must be defined. A valid static resistance function must be developed to predict cold-formed steel-stud wall behavior under blast loads and to improve design methods. This static resistance function is developed next.

Static Resistance Function

To be able to conduct dynamic modeling and arrive at engineering design tools for blast loads, it is necessary to first develop the static load-deflection response of the wall under uniform pres-

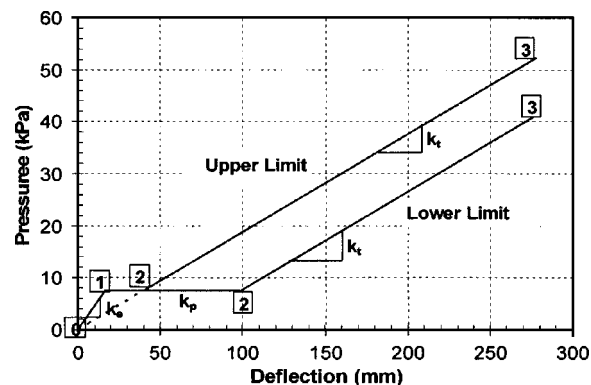


Fig. 4. Static resistance function for steel studs

sure: *Static resistance function*. The performance of the steel-stud wall depends, among other things, on the response of the individual stud components that make up the wall. The elastic-plastic response of the steel studs to failure is obtained theoretically and verified experimentally through static full-scale component and wall tests. The experimental setup described here is for blast-retrofit wall systems, in which the connections and sheathing are sufficiently designed to utilize the studs' full strength and ductility by ensuring failure to occur in the studs, as discussed in previous sections.

Analytical Model

An analytical static resistance function, based on stud size and material properties, has been developed and verified using a combination of many tests. These tests include vacuum chamber tests of wall systems and component tests of the wall systems using a bending tree as described later. The analytical model is a function of pressure and deflection. The elastic portion of the model is well known and easily predicted using the classic beam theory (Yu 2000) while the inelastic portion of the model is based on equilibrium. The elastic and inelastic region is joined together with a horizontal line in the postbuckling region. An upper and lower limit for the resistance function in the tension membrane region was developed to account for two different types of stud behaviors observed experimentally, as well as explained later.

Fig. 4 is a typical static resistance function for steel stud members that is primarily based on theory, but also incorporates some experimental observations into its definition. The static resistance function is defined by a number of points, 0, 1, 2, and 3, and by the slopes between each of the points, k_e , k_p , and k_t . The points are: 0—the origin; 1—the midspan deflection and pressure at yield-buckling; 2—tension membrane resistance begins to dominate behavior; and 3—stud rupture due to excessive elongation. The slopes are: k_e —slope of elastic response; k_p —slope during plastic softening, which is assumed flat; and k_t —slope of tension membrane region. The static resistance function in Fig. 4 can be divided into three regions: The linear elastic region (between points 0 and 1), the postbuckling softening region (between points 1 and 2), and the tension membrane region (between points 2 and 3).

From experimental testing, two major observations were made and implemented into the static resistance function. First, the softening region, between points 1 and 2, was observed when the stud forms a hinge. When this takes place with a steel stud, a plastic hinge forms and can be compared to an ordinary hinge with a constant amount of friction. Second, the amount of softening that takes place varies from test to test and is not well defined. There-

fore, an upper limit and a lower limit have been defined for the resistance function to account for the different amounts of softening that occurs.

Linear Elastic Region

To define the points of the static resistance function, both pressure and deflection must be computed at each transition. The region between points 0 and 1 is the linear elastic region. Point 1 is commonly referred to as “yield-buckling” and is important in determining stud behavior. The elastic strength of cold-formed steel stud sections can directly be correlated to the yield point (Yu 2000). The center deflection at yield for a typical steel-stud member in a wall system with an applied uniform pressure can be computed from

$$\Delta_y = \left(\frac{5SL^4}{384EI_{\text{eff}}} \right) p_y \quad (2)$$

where Δ_y =center deflection at yield; L =length of beam; E =Young’s modulus; S =spacing of steel studs in the wall; p_y =pressure at yield per unit area; and I_{eff} =effective strong axis moment of inertia for the steel stud (Yu 2000). Note, in Eq. (2), for a steel-stud wall system under lateral uniform pressure, the per-unit length loading on the member at yield is: $W_y = p_y S$.

