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Optimum Safe Design of Reinforced Concrete Tanks

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بسم الله الرحمن الرحيم

وقل ربي زدني علما

صدق الله العلي العظيم

DEDICATION

TO MY FAMILY, RELATIVES AND FRIENDS, AND SPECIAL GRATITUDE TO MY WIFE AND DAUGHTER ZAINAB

CERTIFICATION

We certify that this thesis titled "**Optimum Safe Design of Reinforced Concrete Tanks"** was prepared by **Firas Hamza Majeed Al-Masoody** under our supervision at the Civil Engineering Department, College of Engineering, University of Babylon, in partial fulfillment of the requirements of the Degree of Master of Science in Water Resources Engineering.

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Optimum Safe Design of Reinforced Concrete Tanks

ABSTRACT

In view of the importance of concrete tanks in humans life and the relative high cost they usually form, the subject of "optimum safe design of reinforced concrete tanks" has been selected, considering the moment distribution as a method of analysis, taking into consideration the cost of excavation, bedding, asphalt layer materials, the construction of the body of the tanks which involves (formworks, reinforcing, water stop materials, and concrete materials), and fill works.

Because of numerous shapes and materials of the tanks, the wide-fame among them have been selected in this study, namely, the rectangular tanks (with the squared as a special case), and the circular tanks. According to the state of these tanks, they may be on-ground or buried (partially or fully). All the aforementioned type may be open or closed.

For practical considerations, volumes of the case study of (50, 250, 500, 750, and 1000 m^3) have been selected to represent a relatively small, moderately medium, medium, relatively large, and large tanks.

To prepare the optimization model, an objective function has been formulated to cover all the aforementioned costs. The optimization model involves the total cost of the structure as the objective function and dimensions as the design variables with geometrical constraints. The non-linear constraints optimization model is solved by a conventional

optimization technique, i.e., through enumeration and direct evaluation. A computer program is developed to handle the aimed solution.

The following categories of analyses have been considered:

1. Optimum dimensions with respect to tank volume.
2. Optimum dimensions with respect to the level of bed of tank.
3. Optimum dimensions with respect to the shape of the tank.

The results showed the following:

1. The adopted optimization model has been found suitable in giving the final safe results.
2. The rectangular (virtually the square) tanks came always in the first class, whereas the circular one were lag. For example, (V=250 m^3) , the optimum sections of the circular on-ground open tank were (D=9.60 m, H=3.46 m) for which the total cost(ZT= $44.319x10^6$ I.D.), whereas for the same volume and level, the optimum sections of the rectangular shape were (L=9.20 m, B=9.20 m, H=2.95 m) for which the total cost (ZT= $38.286x10^6$ I.D.).
3. The effective efficiency (defined as The overall cost measured in I.D. per one cubic meter of stored water]) of a specified state decreases with the increasing of the tank volume.
4. For all volumes considered in this research, the optimum shape is the R.C square tank.

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CHAPTER ONE

INTRODUCTION

1.1: GENERAL

Water is a very important resource to humanity .The demand on this resource is increasing in the various fields of it use, such as domestic use, industry, agriculture, electric power-generation, … etc.

There are many structures which are used to store, convey, and treat water. Water- storage tanks are very important public structures. According to the location with respect to the land level, such tanks may be on the ground, buried (fully or partially), or above the ground (i.e., elevated). According to the materials the tanks are constructed from, the metal tanks and the reinforced-concrete tanks are the most common in this respect. In respect to the shape of the tank, the most common shapes are the cylindrical (denoted hereafter as "circular") and the parallelepiped (denoted hereafter as "rectangular"). Each of the aforementioned types of water –storage tanks may be open or closed.

The reinforced-concrete tanks are widely used in Iraq because of the cheapness and availability of concrete materials. For the safe use of these structures with water, it is important to find a best and suitable safe design of these structures with an acceptable economical cost.

This research focuses on the optimum safe design of reinforced concrete water-storage tanks as they represent an important type of the hydraulic structures.

1.2 :OBJECTIVES OF THE RESEARCH

The present work deals with the optimum design of reinforced concrete tanks. The subject has a special importance because tanks are structures with repetitive nature; therefore, any reduction in the cost whatever it is small will lead to high reduction in the total cost when a large number of tanks will be constructed. The present work aims at:

1. To analyze and design a selected type of concrete tanks safely.
2. Constructing a general optimization model that contains the assumptions and constraints used in design of concrete tanks.
3. Selecting the best type of concrete tanks for selected cases through an optimization approach.

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1.3: METHODOLOGY OF THE RESEARCH

To satisfy the above mentioned objectives, the following methodology shall be followed:

1. Formulating the optimization model for the considered types and states of tanks.
2. Solving the optimization problems by a suitable method (iterative method) of solution.
3. Analyzing the final results to select the relatively-optimum one of a given shape and state (level of bed).

