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MULTI-OBJECTIVE OPTIMIZATION OF REINFORCED CONCRETE RESERVOIRS SUBJECTED TO DYNAMIC RESPONSE AND BLAST ANALYSIS

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ABSTRACT

Iconic and public buildings have become a universal target of bomb attacks from terrorists. Most of these buildings have been or are built without consideration for their vulnerability to such events. Planning and building control authorities have begun to recognize the risks of these events and have introduced provisions in planning guidelines for mitigation of such impact. This paper is a study of the impact of near field explosions on the structural framing system and key elements such as columns and describes the component material response. This information can be used in planning strategies to mitigate potential catastrophic and progressive collapse of the structure. Reinforced concrete framed buildings have been selected for this study. A two stage finite element modeling (FEM) and analytical technique has been used to interrogate the structural framing system and components for global stability and local residual strength capacity in the linear elastic and non-linear plastic response regimes. The first stage involved linear time history analysis carried out using SAP 2000 to verify the response of the complete framing system and its ability to restore global frame stability and to enable iterative interrogations. An explicit rigorous analysis accounting for strain rate effects of the reinforced concrete elements was carried out in stage two using LS DYNA code to investigate the non-linear response of vulnerable elements identified in the first stage. The damage mechanisms and the extent of damage have been studied using principal stress plots along with plastic strain diagrams and used to assess the residual strength capacity of key elements that can cause catastrophic failure of large sections of the building and propagate progressive collapse. Numerical analysis is based on techniques that have been established in previous research work and the models have been calibrated with similar work by others. The method used in this research work can be used for assessing vulnerability, damage and residual strength capacity of building frames and component elements subjected to near field blast events.

KEYWORDS: Artificial Bee Colony, Durability, Optimization, Reinforced Concrete Reservoirs

INTRODUCTION

In the past, designing the concrete structures has been done based on elastic theory that control of maximum tension under loads is foundation of this theory. The RCR liquid holding structures designing has been done to the authorized amounts based on elastic relations and with limited material stresses, despite the low tension the fracture has not left much of the structure. For this reason, in designing structures, thick sections of concrete with a large amount of steel are used. In that time, analyzing of possibility of thermal cracking and cracking due to concrete drop was not performed based on acceptable bases. Also only a nominal amount of steel was mentioned in the regulations. In recent years, limit state which was on more rational basis was introduced to determine the safety factors. In this method, in designing structure, loads coefficients have been used with the ultimate strength of materials. In order to calculate the flexural crack width and compare it with maximum allowable amount, developed analytical methods are used. In addition to that, methods of calculating the effects of thermal strains and strain caused by the loss of drying concrete have been considered. By the mentioned advances the limit state method has been used to design the blue liquid holding structures. Through designing method of limit states gives conceivability of identification and evaluation of possible failure modes of structures to the designers as premature rupture of structures can be avoidedwhich during times these problem change and research by Ziari and Kianoush (2009) as investigation of flexural cracking and leakage in RCR.

Bomb explosions against iconic and public buildings have become hazardous due to widespread terrorist activities in various parts of the world. Loss of lives and millions of dollars of property damage are the consequence following a successfully targeted bomb attack. While initial casualties are due to direct over pressure released by the explosion, falling of structural elements may extensively increase the total figure. Most buildings which are likely to be the target of a terrorist bomb have been constructed without considering their vulnerability, or their potential to mitigate the impact. However, planning and building control authorities are currently identifying the risks associated with such events in the present environment of global terrorism. It is therefore important to carry out vulnerability and damage assessment of buildings subjected to blast loads to provide mitigation strategies. This paper discusses an evaluation of the response of a reinforced concrete (RC) framed building to a near field blast event.

Research and investigations have been conducted for RC structures to study their vulnerability when subjected to blast loads in order to implement retrofit measures for preventing damage to the structure and its components. Some of these have examined the progressive collapse mechanisms of the building frames and recommended retrofit methods to avoid catastrophic failure of the entire structure while the rest assessed the individual structural elements.

However, limited research has focused on the studies to identify the effects of blast load on the RC framed buildings, but they have focused on either the stability of a sub frame or the response and the damage to a single structural component. An extensive investigation on the progressive collapse of a steel framed multi story building has been carried out and a similar study is needed for RC framed buildings too. This paper presents a two phase finite element analysis procedure to interrogate the structural framing system and components for global stability followed by a rigorous analysis of key structural components for damage evaluation in the linear elastic and non-linear plastic response regimes respectively. In the first phase, SAP2000 is used to carry out the global linear elastic analysis of the entire building frame under blast loading. The outcome of this analysis will provide information on the damage sequence of the frame system, the critical regions which require rigorous analysis and the capacity of the framing system to restore global frame stability. LS DYNA will be used in the second phase to carry out a rigorous non-linear elasto-plastic analysis of a sub frame embodying the critical members identified in the global analysis. The outcome of the second phase will provide information on the damage extent of the critical members and their post blast capacities. The combined assessment can be used in evaluating the global stability and the structural integrity of the building frames in order to protect against catastrophic collapse.