The pressure at yield is computed from the yield stress based on flexure, and the maximum moment of a simply supported beam with a uniformly applied load. The pressure at yield, for a typical steel-stud section is:

$$p_y = \frac{8\sigma_y I_{\text{eff}}}{SL^2 \left(\frac{h}{2} \right)} \quad (3)$$

where h =depth of steel stud section; and σ_y =yield strength of the steel stud material.

The slope of the elastic region is the pressure at yield divided by the deflection at yield, which can be seen from Eq. (2) to be

$$k_e = \frac{384EI_{\text{eff}}}{5SL^4} \quad (4)$$

Postbuckling Softening Region

The “softening” region is the flat portion of the static resistance function, and is due to the formation of a hinge. Any structure will have a curved transition phase even when only one plastic hinge is necessary to develop the full plastic strength of the structure (Biggs 1964). The characteristics of this transition period depend on how many hinges are formed during softening. However, the amount of deflection that occurs during the softening region is not well defined. From experiments, it is observed that in some instances the studs experience a “well-defined” softening region after yield-buckling and before going into the tension membrane region. When this type of behavior occurs, the studs will typically deflect $L/40$ before tensile capacity is developed. In other instances, the studs go into tension membrane right after yield-buckling is achieved, and typically deflect $L/120$ before tensile capacity is developed. The combination of both types of responses has been observed in testing.

The different responses are believed to depend on the mode shape the beam experiences as it approaches buckling, which might be controlled by the load application. Therefore, the analytical model developed accounts for both behaviors by incorporating an “upper limit” response and a “lower limit” response. The upper limit response represents studs with little softening

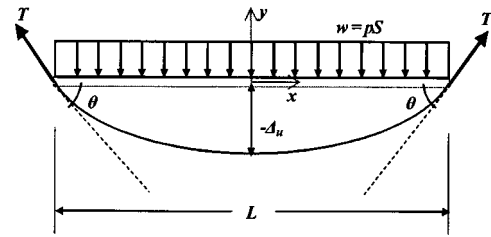


Fig. 5. Assumed parabolic deformed shape of a steel stud under uniform applied loading

before tension membrane dominates, while the lower limit response represents studs with more softening. Studs that form a well-defined three-hinge mechanism tend to soften more and are characterized by the lower limit response. The slope during plastic softening is assumed flat and, therefore

$$k_p = 0 \quad (5)$$

Tension Membrane Region

The plastic region of behavior for a steel stud is the region after softening occurs and the stud begins to develop more resistance, which is the region between points 2 and 3 (Fig. 4). This type of response is important to achieve since this is the region where the majority of energy is absorbed in a system. If the structural members in a system fail due to insufficient material properties, a bad connection failure, etc., its purpose to absorb energy is not achieved because of a reduction of capacity. In dynamic analysis, a stud’s ability to absorb energy is critical in surviving a blast load. Therefore, adequate predictions in the plastic tension membrane region are needed for design.

The slope of the tension membrane region can be determined from equilibrium and geometry of a stud under uniform applied loading. It is important to determine the point in the static resistance function at which the ultimate pressure p_u , which defines the failure, is reached. This is a crucial point since it directly affects the amount of energy the system will ultimately absorb.

Fig. 5 shows a stud under uniform applied loading, the forces that are present, and a general deformed shape. When tension membrane is dominant, the tension in a stud is equal to the stress in the stud at yield multiplied by the effective area of the stud ($T = \sigma_y A_e$). For the forces in the y direction and assuming a positive direction upward, equilibrium is as follows:

$$2T \sin \theta = wL$$

Substituting for T and assuming a small angle θ yields:

$$2\sigma_y A_e \theta = wL \quad (6)$$

Now substituting $w = pS$ and solving for the pressure at ultimate, Eq. (6) becomes