CHAPTER TWO

LITERATURE REVIEW

The review of literature in this chapter covers three aspects; the first concerns the developments in the analysis of the tanks; the second covers the developments in the optimal design of the tanks, and the third is the optimization process.

2.1: ANALYSIS OF TANKS

One of the tank problems is the interaction of its walls with roof or base. The work on the tank problem, as it is a progression to the work on isolated plates, has been approached using direct analytic, numerical, and analogy methods **[Ajaam, 2006]**. The following is a review of the work specifically related to the tank problem.

Buchi [1938] described a simple approximate method of determining the maximum vertical and horizontal bending moments developed in the walls of open square tanks. He assumed complete fixity for side and bottom edges of the walls.

A more rigorous solution to the problem considered by Buchi was given by **Young [1943]** through using a finite series solution based on Levy's method of analysis for rectangular plates.

The **PCA [1951]** published a booklet containing tables of moment and shear values for rectangular tank walls in which the base joint is hinged or completely fixed.

A solution of the square tank problem following a middle course between the very approximate Buchi's method and the more correct but very laborious Young's method has been described by **Craemer [1953]**.

During the development of analytical solution to the problem, **Lightfoot and Ghali [1958]** developed a numerical solution based on finite difference approach to the problem. They described two methods of analyzing rectangular tanks. The first method is based on finite deference expression. The second is called the "analogous circular tank method" and in its application it is similar to that proposed by Craemer. This method, though approximate, gives good agreement with torsionless analysis of square tanks.

The rotation of the wall-base slab and wall-base slab joints was taken into account by making use of a moment distribution method; this work was due to **Ghali [1957] and Ghali [1958]**. In this method the distribution factors were based on freely supported plates , and in this method the influence of subgrade reaction is not accounted in the calculation of the distribution factors in the base slab.

One of the most comprehensive texts on individual plates used for tanks was that of **Timoshenko** and **Woinowsky-Kriger [1959]**, which contained extensive tables and charts for the elastic analysis of plates subjected to various loads including hydrostatic type of loading.

One analogy method is that of the grid frameworks described by **Lightfoot and Sawko [1960]**. In this work, due to symmetry, the tank was developed into a number of single flat plates, and the end conditions of the plates were taken as those that would occur in the tank. These plates were then treated as a grid framework. For analytical purposes, two systems were considered which were superimposed. The first one is the system of in-plane forces and the other includes bending moments and torques. These frame systems comprised rectangular frames, each with two diagonal members. The method took into account any values of Poisson's ratio. The linearly varying hydrostatic loads on the wall and the pressure on the base were treated as equivalent point loads acting at the nodes of the elements.

Davies [1961] proposed a method of analyzing tanks resting on soil of various engineering properties, making use of the moment distribution technique. The data required for the moment distribution are available in the form of tables and charts.

Another paper published by **Davies [1962]** which describes a method of predicting the elastic behavior of cylindrical tanks by assuming contact pressure distribution. The method is suitable for granular or cohesive soils. Although the contact pressure is not actual, the analysis by this method had given a guide to the limits of the order of magnitude of bending moments to be considered in the design of tanks.

A second analogy method called the space-frame method has been presented by **Hussain [1964]**. The developed plate was considered as constituted of grid framework as previously explained. However, in the structural analysis of these analogous grid frameworks, the in-plane forces and the bending moments and torques are not separated. The deformation of the analogous members was thus taken into account in the analysis. The method took into account any value of Poisson's ratio. He concluded that the advantages of the analogy method are:

1. Its ability to be applied to any shape of tank.
2. Its ability to take into account Poisson's ratio and deformations when necessary.
3. Its amenability to computer programming.

He also found that its principal disadvantage is that the analogy has its limitations in predicting plate behavior when length to breadth ratio of plate exceeds 3:1.

The finite element method, as applied to plates, was applied to tanks by **Cheung and Zienkiewicz [1965]**. The finite element method can deal with boundary conditions more easily than the finite difference method. Indeed, it can deal with openings, interaction discontinuities, and point loads much more easily than the finite difference method. As the finite element method is a very powerful method used in conjunction with the digital computer, it has become popular for use in solving difficult design problems, including those involving discontinuities. It was therefore inevitable that it would be applied to tanks.

Cheung and Zienkiewicz [1965] applied the finite element method to the analysis of open square tanks on elastic foundation. They analyzed a square tank on a spring of four different springs constants. The deflections and moments are compared with the experimental and theoretical work. The results gave a reasonable agreement for practical purposes.

Anson and Parker [1965] advanced a computer program based on finite difference solution to calculate the deflections, bending moments, shearing forces, and ring tensions for uniform and tapered cylindrical shells subject to axisymmetric loads with idealize edge conditions. The results gave a reasonable agreement for practical purposes.

