RELIABILITY OF RESIDENTIAL BASEMENTS AS BLAST SHELTERS

A. Longinow* and J. Mohammadi**

INTRODUCTION

This paper describes an analysis method for predicting the probability of failure of a wood-framed basement when subjected to a static, uniformly distributed load. The analysis considers the primary failure modes of each framing member and determines the probability of failure for each mode acting alone. The failure probability of the system as a whole is then bounded. The upper bound is determined on the assumption that the failure modes are independent, while the lower bound is determined on the assumption that the failure modes are perfectly correlated. The analysis is described with reference to an example problem.

DESCRIPTION OF THE STRUCTURE

Plan and elevation views of the wood-framed basement are shown in Fig. 1 and Fig. 2 respectively. The basement is lined with concrete block walls on footings. The floor is a thin concrete slab. The original framing system consists of joists supported by basement walls and girders, which are in turn supported by five wood columns. The flooring, consisting of two layers of 1-in. thick boards is nailed to the joists (Ref. 1).

This basement is an improvised shelter against the effects of blast. To this end, the original framing is strengthened by incorporating a studwall at the center of each of the two joist spans. There is one stud column under each joist in each span. The windows into the basement are blocked off and the protruding basement walls are mounded up to the level of the flooring. Sizes of the framing members considered in the analysis are given next.

Joists: 1.625-in by 5.625-in with an average spacing of 24.12-in
Girders: 5.5-in by 6.75-in
Columns: 4.0-in by 8.0-in
Studwalls: Columns 2.0-in by 4.0-in with bracing at midheight

The material is Jack Pine.

STRUCTURAL ANALYSIS

The joists are continuous over the girders and the studwalls are simply-supported at the basement wall. Experimental results (Ref. 1) indicate that flooring nailed to joists does not result in full composite action between the joists and the flooring and therefore the joists are analyzed as being independent of the flooring.

Girder 1 is simply-supported on column 1 and column 3 and is continuous over column 2. Girder 2 is simply-cupported on column 3 and the basement wall and is continuous over columns 4 and 5. The columns are analyzed

* Associate Professor, Department of Civil Engineering, Illinois Institute of Technology, Chicago, Illinois 60616
** Assistant Professor, Department of Civil Engineering, Illinois Institute of Technology, Chicago, Illinois 60616

as pin-ended. The extent to which the girders and the studwalls provide flexible supports for the joists is neglected. The loading consists of pressure applied uniformly normal to the floor surface.

Computed stresses are compared to ultimate (incipient failure) stresses which were determined based on a load duration of 1 sec (Ref. 2). To this extent the results approximate the load carrying capacity of the structure when subjected to a uniformly distributed dynamic loading (Ref. 3). These ultimate stresses are given as follows (Ref. 2):

F_b = Rupture (Bending) Strength = 7,100 psi

F_c = Compression Strength Parallel to the Grain = 6,050 psi

F_v = Shear Strength Parallel to the Grain = 750 psi

E = Modulus of Elasticity = $1.35(10)^6$ psi

PROBABILISTIC ANALYSIS AND RESULTS

The framing system is analyzed when subjected to a series of loadings of increasing intensity. At each loading a probability of failure is estimated as being between two bounds. Upper bound failure probability $P(F^*)$, is determined on the assumption that conditions between different components are such that the failure modes are independent. Lower bound failure probability $P(F')$, is determined on the assumption of perfect correlation between components and is based on the highest failure probability of one failure mode occuring in some one component of the system. These bounds are defined as follows:

$$P(F^*) = 1 - \prod_{i-1}^{n} [1 - P(F_i)] \quad (1)$$

$$P(F') = \max[P(F_1), P(F_2), \ldots, P(F_n)] \quad (2)$$

where $P(F_i)$, $i = 1, n$ are individual failure mode failure probabilities occuring in the various components. In the analysis of joists and girders, failure modes considered were flexure and shear. In the case of columns and studwalls the only failure mode considered was buckling. For the purpose of illustration, the failure probability of a joist is calculated next.

FAILURE PROBABILITIES OF JOISTS

A joist can fail in flexure or in shear. If these two modes are independent of each other then the failure probability of the joist is

$$P(F_j) = 1 - [1 - P(F_1)][1 - P(F_2)] \quad (3)$$

On the other hand, if the two modes are perfectly correlated, the failure probability is

$$P(F_i) = \max[P(F_1), P(F_2)] \quad (4)$$

Expressions (3) and (4) bound the actual failure probability. In (3) and (4), $P(F_1)$ and $P(F_2)$ are failure probabilities due to flexure and shear respectively. They are computed as follows:

$$P(F_i) = 1 - \Phi(\frac{\ell n \bar{\theta}_i}{\Omega_{\theta_i}}) \quad (5)$$

where $\Phi(\;)$ = standard normal distribution

$\bar{\theta}_i$ = the mean safety factor in flexure or shear

Ω_{θ_i} = coefficient of variation of the mean safety factor in flexure or shear

$$\bar{\theta}_1 = \bar{N}_{g1}\bar{F}_b\bar{S}/\bar{M} \quad (6)$$

where $\bar{N}_{g1}$ = correction factor on the flexure formula

$\bar{F}_b$ = modulus of rupture

$\bar{S}$ = section modulus = $bh^2/6$, where b and h are the width and depth of the cross-section