$$p_u = \frac{2\sigma_y A_e}{SL} \theta \quad (7)$$

To determine the pressure at ultimate for a steel stud in a wall system, it can be seen from Eq. (7) that a relationship between θ and the deformed shape of the stud needs to be established. To establish this relationship, a deformed shape has to be assumed and a relationship between θ and Δ has to be obtained. From a number of tests performed on steel studs, it has been observed that a stud most closely follows the shape of a parabola (Lane 2003). Therefore, assuming a parabolic shape function, a relation-

ship between θ and Δ_u can now be established to determine the ultimate pressure. An approximate equation for a parabola is used, Eq. (8), and the following boundary conditions are applied to the parabolic shape function

$$y = ax^2 + bx + c \quad (8)$$

Boundary conditions: At $x=0$: $y=-\Delta_u$ and $y'=0$; At $x=L/2$: $y=0$ and $y'=\theta$. Solving for a , b , and c and substituting into Eq. (8) gives

$$y(x) = \left(\frac{4}{L^2} \Delta_u \right) x^2 + \Delta_u \quad (9)$$

Hence, the following relationship is obtained at $x=L/2$:

$$\theta = y' \left(x = \frac{L}{2} \right) = \frac{4}{L} \Delta_u \quad (10)$$

Substituting Eq. (10) into Eq. (7), the pressure now becomes

$$p_u = \left(\frac{8\sigma_y A_e}{SL^2} \right) \Delta_u \quad (11a)$$

or in general

$$p = \left(\frac{8\sigma_y A_e}{SL^2} \right) \Delta \quad (11b)$$

To determine the ultimate deflection, failure is assumed to be localized over a reduced cross-sectional area of a stud. This is valid in determining the deformed length since it is known that plastic strain is almost invariably localized. The letter ξ , will be used to represent the localized cross-sectional area of a stud. From definition, ultimate strain is the change in length divided by the original length, or:

$$\Delta_\ell = \varepsilon_u \ell \quad (12)$$

where Δ_ℓ =change in length of the localized cross-sectional area; ε_u =strain at failure (%ductility); and ℓ =length of the localized cross-sectional area.

Keeping Eq. (12) in general terms, the length of the localized cross-sectional area is calculated as follows:

$$\ell = \xi L \quad (13)$$

where ξ =percent of the total length that is assumed to be the length of the localized cross-sectional area of a stud. Substituting Eq. (13) into Eq. (12), the general form for Δ_ℓ now becomes

$$\Delta_\ell = \varepsilon_u \xi L \quad (14)$$

The deformed length, L' , of the stud is equal to the original length of the stud plus the change in length over the localized cross-sectional area, and is as follows:

$$L' = L + \Delta_\ell \quad (15)$$

Substituting Eq. (14) into Eq. (15) and only considering half of the stud length, Eq. (15) now becomes:

$$\frac{L'}{2} = \frac{L}{2} + \left[2\varepsilon_u \xi \left(\frac{L}{2} \right) \right] \quad (16)$$

Eq. (16) gives a deformed length based on a certain amount of ductility of a stud. Relating ductility to ultimate deflection is very important in predicting an accurate static resistance function. This is true because of the variability of material properties in the manufacturing of cold-formed steel studs. Ductility in steel studs

can range anywhere from 5% elongation to 50% elongation, which significantly effects ultimate deflection.

To relate the ductility to the ultimate deflection, the definition of an arc length is used, which gives a relationship between the ultimate deflection and deformed length of a stud. The definition is as follows:

$$L_{\text{arc}} = \int_a^b \sqrt{1 + [y'(x)]^2} dx \quad (17)$$

where $L_{\text{arc}} = L'/2$ and L' =deformed length of stud. Taking the first derivative of Eq. (9) yields $y'(x) = 8\Delta_u/L^2 x$. Substituting this relationship into Eq. (17) and applying the appropriate integration limits gives

$$\frac{L'}{2} = \int_0^{L/2} \sqrt{1 + \left(\frac{8\Delta_u}{L^2} x \right)^2} dx \quad (18)$$