Cheung and Davies [1967 (a)] applied the finite element method to the analysis of rectangular tanks on elastic foundation taking into account the interaction between wall-wall-base. The analysis gave a good agreement with experimental results. **Cheung and Davies [1967 (b)]** applied that work to the case of long-walled tanks, ignoring the interaction between wall-wall-base. They proved that the horizontal bending moments were not negligible at and near the wall corners, as was previously assumed by many designers and recommended by various books.

Davies et al. [1970] extended the work of applying the finite element method to the analysis of long rectangular closed tanks. He proved that the one-dimensional bending action, often assumed by many designers in the long walled tanks, was significantly modified due to the restraining action of the end cross walls.

Ghali [1979] presented an approach based on the stiffness and flexibility method of analysis to derive the relevant coefficients required for analysis. A set of design tables in a dimensionless form have been provided and their use was illustrated by examples. These tables have been calculated assuming classical boundary conditions at the base, i.e, completely fixed, hinged, or free.

Starczewski [1981] applied the fundamental theory of **Ghali [1979]** to the analysis and design of some typical pressure vessels of non-circular cross section, the most common of which are the rectangular section tanks. He designed an open-top rectangular tank with continuous horizontal wall stiffeners.

Al-Muhaidi [1986] has advanced a treatment of circular storage tanks resting on a Winkler-type elastic media. He transformed the entire tank into a single equivalent one-dimension system having six degrees of freedom only. In this reference all components of tank (top slab, bottom slab, and cylindrical shell) are assumed to be prismatic.

Al-Sarrai [1988] presented a computer-based method for the analysis of non prismatic circular cylindrical tanks resting on a Winkler-type elastic media. One-dimensional finite elements were used for this purpose. The study covered open or closed tanks, tanks with central columns, tanks with intermediate columns, and tanks with any number of concentric compartments.

Sultan [1988] presented an analytical model by transforming the entire non-prismatic tank into an equivalent one-dimensional system. The developed analysis procedure was based on examining the behavior of the components of the non prismatic tank separately which are in general the top plate, the cylindrical shell, the bottom plate resting on elastic media, and a general column and/or intermediate columns. These components might be prismatic or non-prismatic. The stiffness and load matrices for each component were developed and then assembled properly to form the total (or the aggregate) stiffness and load matrices of the equivalent one-dimensional system. GTAP computer program (coded in BASIC language) was presented in this reference and applied to obtained the deflections, axial forces, shear forces, hoop forces, and bending moments in all parts of the tank.

Mahdi [1989] examined some of the elastic aspect methods used to idealize the soil-raft interaction. The particular idealization included the Winkler model, Pasternak model, Vlasov model, Kerr model, and brief exposition on Reisner model behavior.

The stiffness method is employed in the examination of axisymmetric loading of circular and annular plates. Exact closed-form stiffness matrices and fixed-end actions vectors are derived for circular and annular plates resting on Pasternak model and Kerr model, respectively. Comparisons are made between different models and the results showed good agreement with alternative exact solutions appearing in literature.

Tin-Loi et al. [1990] developed a simple finite element technique to analyze closed circular cylindrical shells with arbitrarily varying wall thickness and subjected to axisymmetric radial loading which may vary in the longitudinal direction. There technique of analysis was accomplished by considering the analogy between the theories of beams on elastic foundation and the bending of axisymmetrically loaded cylindrical shells. This analogy to a beam on elastic foundation (BEF) provided the analytical bases for the derivation of the basic properties of a typical finite element on an elastic foundation with any variation in wall thickness. The foundation modulus and the beam flexural rigidity were replaced by appropriate parameters pertaining to the shell under consideration. The comparison of results of displacement and forces obtained by this method of analysis with results obtained by **[Timoshenko** and **Woinowsky-Kriger [1959]** and **Thevendran and Thambiratnam [1986]** showed excellent agreement.

Kimence and Erguven [1992] made a study of cylindrical tanks on Vlasov model. The results have been compared to the Winkler model and elastic half-space. It was shown that the Vlasov model and half-space solution were very close.

Al-kenany [1996] made a study of a cylindrical cellular storage tank by the method of grillage. The method is applied to cylindrical cellular tanks with varieties of support and loading conditions. To assess the efficiency and accuracy of the presented grillage analysis, the cylindrical cellular tanks are also analyzed by three-dimensional finite shell element. The results from the grillage analysis are acceptably accurate when compared with the results by the three-dimensional shell element analysis. The proposed simplified method is economical and suitable for the quick and repeated analysis frequently required at the design stages.