Reinforced concrete containment of nuclear power plant must be constructed and operated in order to protect the population and environment against an uncontrolled release of radioactivity in the event of severe internal or external accident occurrence, such as large fires, earthquakes, jet aircraft impact, that might be expected during the plant lifetime.

Studies are reported in literature on the impact of an aircraft on the outer reinforced concrete nuclear containment shells, but the scarce on the effect of explosions in the reinforced concrete containment shells until Fukushima nuclear event March 2011.

The present study has therefore been directed to study the effect on internal explosions on a typical reinforced concrete containment structure. Prediction of response of concrete structures for blast loading requires three-dimensional structural idealizations, true modeling of the material non-linear and precise modeling of the blast phenomenon. Different aspects of non-linearity in concrete, such as pressure sensitivity in three-dimensional loading situations, mesh quality sensitivity in dynamic loading situations and failure of concrete in tension have been included in developing finite element software and different parametric analysis has been performed.

Numerical simulation of the blast loading parameters for a specified scale distance for reinforced concrete containment is very significant in precise determination of the response. Most parameters for modeling the blast shock waves interactions as available for different parameters have been expressively stated. The dynamic response and damage mechanism of the reinforced concrete containment are investigated subjected to internal blast loading at varying scale distances. The effect of explosion inside of containment has been presented in terms of the extent of cracking in the concrete, stress in steel bar and concrete after yielding and deflections.

FUNDAMENTAL THEORIES

Basic Details

In water supply construction, to coordinate production and water use and to regulate the pressure in water supply network water reservoirs are used. Dewatering construction and water refinement are designed based on the maximum discharge utilization daily to weekly. Considering the economic aspects of the design and exploitation construction should be in a manner that could provide the required water in an identical process during the day defined by Chen and Kianoush (2009). Based on Chau and Lee (1991) it is clear that water consumption in the city could not be function of this identical process. Water consumption at different hours depending on conditions may vary up to several times daily. To reduce or eliminate the impact of these fluctuations and create coordination between consumption and production, reservoirs coordinator or modulator construction from the perspective of economic and operational is justified. Since research from Thevendran and Thambiratnam (1987) which work on optimal shapes of cylindrical concrete water tanks, these concrete reservoirs are built anywhere in the water supply construction and be hold safe from hourly water consumption drastic changes. If the conditions require, appropriate location for this repository is in water distribution or behind it. In this position, transmission lines and dewatering construction and refinement are safe from consumption time changes and costs of the design utilization are appropriate from an economic perspective.

In this study, a concrete storage tank with a 20,000 m^3 water network required has been selected regarding structural elements assimilation in the buried rectangular reinforced concrete reservoirs s. In this stage, the thickness of the walls, foundation and roof have been selected to achieve the bonds marginal ambit regarding repositories designing various records to select the optimization space in the appropriate space. In the next stage with finite element method and by computational analysis help, amplitude of sidewalls of the tank shell has been selected regarding lateral and intermediate positions supposing the effect of waterpower loading of inside the tank on the double reciprocal In the case of full water. Among the proposed elements, the range of elements that have the maximum tension have been chosen at first to calculate the optimum thickness of the shell wall and amount of optimal used bar, then in the next stage these investigated parameters are evaluated based on the confirmatory cover.

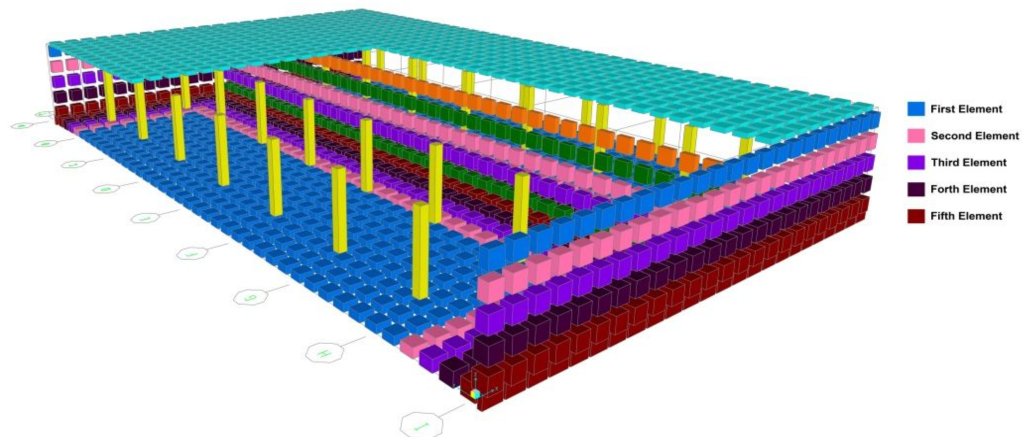


Figure 1: Modeling of Reinforced Concrete Reservoirs

Assumptions of Modeling

In the present study, some assumptions have been chosen based on accomplished modeling regarding the types of structural reinforced concrete reservoirs. The reservoir structure is half-buried which 20,000 m^3 of water tank is directed. The maximum water balance in the reservoir is equal to +4.70 m toward bed balance. Dimensions of the reservoir structure balances in the bed are equal to 86.40 m wide, 52.40 m length, and in height +5.35 m equal to 87.80 m wide, and 53.80 m length. In order to increase confidence and reduce tension levels at the foot of the walls, a heel 40 cm wide is intended in the bed. Reinforced concrete reservoirs structure is included the surrounding concrete walls by height 5.10 m which 40 cm embankment has been done on its roof. This structure has been considered half buried in the ground and there is no possibility of movement of vehicles on the roof of the tank regarding to protection of the cargo.