$\bar{M}$ = maximum moment acting on the joist

$$\Omega_{\theta_1} = \sqrt{\Omega_{g1}^2 + \Omega_{F_b}^2 + \Omega_S^2 + \Omega_M^2} \quad (7)$$

where the parameters inside the radical are coefficients of variation of 1) correction factor on the flexure formula, 2) modulus of rupture, 3) section modulus, 4) maximum moment.

Assuming perfect correlation between "b" and "h", the coefficient of variation of "S" can be computed from

$$\Omega_S = \sqrt{\Omega_b^2 + 4\Omega_n^2 + 4\Omega_b\,\Omega_h} \quad (8)$$

where Ω_b and Ω_n are coefficients of variation of "b" and "h" respectively.

$$\bar{\theta}_2 = 2\bar{N}_{g2}\bar{F}_v\bar{A}/3\bar{V} \quad (9)$$

where $\bar{N}_{g2}$ = correction factor on the shear formula

$\bar{F}_v$ = shear strength parallel to the grain

$\bar{A}$ = cross-sectional area of joist = bh

$\bar{V}$ = maximum shear acting on the joist

$$\Omega_{\theta_2} = \sqrt{\Omega_{g2}^2 + \Omega_{Fv}^2 + \Omega_A^2 + \Omega_V^2} \quad (10)$$

where the parameters inside the radical are coefficients of variation of 1) correction factor on the shear formula, 2) shear strength, 3) cross-sectional area of joist, 4) maximum shear.

Assuming perfect correlation between "b" and "h", the coefficient of variation of "A" can be computed from

$$\Omega_A = \sqrt{\Omega_b^2 + \Omega_h^2 + 2\Omega_b\,\Omega_h} \tag{11}$$

The values of N_{g1} and N_{g2} and the coefficients of variation of p, b, h, N_{g1}, N_{g2}, F_b and F_v were estimated on the basis of available data and engineering judgment. Estimated and computed parameters used in computing $P(F_1)$ and $P(F_2)$ are listed in Table 1. From (5) and data in Table 1:

$$P(F_1) = 1 - \Phi\left[\frac{\ell n(3.79/\bar{p})}{0.354}\right] \tag{12}$$

$$P(F_2) = 1 - \Phi\left[\frac{\ell n(3.898/\bar{p})}{0.317}\right] \tag{13}$$

The failure probability of the joist system is represented in Fig. 3. The upper bound was determined using (3). The lower bound was determined from (4) and is the failure probability due to shear, $P(F_2)$.

Note, that when all joists are identical and subject to the same load distribution and intensity, then conditions between the joists are perfectly correlated. On this basis the failure probability of the joist system is presented by the failure probability of one joist.

The failure probability of the entire framework, considering the joist system, girders and columns, is represented in Fig. 4. The upper bound was determined on the basis of (1) and the lower bound on the basis of (2). The lower bound is the failure probability of the stud-wall located in the east span, see Figure 1.

CONCLUSIONS

The results of the analysis are an upper bound at least because the flexibilities of the girders, the columns and the studwalls are neglected when calculating the response of the joists.

Note that the bounds on the failure probability for the system are fairly close together (See Fig. 4). Therefore the upper bound can conservatively be taken as the failure probability for the system.

ACKNOWLEDGMENT

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Table 1. Parameter Values Used in Sample Problem

$M^* = 9759\bar{p}$ lb-in	$\Omega_M = 0.20$
$V^* = 891\bar{p}$ lb	$\Omega_V = 0.20$
$b = 1.625$-in	$\Omega_b = 0.07$
$h = 5.625$-in	$\Omega_h = 0.07$
$N_{g1} = 0.95$	$\Omega_{g1} = 0.03$
$N_{g2} = 0.95$	$\Omega_{g2} = 0.03$
$S^{**} = 5.585(\text{in})^3$	$\Omega_S = 0.21$
$A = 7.313(\text{in})^2$	$\Omega_A = 0.14$
$F_b = 7100$ psi	$\Omega_{F_b} = 0.20$
$F_v = 750$ psi	$\Omega_{F_v} = 0.20$
$\theta_1 = 3.790/p$	$\Omega_{\theta_1} = 0.354$
$\theta_2 = 3.898/p$	$\Omega_{\theta_2} = 0.317$

* The joist is supporting a uniformly distributed line load equal to $16\bar{p}$ per inch, where $\bar{p}$ is in psi.