Jaiswal et al. [2003] reviewed various codes of design for tanks with the consideration of seismic forces. It is well recognized that liquid storage tanks possess low ductility and energy absorbing capacity as compared to the conventional buildings. Accordingly, various design codes provide higher level of design seismic forces for tanks. In this article, provisions of IBC 2000, ACI, AWWA, API, Eurocode 8 and NZSEE guidelines are reviewed, to assess the severity of design seismic forces for tanks vis-à-vis those for buildings. It is seen that, depending on the type of tank, the design seismic force for tanks can be 3 to 7 times higher than that for buildings. Based on the comparison of provisions in these documents, various similarities, discrepancies, and limitations in their provisions are brought out. At the end a brief description of the Indian code is given along with a few suggestions to remove the inadequacies in the Indian code.

2.2: OPTIMAL DESIGN OF TANKS

Meichoers and Roznavy [1970] studied the optimum design of reinforced circular concrete tanks. It was assumed that the self weight of the tank is negligible, and that no axial (longitudinal) load was applied. Two cases were examined: (1) fixed at base free at top; and (2) hinged at base free at top. Only longitudinal bending and transverse tensile stresses were considered. The concrete was assumed to have zero tensile strength, and then no interaction of behavior of the concrete in each direction would occur.

The steel reinforcing was assumed to take uniaxial stress only; its lateral strength was neglected. The design objective was the minimization of the reinforcing steel volume, subjected to the constrains of equilibrium. The geometry of structure was taken as fixed with certain limits. Costs of labor and formwork were taken as fixed. No consideration was given to design parameters such as crack control, deflection, and code provisions. For the problem being considered, it was shown that the design based on lower bound analysis gave lower total steel volume than equivalent design based on elastic stress distribution, or on uniform isotropic yield-line analysis.

Adidam and Subramanyam [1982] developed a more general formulation of the optimum design of the cylindrical reinforced concrete tanks by using the constrained non-linear programming; the design was conforming to BS5337:1976. The design procedure used was a combination of the limit state method and numerical method of optimization. Cost of the tank consisting of the tank wall and floor, which included cost of concrete, reinforcement, and formwork, was chosen as the objective function in the optimization problem formulation.

Thickness of the wall and its reinforcement were treated as the unknowns of the design. The outer dimensions and the thickness of the floor were taken as fixed. The objective function was subjected to the constrains of bending, hoop tension, and crack width. No consideration was given to shear, shrinkage, and temperature constraints. It has been found that optimization coupled with limit state design results in significant reduction in the cost of the tank.

Abdul-Hussain [1985] studied the optimum design of reinforced concrete underground rectangular water tanks. The design was to conform to BS5337:1976. The provision of this code formed the constraints on the design problem. The analysis was carried out by the stiffness method. The variables of the problem included the external dimensions, number of internal baffle walls, thickness of members, and steel reinforcement area at six locations for each member. The problem in its variables and constraints was solved using Rosenbrock method for optimization after introducing the multi level optimization approach. In that study, it was found that the external dimensions have significant influence on the optimal cost with the square shape forming the best plan shape. The influence of the number of the baffle walls was found to be rather significant.

Thevendran and Thambiratnam [1986] made an optimum minimum weight design of the cylindrical concrete tanks with piecewise linear variation of the thickness, assuming the shell is fixed at the bottom. A combination of the finite element analysis and numerical method of optimization was used in the design procedure. The constraints of the problem were to conform to BS5337:1976, and the design variable was only the variation of wall thickness. The internal radius and the height of the tank were assumed to be fixed. An optimization routine based on Rosenbrock 's direct search method was used for optimization purposes.

Al-Ne'aimi and Al-Sabah [1988] made an optimum, minimum cost, design of the tapered shell R/C cylindrical tanks resting on Winkler type elastic medium. The analysis was performed according to the direct stiffness method. The radius, height of the tank, top and bottom thickness of the shell, floor thickness, and steel reinforcement area were treated as design variables. The objective function was chosen as the cost of the tank consisting of the tank shell and floor which includes cost of concrete, reinforcement, and formwork.

They devised the combined optimization approach to solve the non-linear programming problem. This approach is a combination of two techniques, the Lagrange multiplier which optimized the radius and height of the tank for a specific storage capacity, and the sequential unconstrained minimization technique (SUMT) which optimized the tapered shell thickness, floor thickness, and steel reinforcement. The study showed the important influence of soil-structure interaction, wall tapering, and tank dimensions and resulted in significant reduction in the cost of the tank.

Al-Ne'aimi and Al-Sabah [1989] presented another study on the minimum cost of underground tapered shell cylindrical tanks subjected to multiple load cases. The tank consists of circular top and bottom plates and the cylindrical shell. The soil structure interaction was taken into consideration by assuming that the bottom plate is resting on Winkler type elastic medium. The load cases considered in the problem are:

1. Stresses in the floor due to the shell fresh concrete weight.
2. Stresses in both floor and shell due to roof weight.
3. Stresses in both floor and shell due to leakage testing of tank before the earth was filled around it.