Concrete materials used in the tank are considered with regard to required resistance and also desired structural performance with a minimum 28 days compressive strength of its cubic sample equal to 350 kg/cm^2 and minimum used cement equal to 400 kg/cm^2. In preparation using concrete, the maximum water cement ratio is presumed 0.45 and commixture design will be available to administrative operation in the construction of structures regarding existing conditions. Type of using bar in the different parts of the concrete structure (foundations, walls, ceiling and columns) is deemed of ribbed AIII and with at least flowing 4000 to 4200 kg/cm^2 regarding required persistence and a good charisma creation between steel and concrete materials and simultaneous performance .

Soil on the site has been considered type II. Soil bearing capacity and specific weight of the soil in the site is supposed equal to 0.60 kg/cm^2 and 2.20 (ton/cm^3) respectively. Groundwater level in the construction site is considered below the bed balance and thereupon static and dynamic load side water resulting from earthquake around the structural walls is not defined.

Location of the operation is selected in a city with a high ratio of earthquake risk which the values of earthquake power and dynamic loads of water and soil on the structure can be calculated. In calculating the coefficient of earthquake, structural site, and arena or a high risk earthquake and maximum acceleration of earthquake equal to and regarding that the structure is related to water supply installation, the structure is considered with importance coefficient 1.4 and very important. Also behavior coefficient equal 3 is considered in order to the lateral seismic loads application on the reinforced concrete reservoirs structure and dynamic pressures on the grounds plasticity and depreciation feature of low energy of these structures. Under the terms of regulations in order to application of the terms of the elements cracking and effects of resistance reduction and cross section of each of them, effective coefficient of cracking in the space of inertial member has been considered 0.70 around axes y and z for the columns. Also effects of reduced stiffness due to fraction are assumed 0.70 in the walls with reduction coefficient, 0.25 in the ceiling with reduction coefficient and rigidity of rigid end-users in the arena at connections is assumed equal to 0.50 estimated.

Elements of RCR Structure

Thick shell elements for surrounding walls, foundation and roof is defined based on previous experiences in modeling regarding the dimensions of each element and its loading conditions, so that by reducing tension in the walls and movement towards the reservoir roof thickness values are gradually declined. Also in the foundation that these walls are connecting to it, greater thickness defined in order to provide appropriate charisma and freightage at the foot of the walls, which these amounts reduced by moving towards the reservoir center regarding tension levels decrement. In defining these elements, local axes direction pursuant to the original design assumptions and structural loading circumstance is defined.

Sections dimensions of the columns frame have been studied based on values of applied loads subjected to the size of craters, structure height and loading conditions and also previous experiences and regarding the loading conditions, ease of framing and bar bending and the thickness of the roof have been intended in the models initially. In defining these elements, local axes direction is defined according to designing the original assumptions and cracking column sections, in the models are considered based on the regulations and reduction effect of rigidity at the junctions is defined too. The primary dimensions of the columns is considered through previous experiences for modeling and regarding to the mouthpiece length, height of the structure and values of applied loads equal; to 40 cm in 40 cm. In addition, the number of the bars supposed equal to eight in each column primarily. Level of the crossbar consumption has been determined equal to 0.004 to 0.04 cm^2 plus 5 cm covering base on durability of concrete structure in using services.

Analysis Load Combinations

Regarding to the matter that structure has half-buried saved in the soil and static and dynamic water pressure, although have been applied to define different types of loads and regulations criteria of structures. To lateral forces of earthquake on reservoir structure, the seismic coefficient amount is used C equal to 0.35 and after the initial analysis and calculated intermittence time; assumption authenticity of reflection coefficient is selected equal to 2.5 estimated. Modal analysis performing and right defining of the reservoir alternation time in the model and earthquake structural tank loading by using seismic coefficient signification requires to define an effective mass for the structure during an earthquake, so regarding to the concepts of earthquake analysis, this mass defined as sum of dead load with 0.2 of live load in the model.

In brief, loads on the structure during the calculations are total of the dead load, equal to 0.72 ton/m^2 and 0.150 ton/m^2 snow load according to relevant criteria. Weight of shell elements and frames available in the models are recorded in the self- increasingly dead load by applying coefficients 1 in the loads definition. Regarding to being half-buried the structure in the soil effects of lateral soil, pressure must apply on the walls elements in contact with the concrete walls of the tank shell, so coefficient of lateral earth pressure is calculated equal to 2.57 ton/m^2 in one width meter.