Integrating Eq. (18) yields the relationship between the deformed length of the stud and the ultimate deflection at failure:

$$\begin{aligned} \frac{L'}{2} = & \frac{2 \left(\frac{8^2 \Delta_u^2}{L^4} \right) \frac{L}{2} \sqrt{1 + \left(\frac{8^2 \Delta_u^2}{L^4} \right) \left(\frac{L}{2} \right)^2}}{4 \left(\frac{8^2 \Delta_u^2}{L^4} \right)} \\ & + \frac{1}{2} \left[\frac{1}{\sqrt{\frac{8^2 \Delta_u^2}{L^4}}} \sinh^{-1} \left(\frac{2 \left(\frac{8^2 \Delta_u^2}{L^4} \right) \frac{L}{2}}{\sqrt{4 \left(\frac{8^2 \Delta_u^2}{L^4} \right)}} \right) \right] \end{aligned} \quad (19)$$

Eqs. (19) and (16) can now be used to find the ultimate deflection. With the use of these two equations, the deformed length is both a function of ductility [Eq. (16)] and ultimate deflection [Eq. (19)]. The two equations can be set equal to each other and solved numerically to determine Δ_u , which represents the ultimate beam deflection, based on a given value for ξ . The value of ξ is directly affected by the ductility of the stud material. So now, the ultimate deflection can be determined for any stud if the ductility is known, and a value of ξ is selected. Once again, ξ represents the percent of the total length L that is assumed to be the localized cross-sectional area of a stud where most of the yielding occurs. Since the ultimate deflection is known, the pressure at ultimate can be determined from Eqs. (11a) and (11b).

Finally, the slope of the tension membrane region can be derived from Eq. (11b) for any value of p and Δ :

$$k_t = \frac{p}{\Delta} = \left(\frac{8\sigma_y A_e}{SL^2} \right) \quad (20)$$

Now the static resistance function can properly be predicted in the tension membrane region of behavior, for both the upper limit response and lower limit response. Each point on the resistance function has been defined as well as the slope for each of the regions. Again, properly defining the function throughout all regions of stud behavior is crucial for the dynamic modeling of a wall system. Now that the analytical static resistance function has been developed and is a function of stud size and material properties, the experimental data will be presented for further verification.

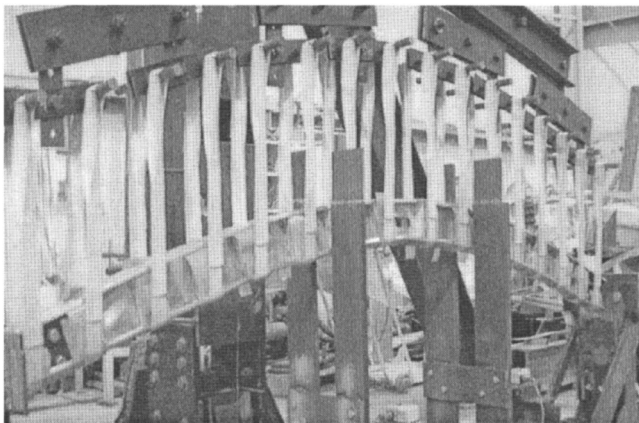


Fig. 6. Loading tree component test setup

Static Experimental Verification

The component tests are conducted by loading a stud or a stud pair via a *loading tree* that distributed the load from hydraulic actuators to 16 equally spaced points on the stud (Fig. 6). Note the wooden blocks used as bearing stiffeners at the loading points to prevent local buckling, and the vertical steel guides, which prevented torsional buckling of the section. The load and deflection response to failure is recorded, and equilibrium is then used to calculate an “equivalent” pressure per unit width. Another device that was used to obtain a static resistance function on a full-scale wall section is the static uniform resistance chamber, which is capable of applying a uniform load to a 3.66×3.05 m wall section using a vacuum pump (Fig. 7). A typical response of a steel-stud component test and a wall test are shown in Figs. 8 and 9. The studs shown in Fig. 8 did not experience a softening region, whereas in Fig. 9, the studs went into tension membrane after a well-defined softening region after the yield-buckling is achieved. The shape of buckling at the stud mid-span might control such responses. Therefore, the analytical predictions were developed to give a “low-end” and a “high-end” resistance function to completely represent both possible behaviors.