4. Stresses in all parts due to earth pressure exerted by the surrounding earth.
5. Stresses in all parts due to water and earth pressure.

It was found that the stresses constraints of the third and fourth load cases were proved to be critical.

Uraiby [1997] investigated an optimum design of a reinforced concrete rectangular water tank on elastic foundation. A combination of the finite element analysis and the numerical method of optimization was used in the design procedure. The constraints of the problem were to conform to BS8007:1987 for water retaining structures. The objective function is chosen as the cost of tank consisting of the tank walls, roof, and floor, which included cost of earth works, formworks, water proofing, reinforcement, and concrete.

The width, length, the height of the tank, plate's thickness, and the steel reinforcement area are treated as design unknowns. An optimization process was formulated using the multilevel approach. The constrained non-linear programming of the problem was performed using a combination of two methods, the Lagrange multipliers method and constrained Rosenbrock method. The soil structure interaction was taken into consideration by assuming that the floor slab is resting on Winkler type elastic media.

Hamdi [1998] performed an optimum design of a reinforced concrete cylindrical water tank resting on elastic foundation. The constraints of the problem were to conform to BS8007:1987 for water retaining structures. The objective function has been chosen as the cost of tank consisting of the tank wall, roof, and floor, which included cost of earth works, formworks, water proofing, reinforcement, and concrete.

A combination of the finite element analysis and the numerical method of optimization were used in the design procedure. The diameter and the height of the tank, plate's thickness, and the steel reinforcement area were treated as design unknowns. An optimization process was formulated using the multilevel approach (model coordinate method) developed by Kirch. The problem is subdivided into four levels. The soil structure interaction was taken into consideration by assuming that the floor slab is resting on Winkler type elastic media.

Iyengar [2000] made an optimum design of an elevated water tank supported by a truss structure, considering minimizing its weight. The problem is formulated as an non-linear program. The base of the structure is subjected to ground acceleration during an earthquake. For the response analysis, the structure is idealized as a single-degree-of freedom system. The stiffness of the system is computed by using the flexibility analysis. The objective function considered is the total volume of the structure. The lengths of the vertical members of the truss are treated as design variables with the condition that the sum of their lengths is a constant. The constraints on the natural frequency of the structure is so specified that when the tank is partially filled the first few frequencies of the liquid oscillations are kept well below the natural frequency of the structure. This is done to avoid large amplitude liquid sloshing due to earthquake excitation. For the stress constraint, the local ground acceleration during an earthquake is assumed to be a stationary random process. For response calculation, the ground acceleration is considered as a white noise. The following observations were drawn on the basis of this study:

1. Stress constraint is the only active constraint at the optimum point. The optimal design is sensitive to the way in which this constraint is defined.
2. The optimal design is sensitive to the degree of correlation between member stresses.

Ajaam [2006] studied the optimal design of steel rectangular tanks on elastic foundation based on nonlinear analysis. The optimal plastic design was based on elastic- plastic incremental – iterative finite element analysis. The eight node flat shell element is used in which five degrees of freedom are specified in each nodal point, corresponding to its three displacements and the two relations of the normal at the node. For the structural optimization problem, which is dealt with as a constrained non-linear optimization, the Modified Hooke and Jeeves method is employed considering the volume of the structure as the objective function and dimensions as the design variables with geometrical constraints.

2.3: THE OPTIMIZATION PROCESS

2.3.1 General

The purpose of optimization is to find the best possible solution among many potential solutions satisfying the chosen criteria. Designers often base their designs on the minimum cost as an objective, taking into account mainly the costs of the structure itself, safety, and serviceability.

A general mathematical model of the optimization problem can be represented in the following form:

$$\text{Optimize } Z = f\,\{X_i\} \quad ; \quad i=1, 2,\ldots,n \tag{2.1}$$

which is usually the expected benefit (or the involved cost), involves (n) decision variables {X}. Such a function is to be maximized (or minimized) subject to certain equality or/and inequality constraints in their general forms:

$$g_h\,\{X_i\} = b_h \quad ; \quad i= 1, 2,\ldots,n \;;\quad h=1,2 ,\ldots, H \tag{2.2a}$$

$$q_j\,\{X_i\} \geq b_j \quad ; \quad j= H+1, H+2, ..., H+J \tag{2.2b}$$

$$q_k\,\{X_i\} \leq b_k \quad ; \quad k= H+J+1, H+J+2, \ldots, H+J+K \tag{2.2c}$$

The constrains reflect the design and functional requirements. The vector {X} of the decision variables will have optimum values when the objective function reaches its optimum value.

2.3.2: Methods of optimization

In the last three decades, most of the developments in the field of optimization theory and methods have occurred due to the explosive growth of large computers.