Changes in soil Pressure, during an earthquake for lateral soil pressure application in the dynamic mode can be found through the relation between Whitman and Sid based on the design criteria and calculation of ground water. In addition, the resulting dynamic pressure to the shell elements of the buried in soil sidewalls can be applied. So this variable varies at height of 3m start as 1.57 (ton/m^2) to the reservoir bed level. Lateral pressure and vertical fluids are calculated by the theory of fluid mechanics in accordance with relation which is lateral pressure or vertical fluid is specific gravity of liquid, and y is desired balance depth. Regarding this structure using and existence of 4.7 m water in the reservoir, side effects of water pressure must be entered to the elements of shell surrounding concrete walls, and then vertical pressure resulting from weight of water on the reservoir bed arrives widely. Specific weight of water is considered equal to 1 ton/m^2 and amount will be equal to 4.70 ton/m^2 estimated.

In repository loading, structure wall is hard, so according to Hasner theory, fluid dynamic model is same with two degrees of freedom with viscosity about water, which are into a reservoir with a hard wall. A freedom degree is related to a mass which contains the weight of a liquid part that vibrates along the reservoir and is called hard mass (w_1), and a freedom level contains the weight of a liquid part that vibrates along alternative time independently much larger than the alternative time of the hard part and the structure calling wavy mass (w_2). Regarding changes in water pressure during an earthquake, for applying this lateral pressure in the dynamic mode, existing relations in the criteria and the design scales and calculation of groundwater can be used, and apply resulting dynamic pressure to the elements of the side walls shell. This measure in raster for two q_1 measures is equal to 1.55 ton/m^2 on ceiling and 3.97 ton/m^2 in the bed, in addition in raster for two q_2 measures is equal to 1.71 ton/m2 in ceiling and 4.13 ton/m^2 in the bed. In order to applying the forces resulting from change in temperature on the shell elements, temperature changing is applied equal to 20°C on these members regarding to this structure in the earth and bulwark on the roof. According to modeling and analysis of RCR with software 20 load combination for this structure selected based on codes and durability design of structure. Load combinations reached from dead load (DL), soil load (KHAK), soil dynamic load in x and y direction (EKHAKX/Y), snow load (SNOW), water load (WAT), dynamic water load in x and y direction (EWX/Y), earthquake loads in x and y direction (EQx/y), and temperature load (T) which related each other and make 20 load combinations (Table1).

Table 1: Load Combinations for RCR Structural Analysis

Number	Combo. Name	Load Combinations
01	Comb1	1.4 DL+ 1.7 WAT + 1.7 SNOW
02	Comb2	1.4 DL+ 1.7 KHAK + 1.7 SNOW
03	Comb3	0.75 (Comb1 + 1.87 EWX + 1.87 EQx)
04	Comb4	0.75 (Comb1 + 1.87 EWY + 1.87 EQy)
05	Comb5	0.75 (Comb2 + 1.87 EKHAKX+ 1.87 EQx)
06	Comb6	0.75 (Comb2 + 1.87 EKHAKY - 1.87 EQy)
07	Crackstc1	1DL+1WAT+1SNOW
08	Crackstc2	1DL+1KHAK+1SNOW
09	Crackdyn1	0.75 (Crackstc1 + EWX + EQx)
10	Crackdyn2	0.75 (Crackstc1 + EWY + EQy)
11	Crackdyn3	0.75 (Crackstc2 + EKHAKX + EQx)
12	Crackdyn4	0.75 (Crackstc2 + EKHAKY - EQy)
13	TEMP1,2	0.75 (Comb1 + 1.4 T)
14	TEMP3,4	0.75 (Comb2 + 1.4 T)
15	FOUND1	1.0 DL+ 1.0 WAT + 1.0 SNOW
16	FOUND2	1.0 DL+ 1.0 KHAK + 1.0 SNOW
17	FOUND3	0.75 (FOUND1 + 1.0 EWX + 1.0 EQx)
18	FOUND4	0.75 (FOUND1 + 1.0 EWY + 1.0 EQy)
19	FOUND5	0.75 (FOUND2 + 1.0 EKHAKX + 1.0 EQx)
20	FOUND6	0.75 (FOUND2 + 1.0 EKHAKY - 1.0 EQy)

BLAST LOADING AND ANALYSIS

Theoretical Background

The effects of bomb explosion on a particular target are based on the combination of charge weight and corresponding distance available from origin of the blast. Explosion load characteristics and related parameters have been described in a previous paper in detail. Over pressure time history variation following an explosion is expressed by the Friedlander equation assuming infinite target dimensions;

$$p(t) = p_s(1 - \frac{t}{t_d})\exp(-\frac{b_t}{t_d}) \tag{1}$$

where b is the waveform parameter, p_s is the peak overpressure, t_d is the duration of the positive phase, t is the time measured from the instant that the blast wave arrives, p (t) is the pressure at time t. Assumptions are made for the simplification of the over pressure profiles in conventional practice with linear variation of the pressure with time as.

