DETAILING STRUCTURAL CONCRETE FOR BLAST CONTAINMENT

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INTRODUCTION

Reinforced concrete structures must often be designed to resist short duration, high amplitude, dynamic loads. These loads may arise from a wide range of sources, including industrial or military explosions, vehicle impact, dropped objects, or wave impact on marine structures. The recent terrorist actions against public buildings in London, New York City, Buenos Aires and Oklahoma City caused various combinations of structural damage and casualties, highlighting that designers may need also to consider loads from deliberate attack. All these realistic cases illustrate the great need for technical information to be used for enhancing structural protection. Unfortunately, current civilian design procedures usually do not explicitly address these issues. However, considerable information on this subject can be found in several references (1-8).

There has been a strong international research activity over the last five decades in the general area of short duration dynamic effects on structures. Initially, such work was mostly empirical; however, in recent years, more fundamental research has been undertaken. These studies have shown that short duration, high magnitude loading conditions and the associated inertial effects have a major influence on the response. These loads are typically applied to structures at rates approximately 1000 times faster than earthquake-induced loads, and the frequencies of the structural response are much higher than those induced by conventional loads. Further, short duration dynamic loads often exhibit strong spatial and time variations, resulting in sharp stress gradients in the structure. High strain rates also affect the strength and ductility of concrete and steel, the bond stress-slip relationship for reinforcement, the failure modes, and the fracture/failure toughness of the structure.

The definition of the loading function can be a difficult task for any type of dynamic load, often involving many assumptions and strongly random variables. This is particularly true for blast and impact loadings. The first step is to determine the potential source of the load; while industry guides provide reasonable estimates for chemical processing facilities, the determination of loads due to deliberate attack is often more speculative. After a designer has selected the location and energy of a potential explosion, the next step is to predict the blast loads. These design loads typically take the form of pressure versus time curves or, else, as values for the peak pressure and the total impulse.

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Analysis methods commonly used in blast resistant design can be divided into three categories: single-degree-of-freedom (SDOF) methods, multi-degree-of-freedom (MDOF) methods, and general purpose numerical methods. When simple models are used to represent a complex structure, the coupling that occurs between members in the structure is often ignored. Typical simple SDOF methods for the design of structures to resist dynamic loads are contained in manuals (1) and in books (9). Other more advanced SDOF codes can address structural coupling, strain rate effects, soil-structure interaction, tension stiffening, and interaction between flexure, shear, and axial forces (10-12). The MDOF methods are an intermediate class of special purpose solutions that are based on several degrees-of-freedom, usually two. The general-purpose numerical methods are generally sophisticated computational approaches (Finite Elements, Finite Differences, Boundary Elements, and various combinations of these), which are used to solve the response of a complex structural system by using many degrees of freedom, and the computational expenses can be high if material nonlinearities (i.e., plasticity, damage, fracture, rate effects) are included and a dynamic solution is obtained.

Reinforced concrete structures cannot develop their full capacity if their reinforcement details are inadequately designed. In connections, for example, the joint size is often limited by the size of the elements framing into it. This restriction along with poor reinforcement detailing may create connections without sufficient capacity to develop the required strength of adjoining elements. Knee-joints are often the most difficult to design when continuity between the beam and column is required. The work by Nilsson (13) provided extensive insight into the behavior of joints under static loads, and on the relationships between performance and internal details. He showed that even slight changes in a connection detail had significant effects on the strength and behavior of the joint. Park and Paulay (14) proposed the diagonal strut and the truss mechanisms to describe a joint's internal resistance to the applied loads. The diagonal compression strut is obtained from the resultant of the vertical and horizontal compression stresses and shear stresses. When yielding in the flexural reinforcement occurs, the shear forces in the adjoining members are transferred to the joint core through the concrete compression zones in the beams and columns. The truss mechanism is formed with uniformly distributed diagonal compression, tensile stresses in the vertical and horizontal reinforcement and the bond stresses acting along the beam and column exterior bars. These two mechanisms can transmit shear forces from one face of a joint to the other, and their contributions were assumed to be additive. More recently and Paulay and Priestly (15) used equilibrium criteria to address the joint reinforcement that is necessary to sustain a diagonal compression field in a joint core. Their approach addressed the strength requirements for beam-column joints under severe, quasi-static, loads. They showed how the external loads are resisted by internal forces, and how these forces vary as the damage in the joint accumulated. The design procedures for determining joint size and the longitudinal and transverse reinforcement requirements in beam-column joints for monolithic reinforced concrete structures were established not too long ago (16) were revised recently (17). This brief discussion shows that until recently, recommendations for the design of connections were empirical, however, developments in the application of advanced truss analogies also contributed to the development of a rational approach for the design of connections.