The available methods of optimization can be divided into three basic categories, namely, the linear programming, the non-linear programming, and the dynamic programming.

The main characteristic of a linear programming (LP) problem is that the objective function and all the constraints are linear relationships of the decision variables. This method is widely used **[Phillips et al. 1976]**.While, if the objective function or any of the constraints is non-linear, the optimization problem is termed a non-linear problems. A variety of real-world design problem are in fact non-linear **[Phillips et al. 1976]**..

Dynamic programming is used to solve special types of optimization problems which involve multistage decision processes. The optimization of each stage will affect the next stage and the final optimum decision is ensured to be the sum of all the optimization decisions of all stages. The method is very powerful and used to solve continuous and discrete non–linear programming **[Slaby, 1987]**.

The analysis and design information given in items (3.4) and (4.2) delineate that when the objective of the optimization process is to minimize the construction cost of the respective tanks, the objective function would certainly be non-linear. Consequently, non-linear programming shall be discussed in some details hereinafter.

2.3.3: The non-linear program

There are several algorithms and techniques for solving NLP-problems. Some of these are briefly reviewed hereinafter.

2.3.3.1: The analytical approach

In this approach the problem represented by mathematical relationships (equations) which aid in the search for an optimum. The solution usually requires the use of differential calculus and the optimum exact solution is found theoretically **[Gallapher and Zienkiewicz, 1973]**. The most familiar methods, underlying this approach are:

a. Differential calaulus

This method is generally used to solve simple unconstrained NLP–problems by using the laws of differential calculus to find the optimum solution.

b. Lagrange multiplier

This method is used to solve constrained non– linear optimization problems with equality constraints and can be extended to the case of inequality constraints **[Bundy, 1984]**.

c. Geometric programming

Geometric programming is one branch of (NLP) which aims at obtaining the optimum solution for a non-linear objective function subjected to non-linear constraints. It differs from other methods in that it tries to distribute the total cost among the various terms of the objective function before finding the corresponding design variables [**Duffin et al. 1967]**.

2.3.3.2: The numerical approach

In this approach, a near optimum solution is automatically generated in an iterative manner. An initial guess is used as a starting point for search of better solutions. The search continues until no further improvement in the objective function is possible or until a certain convergence criterion is satisfied, which indicates that the optimum solution has been achieved within the desired accuracy. Since most of practical design problems can not be solved by analytical methods, numerical techniques are of great importance.

Two such approaches, namely, the direct search and the gradient search, are discussed briefly hereinafter.

a. The direct search

Here, the search proceeds with evaluation of the objective function only in an iterative manner until a local or an approximate optimum solution is reached. The steps sizes and the direction of moves at each iteration represent the main features of this approach. Many methods are proposed as standard algorithms such as Rosenbrock method **[Rosenbrock, 1960]**, the pattern search of Hooke and Jeeves **[Hooke and Jeeves, 1961]**, the complex method **[Bundy, 1984]**, and Fletcher and Powell method **[Bundy, 1984]**.

b. The gradient search

The basic idea here is to evaluate the gradient of the objective function at a point and utilize it to improve and accelerate the search. The acceleration is attained by finding the direction of the steepest descent, following this direction until no further improvement is possible, then the direction is changed again. The search continues until the gradient becomes zero, indicating that the optimum solution is reached.

The most widely used methods following this approach are the Rosen method [**Rosen, 1960]**, the Sequential Unconstrained Minimization Technique (SUMT)[**Fiacco and McCormick, 1968],** and Fletcher and Powell method [**Bundy, 1984]**. These methods require smaller number of iterations than the direct search methods; however, **[Carpenter and Smith, 1975]** showed that the high order derivatives of this method are more expensive, needing longer computer time than the direct search methods.

CHAPTER THREE

REINFORCED - CONCRETE TANKS

3.1: WATER-STORAGE TANKS

The present humans' life enforced the need to certain fluids on a daily scale, although such a need varies, to some extent, spatially and temporarily. However, the availability of the needed fluids may vary, on a very larger extent, spatially and/or temporarily, but in a contradicting fashion to that of the needs. Consequently, the storage of such fluids during their presence in excess of the needs to times (or locations) of shortage is inevitable. Of particular interest in this respect are liquids, and of a very specific interest is water.

The primary purpose of water storage is to ensure that an adequate supply of water is available at all times **[USEPA, 2002]**. The need to water may be on a very large scale, e.g., in agriculture. For such a need , impounding reservoirs are usually established. However, for a much-lesser extent, various types of storage facilities are usually employed. Practically, the capacity of such storage facilities may vary from, say (10 m^3), to more than (10000 m^3). The common designation to such facilities is "storage tanks".

Water storage tanks may be classified in a variety of ways.

According to their use, water storage tanks may store raw water, clean (purified) water particularly for drinking and some specific industrial uses, and waste water.