$$p(t) = p_o + P_s(1 - \frac{t}{t_d}) \tag{2}$$

Where p_o is the ambient pressure, the force function can be modeled as a triangular pulse having a peak force, F_m and positive phase duration, t_d and is given by

$$F(t) = F_m(1 - \frac{t}{t_d}) \tag{3}$$

Where $F(t) = P(t) \times Area$, the equation of motion for the positive phase duration 0 to t_d, of the un-damped elastic single degree of freedom (SDOF) system is:

$$M\ddot{x} + Kx = F(1 - \frac{t}{t_d}) \tag{4}$$

Where M and K are the mass and stiffness matrices respectively while and x and $\ddot{x}$ are the displacement and acceleration respectively. By confining the response to a time less than positive phase duration t_d, the general solution of Eq. can be obtained at time t as

Displacement $x(t)$,

$$x(t) = \frac{F_m}{K}(1 - \cos\omega t) + \frac{F_m}{Kt_d}\left[\frac{\sin\omega t}{\omega} - t\right] \tag{5}$$

Velocity $\dot{x}(t)$,

$$\dot{x}(t) = \frac{F_m}{K}\left[\omega\sin\omega t + \frac{1}{t_d}(\cos\omega t - 1)\right] \tag{6}$$

Where ω is the *(circular)* natural frequency of the structure

The maximum response is defined by the maximum dynamic deflection x_m at time t_m when structural velocity is 0. Therefore, either $\dot{x}(t) = 0$ in Eq. (5) or Eq. (6) gives

$$0 = \frac{F_m}{K}\left[\omega \sin \omega t + \frac{1}{t_d}(\cos \omega t - 1)\right] \tag{7}$$

This simply implies that, at $t = t_m$

$$\omega t_m = f(\omega t_d) \tag{8}$$

Therefore behavior of a structure when subjected to blast loads is influenced by ωt_d and thus loading regimes are defined and classified as less than 0.4 for impulsive loading regime, higher than 40 for quasi- static loading regime and between them for impulsive loading regime.

The combination of peak pressure, impulse and duration of the blast load are incorporated with the loading regimes to a structure. This classification is therefore important during the structural analysis and design. The structural elements (columns or beams) may experience large inelastic deformations when subjected to blast loads. In addition, there is a degree of uncertainty in both the determination of the loading and the resulting deformation [32]. In such situations the equivalent idealized elasto-plastic system [31] can be used.

Blast Loading

The sudden release of energy from an explosive in the air produces an instantaneous high temperature, high pressure detonation wave in the atmosphere. The pressure wave causes a rapid expansion and propagation of ambient gases, and the high pressure air at the front end of these gases contains most of the explosive energy is known as the blast pressure wave. The energy carried by the blast pressure wave will decrease as the propagation distance and time increase and the pressure behind the shock wave front can instantly reduce to below the air pressure of the surrounding atmosphere. During the negative pressure phase, the air evacuated to create a vacuum and the pressure and temperature then return to the same as the ambient air. The variation of the over pressure with time for a typical shock at a particular location is shown in Figure 2. Before the arrival of the shock wave, the atmosphere pressure is Pa at ta. The pressure suddenly rises to maximum over pressure peak P_{so} after the explosion, then decays to maximum over pressure peak P_{so} after the explosion, then decays to P_a at $t_a + t_o$, later falls below the atmospheric pressure to the negative pressure peak P'_{so}, and finally returning to P_a at $t_a + t_o + t_{on}$.

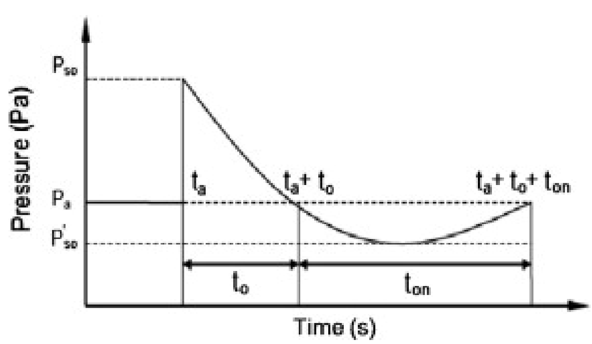


Figure 2: Over Pressure with Time for a Typical Shock

It is difficult to obtain the parameters of the blast pressure wave through theoretical analysis due to the complexity of the explosion process. Baker proposed an equation to express the pressure attenuation process:

$$P_{so}(t) = P_{so}(1-\frac{t}{t_o}) \tag{11}$$

Here, t is duration time of the pressure wave travel from the explosion to the given location. The impulse formed by positive pressure can be obtained using its integral to time.

$$i_{so} = \int_{t_a}^{t_a+t_o} P_{so}(t)dt \tag{12}$$

Some studies have proposed some usable empirical equations for shock wave parameter calculation through theoretical analysis and numerical simulations. The commonly used empirical equations include that proposed by:

$$\begin{aligned} P_{so} &= 1.4072Z^{-1} + 0.554Z^{-2} - 0.0356Z^{-3} + 0.000625Z^{-4}(0.1 \le Z \le 0.3) \\ P_{so} &= 0.619Z^{-1} - 0.033Z^{-2} + 0.213Z^{-3}(0.3 \le Z \le 1) \\ P_{so} &= 0.066Z^{-1} - 0.405Z^{-2} - 0.329Z^{-3}(1 \le Z \le 10) \end{aligned} \tag{13}$$