** The section modulus was calculated using a depth equal to 0.8h to account for possible notches or knots.

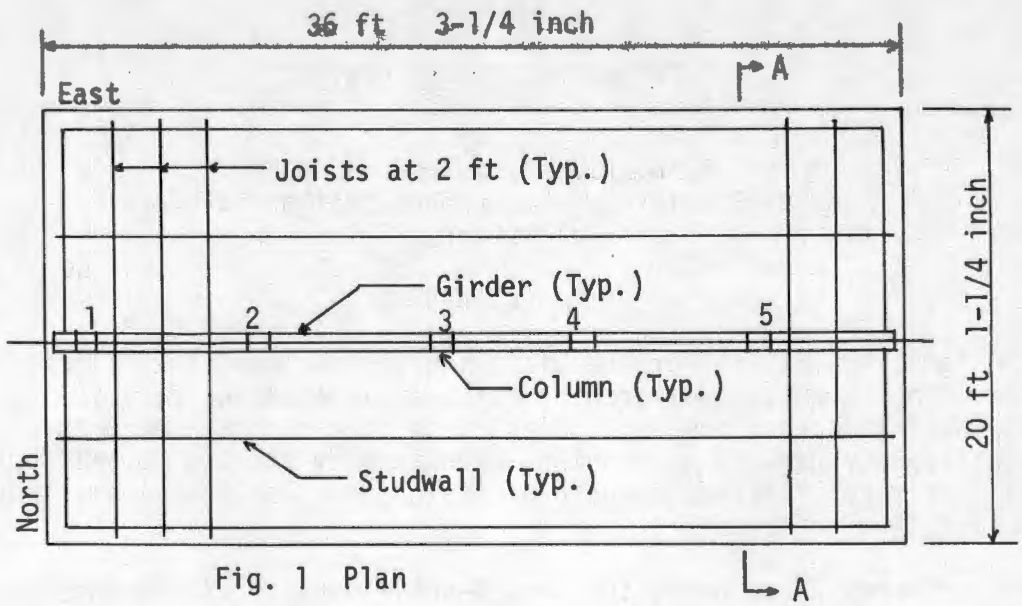

Fig. 1 Plan

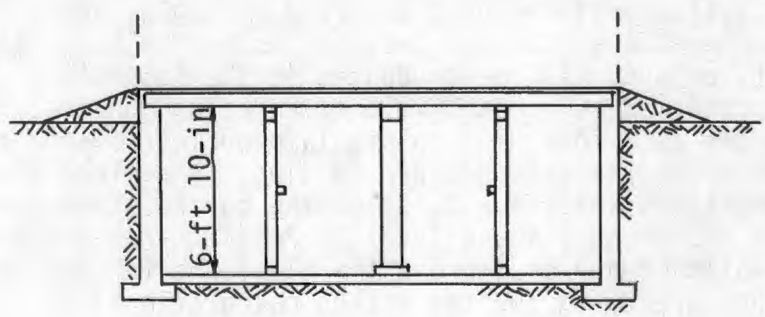

Fig. 2 Section A-A, Elevation

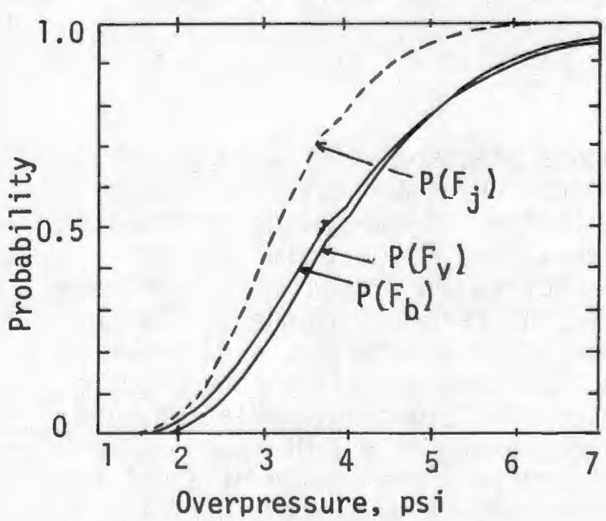

Fig. 3. Probability of Joist Failure

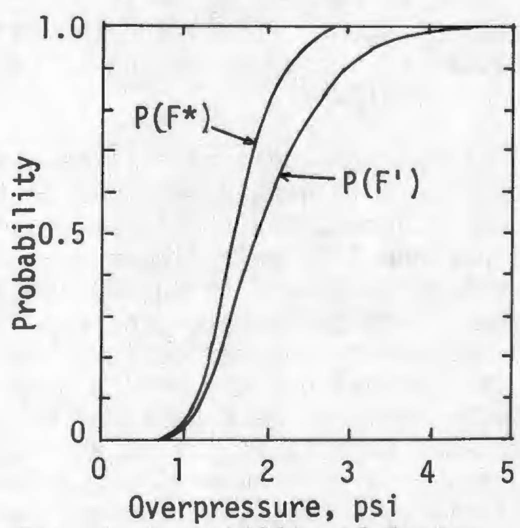

Fig. 4 Probability of System Failure