Designing reinforced concrete connections is much more difficult for explosive loads. Explosive tests on elements without shear ties in (18) showed a definite diagonal shear failure located immediately near the joint region. The magnitudes, intensities and distribution (both in time and space) of such inertia forces depend on the dynamic behavior of the structure. These parameters can be evaluated either experimentally or numerically, and both approaches pose serious difficulties due to the severe loading environments and their effects on materials and structural components. Krauthammer and DeSutter (19) included the evaluation of connection details under the effects of localized explosions by simulating numerically physical tests, and preliminary joint details were recommended for implementation. Similar consideration of structural concrete knee-joints under internal explosions was presented in (20 and 21). Those two preliminary numerical studies were based on using finite element codes for two- or three-dimensional analyses, however, they did not enable precise information on a wide range of joint parameters.

The present investigation was part of a broader study to evaluate numerically the proposed knee-joint detail and to provide conclusions and recommendations based on the findings (21-23). It can be used to illustrate the importance of attention to structural concrete detailing under these severe loading conditions.

PARAMETRIC STUDY DESCRIPTION

The structure under consideration was a reinforced concrete blast containment facility with the dimensions of 46.5 feet long by 31 feet wide and 19.5 feet high. Wall thicknesses were 3 feet for the side walls, 3.25 feet for the front and back walls and 4.5 feet for the roof. The structure was considered both as a beam-column knee-joint (20,22), and in its three-dimensional configuration (21,23).

Also, for the two-dimensional analysis, a hybrid (finite element - finite difference) computer code was developed in (22). The mesh represented a horizontal slice of back wall - to - side wall connection subjected to an internal blast load. The joint region was divided into 18x19 segments, while the finite element domain included 18x38x2 elements for the back wall, 18x60x2 elements for the side wall, and 405 beam elements depicting discrete steel reinforcing bars. The far ends of both structural members (away from the connection) were at the walls' center-lines, at which symmetry boundary conditions were assumed (i.e., only translations perpendicular to the axes of each leg were permitted at these locations). DYNA3D (24) was used for the three-dimensional analysis of the various cases, as described in (21). Taking advantage of the planes of symmetry along the slabs' center lines, the finite element model represented one eighth of the total structure. To understand better and document the contributions of each reinforcement component, nine different cases were considered, using incrementally varying details of the reinforcement.

The following material properties were used for all computations: Concrete: f'_c (uniaxial compressive strength) of 4,000 psi, and ν (Poisson's ratio) of 0.16. Steel: E (elastic modulus) of $29X10^6$ psi, ν (Poisson's ratio) of 0.33, f_y (yield strength) of $6X10^4$ psi, and E_t (tangent modulus) of $5.1X10^5$ psi. The design load simulated a 300-lbs TNT contained explosion with prescribed venting. The loading function was bilinear, consisting of a shock overpressure and a gas overpressure. For each case, the roof and sidewall were subjected to a blast load with a peak pressure of 1115 psi, while the backwall loading had a peak pressure of 2470 psi.

RESULTS AND DISCUSSION

The main objective of generating and analyzing the different cases was systematically to determine the overall effects of each type of reinforcement as it was added to the structure, and the relationships with varying its position. It was found in (22) that the diagonal bar is very important, and its location can significantly affect the connection behavior. Also, it was noted that the diagonal bars must be well anchored to resist the induced tension. Further, it was found that the reinforcement details proposed for connections in (1) should be followed carefully to insure that the connection can resist the required loads. However, the plastic hinge locations will be outside the connection regions (just beyond the ends of the diagonal bars), that those areas must be well detailed to resist the combined effects of tension, shear and localized rotations.

The support rotations of Type I containment structures are restricted to 2 degrees (1). The addition of the diagonal bars resulted in a significant strengthening of the structure, yet further examination of the deformed structure revealed the formation of plastic hinges near the ends of the diagonal bars. It is noticed that the plastic hinge mechanism was characterized by very large local rotations. The rotations resulting from the formation of the plastic hinges are important because of the high stresses and possible failure of both concrete and steel associated with such mechanisms. The local rotation was defined by the deformation of the plastic hinge region, while the global joint rotation was defined by the peak deflection at the center of the wall over half the wall length. It is important to note that the values of the local θ_1 and global θ_2 rotations, as presented in Table 1, for all of the slab-to-slab connections exceeded the prescribed maximum 2 degrees support rotations.

Following the above analyses, it was decided to investigate further the effects of the diagonal and radial reinforcements on the behavior using the hybrid computer code for the two-dimensional knee-joint model (22), and the findings are presented next.

Table 1 Local and Global Rotations

Connection	**Side**	θ_1 **(degrees)**	θ_2 **(degrees)**
Backwall/Roof	Roof	25.9	11.9
	Backwall	15.5	6.9
Sidewall/Roof	Sidewall	17.7	6.8
	Roof	32.3	18.4
Backwall/Side wall	Sidewall	12.7	2.9
	Backwall	12.7	4.8

Effects of Diagonal Reinforcement

Twenty cases with different diagonal bar cross-sections and locations were investigated. Two parameters were introduced: d was defined as the distance measured from the outer corner of the connection to the diagonal bar location, and d' was the diagonal length of the connection that was 52.35 inches. The locations of the diagonal bar varied around the point A (i.e., five cases of different value of d: 49.52, 52.35, 55.18 58.01 and 60.84 inches, respectively). These were derived at distances between $2\sqrt{2}$ and $8\sqrt{2}$ inches from the inner corner of the joint). The amount of steel in the diagonal bar was incremented by multiples of two from 0.25 to 2 times the area of a Number 11 bar (i.e., 0.25, 0.5, 1, and 2 times the area of a Number 11 bar, which is 1.56 in^2.).