Consider:

$$\begin{aligned} P_{so} &= 20.06Z^{-1} + 1.94Z^{-2} - 0.04Z^{-3}(0.05 \le Z \le 0.5) \\ P_{so} &= 0.617Z^{-1} - 3.01Z^{-2} + 4.3Z^{-3}(0.5 \le Z \le 70.9) \end{aligned} \tag{14}$$

And Brode's equation:

$$\begin{aligned} P_{so} &= \frac{0.975}{Z} + \frac{1.455}{Z^2} + \frac{5.85}{Z^3} - 0.019(0.1 \le P \le 10bar) \\ P_{so} &= \frac{6.7}{Z} - 1(P_{so} \ge 10bar) \end{aligned} \tag{15}$$

Where Z is the scaled distance, expressed by $Z = R/W^{1/3}$, R is the standoff distance and W is the mass of explosive.

Blast Wave Scaling Laws

The most widely used approach to blast wave scaling is the cube root scaling law proposed independently by Hopkinson and Cranz. The law states that, similar blast waves are produced at identical scaled distances when two explosive charges of similar geometry and of same explosive but of different sizes are detonated in the same atmosphere. Thus if charges of weights W_1 and W_2 are detonated then the same peak pressure is produced at distance of R_1 and R_2, respectively. The distances $R_1$and R_2 are related as given below:

$$\frac{R_1}{R_2} = \left(\frac{W_1}{W_2}\right)^{1/3} \tag{16}$$

The duration of positive phase of a pressure wave is also given by a similar equation.

Explosive

For explosive material, *MAT_HIGH_EXPLOSIVE_BURN is used. The JWL equation of state is necessary to simulate the behavior of an explosive, its detonation velocity is 6930 m/s, and EOS is expressed as follows:

$$P = A(1-\frac{\omega}{R_1 V})e^{-R_1 V} + B(1-\frac{\omega}{R_2 V})e^{-R_2 V} + \frac{\omega E}{V} \quad (17)$$

Where A, B is linear explosion parameters; ω, R_1 and R_2 are non-linear explosion parameters; V is relative volume and E is specific internal energy of every unit of mass. The JWL EOS is used for determining the pressure of the detonation products of high explosives. According to the explosives manual, JWL EOS parameters of TNT are shown in.

AIR

The linear-polynomial EOS is used to model the behavior of the air and linear in internal energy. The pressure is given by:

$$P=C_0+C_1\mu+C_2\mu^2+C_3\mu^3+(C_4+C_5\mu+C_6\mu^2)\,E_0 \quad (18)$$

WhereE_0 is the specific initial energy, and $\mu = \rho/\rho_0 - 1$, C_i (i = 0–6) are the coefficients. For the ideal gases, the coefficients in the EOS are setting as $C_0 = C_1 = C_2 = C_3 = C_6 = 0$, and $C_4 = C_5 = \gamma - 1$, γ is the polytrophic ratio of specific heats. The pressure is then given by:

$$P = (\gamma - 1)\frac{\rho}{\rho_o}E_o \quad (19)$$

Where ρ/ρ_0 the ratio of current density to reference density, γ is the ratio of specific heats, ρ_0 is the initial density of air, and ρ is the current density of air. E_0 is the specific initial energy, with the gamma law EOS under standard atmospheric pressure and $\gamma = 1.4$, its initial energy is $E_0 = 2.5 \times 10^5$ J/kg.

Steel bar Analysis

A plastic kinematic model with a hardening effect is applied for simulate steel bar. The elastic–plastic behavior of the material with kinematic and isotropic hardening is shown in Figure 3, in which l_0 and l are the unreformed and deformed lengths of uniaxial tension specimen. E_t is the plastic slope of the bilinear stress strain curve. $\beta_0 = 0$ is for kinematic hardening and $\beta_0 = 1$ is for isotropic hardening. The present study assumes $\beta_0 = 0$.

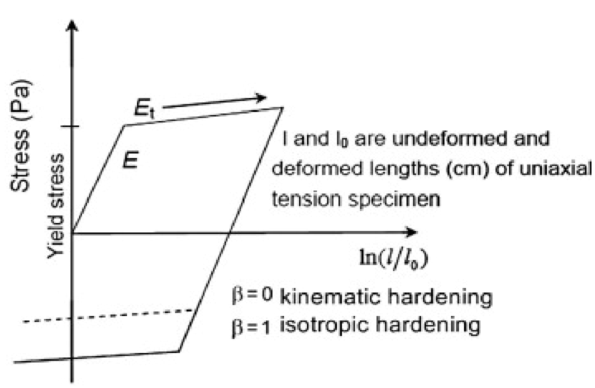
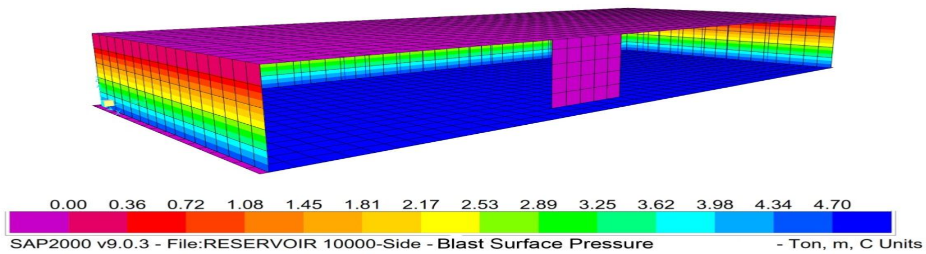