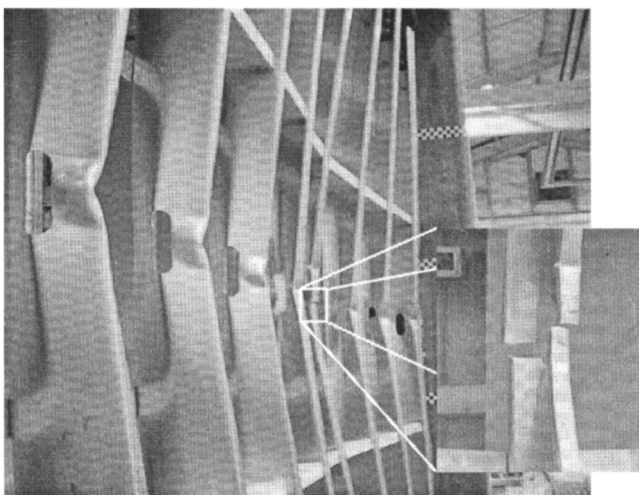


Fig. 7. Typical wall after test—Studs developed full capacity signified by the rupture of a stud at the middle section

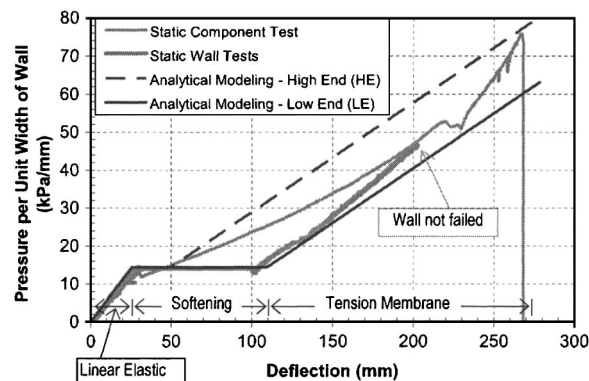


Fig. 8. Typical static resistance function of steel-stud wall and component systems—no softening

The analytical static resistance function was used along with the SDOF dynamic model to develop a wall analysis code (SSWAC 2003) for the dynamic modeling of steel-stud walls under blast loads. The code was used to design three stud wall systems and their response was verified in the field using live explosives simulating large vehicle bombs. In the following section, the results of the dynamic field testing are presented.

Full-Scale Dynamic Verification

The resistance function is used in a SDOF model to predict the behavior of the wall system when subjected to blast loads. Two of the walls tested were part of the blast response of exterior walls (BREW) research program. A third dynamic field test was part of the EWRP, a research program performed by the U.S. Army Engineer Research and Development Center (ERDC).

The purpose of the experiments was to validate the performance of the anchor systems in developing the full tensile capacity of the studs, to demonstrate the contribution of the mass to the wall response, and to compare the results of the experimental data to the preliminary model.

BREW-1 Dynamic Field Test

A full-scale blast experiment (BREW-1) was conducted at the Air Force Research Laboratory range at Tyndall Air Force Base

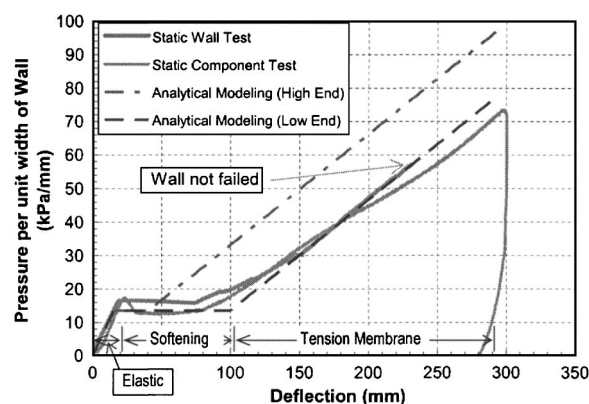


Fig. 9. Typical static resistance function of steel-stud wall and component systems—with softening