According to the base level with respect to prevailing land level nearby, such tanks may be on the ground, buried (fully or partially), or above the ground (i.e., elevated).

On the ground and buried water-storage tanks are used to reduce the water treatment plant peak production rates and also as a source of supply for repumping to a higher pressure level. However, the above-ground (i.e., elevated) water storage tanks are used to provide adequate system pressures and flows during periods of peak water demands **[DAAF, 1985].**

According to the state of the top of the tank, the elevated, the on-ground, and the partially-buried tanks may be open or closed, whereas the fully-buried tanks should be of the closed type. The covered tanks may have concrete, structural metal, or flexible covers **[USEPA, 2002]**.

According to the materials the tanks are constructed from, the most common in this respect are the metal tanks and the reinforced-concrete tanks.

In respect to the shape of the tank, water storage tanks may take any reasonably-practical geometrical shape. The most common shapes are the cylindrical (denoted hereafter as "circular") and the parallelepiped (denoted hereafter as "rectangular").

Each of the aforementioned types of water storage tanks has its own advantages and disadvantages in respect to fulfilling the requirements of the intended uses and the considerations related to the chosen location, beside the general technical and economical aspects.

This research shall consider the reinforced-concrete tanks of the cylindrical and the parallelepiped shapes as shown in Fig(3.1), constructed on and under the ground surface.

(a) A circular reinforced concrete tank under construction.

(b) A rectangular reinforced concrete tank under construction.

Fig. (3.1) Concrete tank shapes.

3.2:REINFORCED-CONCRETE WATER-STORAGE TANKS

3.2.1: General requirements

In all water-retaining structures, strength and stability are of paramount importance. In environmental engineering, concrete structures serviceability, in terms of limited deflections and cracking, durability, and low permeability , demand equal consideration. In these structures, concrete will be in contact with water or waste water and, thus, should: **[ACI 350-R1989]**.

a. Be extremely dense and impermeable to minimize contamination of water supplies or the environment.
b. Provide maximum resistance to natural or processing chemicals.
c. Provide smooth surface to minimize flow resistance.
d. The tank should be water-tight.
e. Circulating (quiescent water for a long time will worsen the quality of the stored water)

3.2.2: Location

The storage tanks should be located above the level of the (100-year) flood elevation. When feasible, the foundation for on-ground level storage tanks, and elevated storage tanks should be located at least about (1 m)(3 ft) above the (100-year) flood elevation, while buried (partially or fully) water-storage tanks should be located at least about (15m) (50 ft) from any parts of a subsurface sewage disposal system or sanitary sewers and at least about (7.5 m) (25 ft) from the nearest water course, storm drain, or other source of pollution [**SCDPHDWS, 2006**].

3.2.3: Sizing

Usable storage means the volume of stored water that can be effectively utilized without causing the distribution system pressure to fall below the minimum required pressure levels or causing damage to the pump units, if any.

The following are intended to be general guidelines for sizing the most common types of storage tank configuration. Additional criteria specific to the public water system or other types of storage tank configuration may be required to be evaluated in order to properly sizing a storage tank [**SCDPHDWS, 2006**].

(i) Capacity, operational strategy, and back-up capability of source of supply.

(ii) Source of supply pump stations and treatment facilities.

(iii) Equalization storage during peak demands.

(iv) Fire flow storage (if fire protection is provided).

(v) Emergency reserve storage.

(vi) Future growth of the system or service area served by the storage tank.

(vii) System pressure at the highest and lowest customer service elevations served by the storage tank.

The usable storage capacity should be able to satisfactorily meet all the required demands for which the storage tank is sized. The minimum usable storage capacity for storage tanks not providing fire protection should be equal to the average daily demand, (ADD), of the public water system or service area served by the storage tank. Moreover, storage tanks should be sized to achieve a balance between hydraulic requirements and water quality maintenance. Excessive storage capacity should be avoided whenever possible to minimize long detention times and water quality deterioration. If excessive capacity is required due to future growth, consideration should be given to a phased approach by designing the storage tanks with the ability of being expanded in the future, or space should be provided to construct additional tank(s) to met additional future demands [**SCDPHDWS, 2006**].

3.3: GENERAL DESIGN REQUIREMENTS

3.3.1: Considerations concerning cracks

The formation of cracks in concrete results from several sources such as plastic shrinkage, structural cracks, and drying shrinkage**[Barenberg and Allison, 2003**].

Plastic shrinkage cracks develop almost immediately after placing the concrete while it is still fresh or in a "plastic" state; plastic shrinkage cracks are usually the result of improper placement and/or curing and are often associated with hot weather placement. However, it may occur due to any condition that produces rapid evaporation of moisture from the concrete surface. Typically, concrete specifications call for proper placement, vibration, and curing the concrete. If these items are performed correctly, paying particular attention to proper curing, plastic shrinkage cracks can usually be avoided.