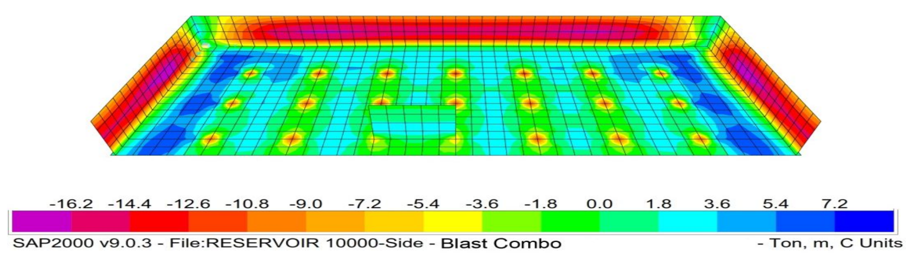
Figure 3: Kinematic Model and Isotropic Hardening

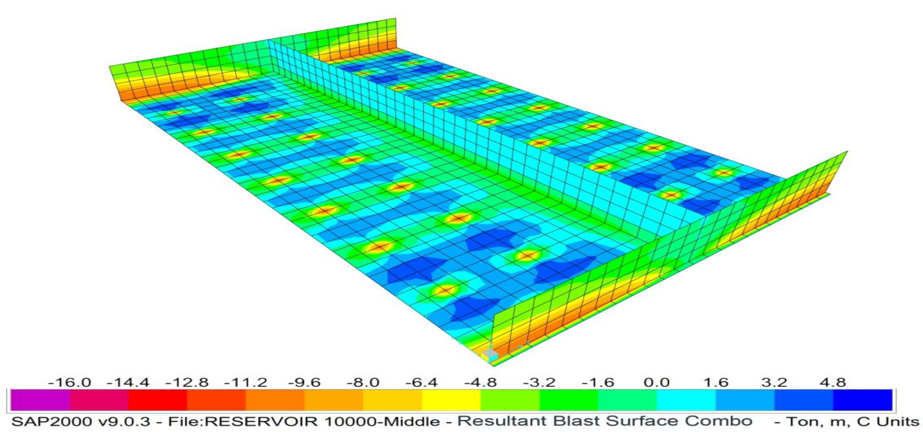
REINFORCES CONCRETE RESERVOIRS DESIGN

Modeling and Analysis Software in Use

Rao and Hinton (1993) work on analysis and optimization base on shell structures and because of using the shell elements and the need for networking and analyzing different parts of this structure to the reservoir modeling, finite element software SAP2000 with the linear analysis of the shell elements connected to the frame elements has been used. The reservoir structure is divided into six parts regarding to expansion fissures which three models has been made and checked in the abovementioned program to design the different parts of this structure. For these wall members, thickness of the shell element regarding to the length and height of the wall and its loading conditions, at the junction to the foundation moving upward is reduced up to 80 cm and is supposed at least 35 cm at the junction to the ceiling. The thickness of shear walls is supposed at least 35 cm, the foundation thickness is presumed up to 80 cm regarding to previous experiences, and anchor, transferred cutting from the walls that by moving toward the center of the tank, this amount reduces gradually, and is supposed at least 40 cm after seven meters. In addition, the thicknesses of the ceiling and excess load soil is intended at least 25 cm regarding the experiences and the anchor and transferred cutting from the walls. After geometry and loading finalization, all above models are linear static analyzed, the shell elements, and the frame elements are designed manually and by program respectively according to outcome results.







The Objective Function of RCR base on ABC Algorithm

KIA and Ghasemi (2012) research on finding objective optimization function to minimizing the RCR structure weight and developing equation during time. In equation (9) $f(W)^t_{d_i}$ is function of minimizing the RCR weight based on changing the thickness of the i^{th} element in t-time base on durability of structure , F_y is characteristic strength of steel, f'_c is characteristic compressive strength of concrete, M_{u_i} is the ultimate flexural resistance of the anchor of the i^{th} element, ϕ and λ are partial safety factors, γ is specific gravity of materials, b_w is spirit width, d_i is the thickness of the i^{th} element, V_{u_i} is The ultimate shear strength of the i^{th} element, h_i is the total thickness of the i^{th} element, α and β are design coefficients, l is the tank length and b is the tank width. A range of the bending anchor amount and the final cutting power is obtained regarding minimizing the parameters involved in the tank optimization according to performed analysis. In each of the elements, the respective function has been minimized regard to the design criteria to the optimum thickness amount of the tank shell and the used bar are calculated based on the design criteria.