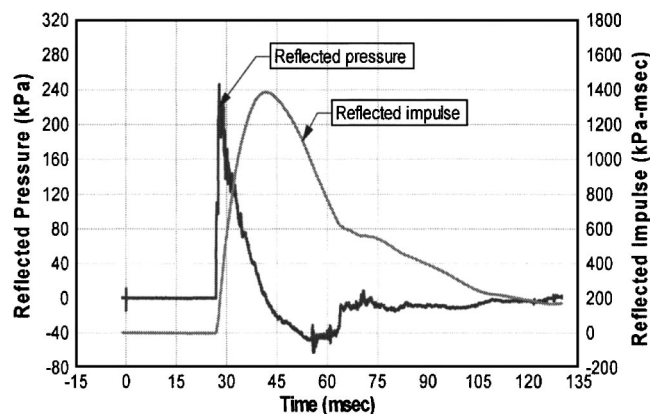


Fig. 10. Reflected pressure and impulse wave forms measured of wall surface for BREW-1

(AFB), Fla. Two steel-stud walls with blast-design connections were tested. The steel-stud walls consisted of 600S162-43 studs (AISI 2002), with a specified yield strength of 228 MPa, with single studs spaced 406 mm apart. The walls were approximately 3.66 m tall, and were attached at the bottom to a reinforced concrete slab using concrete anchors, and at the top to a steel plate (representing either a steel beam, or an embedded steel plate in concrete) using a steel angle welded to the plate, and a hole in the vertical leg of the angle to allow for a hinged connection.

One wall contained a brick façade consisting of 192 mm wide $\times$ 55.5 mm tall $\times$ 89 mm deep clay bricks with an area density of 1.46 kN/m². The façade of the other steel-stud wall consisted of a typical external insulation and finish system (EIFS) exterior with an area density of approximately 0.072 kN/m². The exterior side of the studs was sheathed with 1.37 mm (16 gauge) sheet steel, and the interior studs were sheathed with a product consisting of 6 mm gypsum board glued to 1 mm (20 gauge) steel sheets to provide a finished interior surface while preventing secondary fragmentation from the gypsum board Dinan et al. (2003).

The static resistance function of the walls is shown in Fig. 8, and the walls were subjected to a blast loading with reflected pressure and reflected impulse as shown in Fig. 10. The posttest

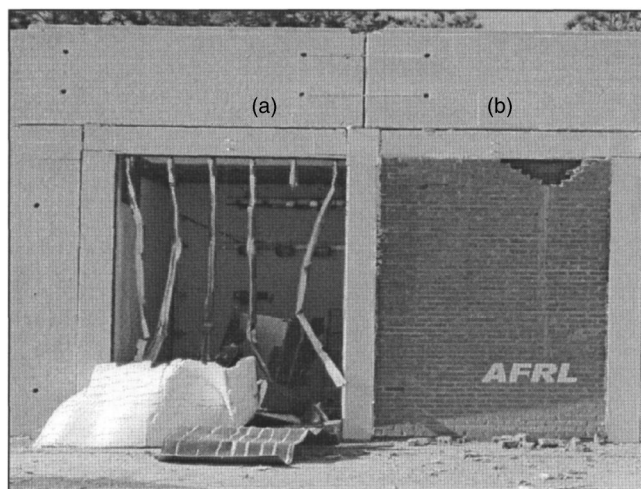


Fig. 11. Posttest exterior views of BREW-1: (a) external installation and finish system façade; and (b) brick façade

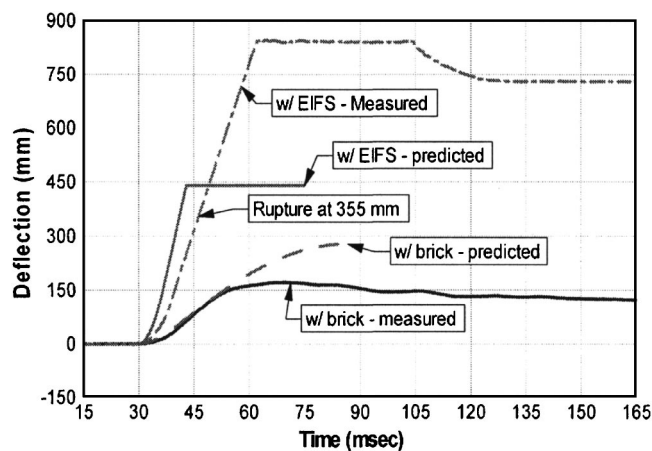


Fig. 12. Measured and predicted deflections at center of steel-stud walls for field tests BREW-1

photos shown in Fig. 11 and the deflection measurements shown in Fig. 12 demonstrate the dramatic difference in wall response resulting from the inertial effects of the mass of the wall. The deflection measurement at the center of the steel-stud wall with the brick façade indicated a peak inward deflection of 173 mm, with a residual deflection of 132 mm. The preliminary model predicted that the wall would survive and predicted a peak center deflection of approximately 279 mm.