The maximum concrete stresses inside the connection region for the different cross-sections of the diagonal bars in the X and Y directions were plotted as a function of the parameter d by least square polynomial curve fitting to the numerical results. It was observed that after d reaches about 61 inches the different reinforcement ratios of the diagonal bars did not influence the maximum concrete stresses in the joint. The smallest values of f_{max} seem to occur when d was about 60 inches (which is probably the optimal design location for placing diagonal bars). It was also noticed that the cases with two No. 11 diagonal bars at 49.52 and 55.18 inches, respectively, had the maximum concrete stress, as compared with those of the other three reinforcement ratios. Whereas at 52.35 and 58.01 inches, respectively, the case with two No. 11 diagonal bars showed the minimum concrete stress.

Effects of Radial Reinforcement

The purpose of radial reinforcement across the joint was to prevent concrete cracks parallel to the diagonal bar inside the connection. Three cases (each case had a different bar size for the radial bars, e.g., No. 4's, No.7's and No.11's) each with five pairs of radial bars were studied. For these three cases, the diagonal bar was one No. 11 bar located at d=58.01 inches (i.e., from the outer corner of the connection). In all three cases, the maximum stress occurred in the central radial bar, and they are shown in Table 2. As the volume of radial bars increased, the maximum stresses in the diagonal reinforcement also increased, and they occurred later. Since the radial bars were tied to the diagonal reinforcing bars across the inner corner, the higher forces in the radial bars induced higher stresses also in the diagonal bars. By inspecting the negative stress distribution in the joint, it was possible to detect the formation of a diagonal compression field parallel to the diagonal bar. The compressive forces induced by the two adjoining members on the joint formed a diagonal compressive strut. The diagonal tensile forces at the inside corner were resisted by the diagonal reinforcement and the radial bars counteract the diagonal tension across the connection. However, unlike in the static cases, the distribution of tensile and compressive stresses varied with time.

CONCLUSIONS

The findings from this investigation showed that the location of the diagonal bar across the inner corner has a significant effect on the joint's strength. The relationships between the maximum stress and the location of the diagonal bar, and/or its cross-sectional area, could be examined to produce

Table 2 Reinforcement Stresses

Radial Bar Size	No. 4	No. 7	No. 11
Max. Stress in Diagonal Bar (ksi)	80.5	82.8	85.7
Time (ms)	55.5	57.9	62.6
Max. Stress in Diagonal Bar (ksi)	Fracture	112.6 Possible Fracture	25.5
Time (ms)	58.2	92.3	75.0

design recommendations that can insure a desired level of performance. It was observed that the radial reinforcing bars affect the tensile stress in the diagonal bar at the inner corner of the connection. Furthermore, the diagonal compressive strut to resist the applied loads can be mobilized effectively by proper combination of radial and diagonal bars.

The strengthening of the joint regions by the diagonal bars was characterized by the formation of plastic hinge regions near the ends of the diagonal bars. The relocation of the largest rotations from the support faces to the plastic hinge regions showed a shift of maximum moment and shear along the slabs. Examination of the stresses in the flexural bars along the planes of symmetry revealed that yielding and the maximum stresses in the interior flexural and tension bars were located in the plastic hinge regions. Besides the damage to the flexural reinforcement, maximum shear stresses in the concrete and large tensile shock wave stresses also occurred in the plastic hinge regions. These concrete and steel stress patterns, besides the presence of direct in-plane tension in the slab (due to the expansion of the walls and roof caused by the interior explosion), showed that excessive damage could occur at these regions if not properly designed. Current design procedures (1) do not address the issue of plastic hinge regions, nor do they anticipate the location of maximum negative moment (negative yield line location) to occur near the ends of the diagonal bars. Furthermore, the computation of structural capacity, in current design procedures, is based on the assumption of yield line formation at the supports. Clearly, the shift in hinge location must be included to derive a more realistic structural capacity. Also, the classical yield line theory treats yield lines that have a zero thickness. In reality, the plastic hinge regions have finite thicknesses (roughly the member's thickness), and this issue should be addressed in design procedures.

As noted previously, the local rotations of all the connections exceeded 12 degrees and the global rotations exceeded 2 degrees (Table 1). The sidewall/roof connection, which had the largest local and global rotations, also had the largest diagonal and longitudinal stresses. This indicated that the magnitudes of the local and global rotations provide a good indication of the extent of damage to the slab. This emphasizes the need to address both local and global rotations in design approaches.

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