$$f(W)^t_{d_i} = \sum_{i=1}^{n}\left(\frac{\left(1-\sqrt{1-\dfrac{2\dfrac{F_y}{.85f'_c}\dfrac{M_{u_i}}{\phi b_w d_i^2}}{F_y}}\right)}{\dfrac{F_y}{.85f'_c}}\alpha b d_i \gamma_1 (l+b)\right) + \sum_{i=1}^{n}\left(\frac{\beta\gamma_2 h_i V_{u_i}(l+b)}{.17\lambda d_i\sqrt{f'_c}} tg\left(\frac{h}{n(b+l)}\right)\right) \quad (20)$$

Regarding to the different variables existence in accordance with (9) relation and considered suppositions to solve the problem M_{u_i}, V_{u_i}, d_i of the i^{th} element according to Figure 3 are considered as the scope and constraints of the problem to minimize the structures weight. Regarding to the problem solving by artificial bee colony algorithm, a part of used parameters in MATLAB software are considered as colony size 10^3, limit range and parameter range estimated base RCR model analysis database, number of cycle set to 5×10^3, number of runs set to 50. The optimization code was run on a personal computer with a Intel core i7, 3.1 GHz processor with 6 GB DDR3 RAM under the Microsoft Windows 7 professional operating system.

Optimal Shell Elements Comparison

After solving the function based on the debated terms, the optimal thickness of reinforced concrete reservoirs shell cab ne considered by particle swarm algorithm methods according to Figure 4 and based on comparison of thickness of the concrete shell wall side to for situations as follow in tables.

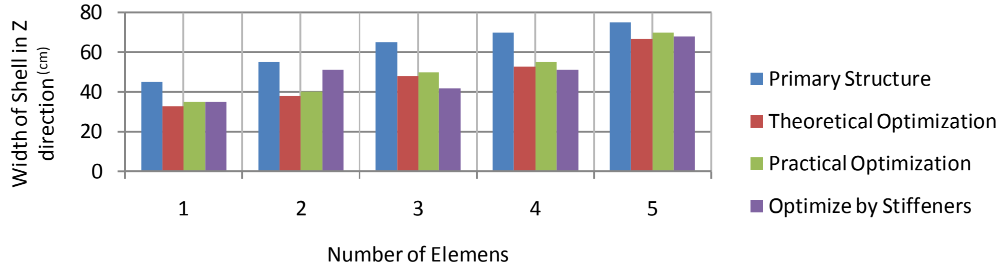


Figure 4: Thickness of RCR Shells Elements

Common-mode design, the optimized mode-design based on theory numbers, the optimized mode-design based on executive numbers, and the optimized mode-design by peripheral confirmatory sheath. In Figure 5 amount of the used bar is studied.

This section is to achieve the best confine, which contains four phases as follow which common mode design with the maximum used bar, optimization of the using bar amount based on the theory results, optimization of the using bar based on numbers with executive possibility, and optimization of this characteristic by the using confirmatory sheath. In the structure calculation by executive method, accuracy is based on the theory solution but respective numbers are chosen according to executing possibility and minimizing materials tails.

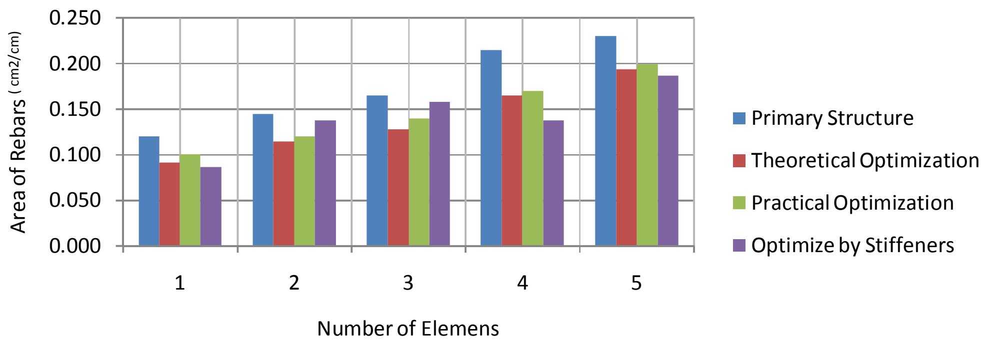


Figure 5: Area of Steel Rebars in RCR Shells Elements

CONCLUSIONS

The non-linear dynamic finite element analysis software LS–DYNA is employed to investigate the damage mechanism and dynamic responses of the RC containment subjected internal blast loading. A comparison between the fine mesh and coarse mesh for the stress, pressure and displacement shows that increasing mesh quantity may improve the accuracy of the analysis results for a scale distance of 0.368–2.181 m/kg$^{1/3}$, thus, to guarantee that the results are closer to the actual situation the finite mesh division should be as fine as possible. A fluid–structure coupling algorithm and multi-material model are adopted in this study. For the 20 m standoff distance between the explosive and containment, the relative difference in the pressure and von misses stress are compared. The weakest position and failure occurred of the containment is dome which more close to the explosion source may be subjected to the strongest impact. The blast pressure wave when the explosion in the RC containment was different from free-field blast pressure due to multiple reflections during the propagation process of blast wave which was not occurred in the free filed detonation.

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