The deflection at the center of the wall with the EIFS façade measured the gauge maximum of 813 mm inward deflection, and gave no indication of when the steel-stud wall failed. The steel studs that failed on the EIFS wall during testing were eased back into place during posttest forensics in order to estimate the plastic deflected shape when stud failure occurred. The average peak midspan plastic deflection that occurred at stud failure was approximately 355 mm. This compares well to the EIFS-steel-stud model that predicted stud failure at 457 mm of plastic plus elastic deflection. Details of this tests were reported by Salim et al. (2003).

EWRP-8 Dynamic Field Test

A full-scale blast experiment (EWRP-8) was conducted at Eglin AFB, Fla. The importance of mass provided by brick veneer was demonstrated in BREW-1 tests. Similar results demonstrating the effect of granite cladding on blast protection has been previously

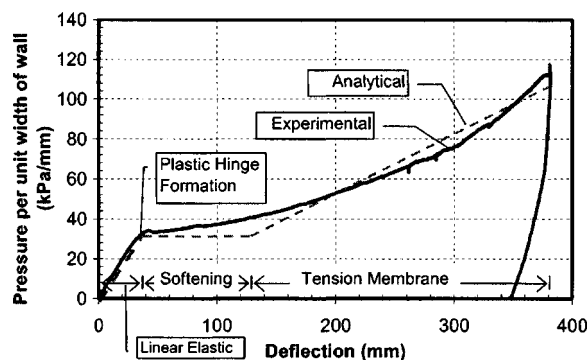


Fig. 13. Static resistance function per steel stud of wall system EWRP-8

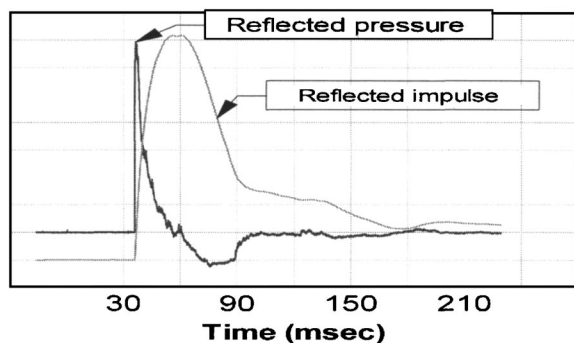


Fig. 14. Reflected pressure and impulse wave forms measured of wall surface for EWRP-8

reported by DiPaolo et al. (2003). Therefore, the third wall is expected to resist the blast loads with minimum mass contribution. This was achieved by using EIFS. Therefore, the ductility and strength of the steel studs alone are responsible for resisting the blast load and absorbing the energy from the explosion. This test presents the performance of an exterior curtain wall system with a window opening under blast loads. Details of this tests were given by Salim and Townsend (2004).

The steel-stud wall consisted of 12-gauge 600S162-97 studs (AISI 2002), with a specified yield strength of 228 MPa, with double studs spaced 406 mm apart. The wall was approximately 3.51 m tall, and was attached at the bottom to a reinforced concrete slab using concrete anchors, and at the top, the steel studs were anchored to the concrete wall at a height of 4.616 m. The façade on the steel stud wall consisted of an EIFS exterior with an area density of approximately 0.072 kN/m^2 . The exterior and interior sides of the wall were sheathed as in BREW-1 test.

The static resistance function for the wall is shown in Fig. 13. The wall was subjected to explosive effects simulating a large vehicle bomb with a wave form as shown in Fig. 14 for one of the instrumentation gauges on the wall (*Note: The numeric values on the vertical axis of Fig. 14 are not shown at the request of the sponsor, ERDC*). The posttest photos are shown in Fig. 15, and the deflection measurement is shown in Fig. 16. The deflection measurement at the center of the steel-stud wall indicated a peak inward deflection of 340 mm, with a residual deflection of 170 mm.



Fig. 15. EWRP-8 steel-stud wall posttest

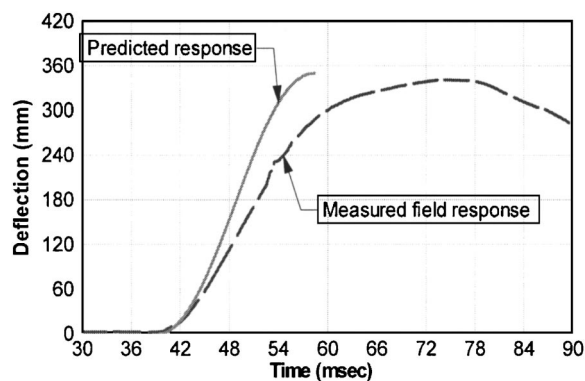


Fig. 16. Measured and predicted deflection at center of EWRP-8

The analytical model predicted that the wall would survive and predicted a peak center deflection of approximately 350 mm. The analytical model predicted a maximum response of approximately 380 mm before the steel studs would fail under excessive elongation. The experimental response was very close to the theoretical upper limit, and it was seen from the posttest photos that some failure in some studs has already started. Nevertheless, the wall survived the predetermined explosion threat.

Conclusion

Properly anchored steel-stud walls have proven to be an effective solution for construction of blast resistant walls for either new or retrofit construction. Some research has been performed to date to develop design methodologies for the required connection details, and to understand plastic postbuckling behavior and strain limits of the steel studs. The effectiveness of mass at reducing the response of the wall to blast loadings is dramatically demonstrated in the experiment and in the model. The theoretical resistance functions and preliminary design code provide a conservative prediction of the steel-stud wall dynamic response, allowing engineers to design a blast-resistant steel stud wall that will survive a given explosion threat. The analytical design model was verified experimentally using full-scale static and dynamic tests. The dynamic tests simulated large vehicle bombs, and the blast-resistant steel stud walls performed as predicted by the design methodology. The analytical model conservatively predicts the wall response. Note that the model assumes a uniform pressure over the walls and a one-way action, which does not represent the field response completely. In addition, the resistance of the external and internal sheathing was not accounted for. All of which contributed to the prediction model being slightly softer than the experiment.

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Notation

The following symbols are used in this paper:

- A_e = effective cross-sectional area of stud; gross less the area of holes;
- E = elastic modulus;
- F_e = equivalent single degree of freedom force;
- h = depth of steel stud cross-section;
- I_{eff} = effective moment of inertia of steel stud;
- k_e = equivalent single degree of freedom stiffness;
- k_p = equivalent single degree of freedom plastic stiffness;
- k_t = equivalent single degree of freedom tension membrane stiffness;
- L = length of studs in a wall system;
- L' = deformed length of stud under loading;
- L_{ARC} = arc length of elastic curve of beam;
- ℓ = length of the localized cross-sectional area;
- M_e = equivalent SDOF mass system;
- p = pressure per unit area of wall;
- p_u = ultimate pressure;
- p_y = pressure at the onset of yield-buckling of steel studs;
- R = equivalent resistance of SDOF system;
- S = stud spacing in a wall system;
- T = tension membrane force;
- t = time;
- w = equivalent uniform load on beam per unit length;
- x = distance along beam;
- $\ddot{y}$ = transverse acceleration;
- $y(x)$ = elastic curve of beam representing the transverse displacement;
- $y'(x)$ = slope of deformed shape at any location x along beam span
- Δ = midspan deflection of studs in a wall system;
- Δ_ℓ = change in length of the localized cross-sectional area;
- Δ_u = ultimate midspan deflection of studs in a wall system;
- ε_u = ultimate strain;
- θ = slope of elastic curve of beam at ends;

ξ = percent of stud length where excessive elongations are focused; and

σ_y = yield strength of stud cross-section.

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