Structural cracks are the result of the concrete being subjected to load in excess of the design loads, provided that deflections have been checked in the design. Most structural cracks can be avoided by following the code requirements for deflection and reinforcement.

Drying-shrinkage cracks occur when water is released from the concrete mix during the curing process. As drying takes place, concrete near the surface dries and shrinks faster than the inner concrete, causing a build-up of tensile stresses and eventual cracks; these cracks account for the majority of cracks in a concrete structure. This is where careful design and specifications can make the difference between structures that leak and structures that perform well.

Several factors in the mix design have an effect on the drying shrinkage of concrete; these factors include the water/cement ratio, size and type of the coarse aggregates, and the proportion of fine to coarse aggregates. Though a well-designed mix can minimize shrinkage of the concrete, some shrinkage will still occur. Understanding the interaction between the structure elements is also critical to reducing cracks in concrete. If concrete is allowed to shrink without restrains, it is simply to get shorten and cracks will be developed; for this reason many slabs on grade have minimal cracking, provided that joints are reasonably spaced. As the concrete shrinks, it slides on the soil below. If the distance between joints is too large, the drag on the soil gets to be too great and cracks will develop**.** For this reason, concrete is particularly prone to cracking where it is placed on top of existing concrete, such as the base of the wall placed on previously placed floor slab. Generally, the floor has already had a chance to shrink before the wall is placed; as the wall tries to shorten, the bond with the hardened concrete restrains it, creating vertical cracks in the wall as illustrated in figure (3.2). It is common to see these cracks end just above the floor. This is because the upper portion of the wall is not restrained by the slab and simply gets shorter as the shrinkage takes place **[Barenberg and Allison 2003].**

Cracks width must be minimized in tank walls to prevent leakage and corrosion of reinforcement and the wall thickness should be sufficient to keep concrete from cracking [**PCA, 1993**].

The amount, size, and spacing of reinforcement bars have a great effect on the extent of cracking. The amount of reinforcement provided must be sufficient for strength and serviceability including temperature reinforcement and shrinkage effects; the size of reinforcing bars should be chosen recognizing that

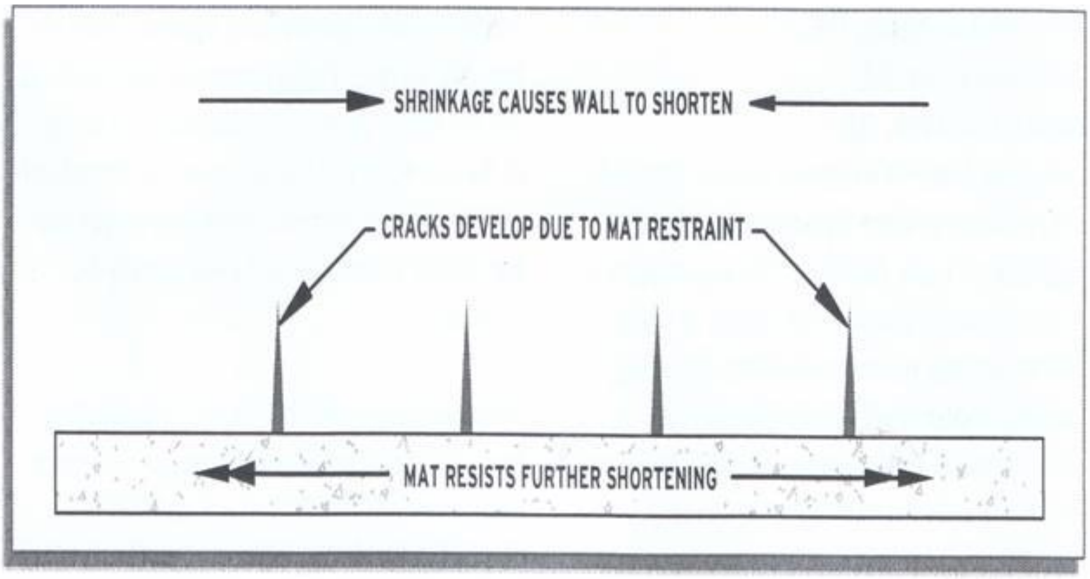


Figure (3.2) Cracks in the wall due to restraint with mat

[After **Barenberg and Allison, 2003]**

cracking can be better controlled by using a large number of small diameter bars rather than fewer larger diameter bars.

Spacing of reinforcing bars should be limited to a maximum of (300) mm, and the minimum concert cover for the reinforcement for the tank wall should be at least (50mm) **[ACI 350-R, 1989]**.

3.3.2: Joints

Concrete normally undergoes small changes in dimensions as a result of exposure to the environment or by the imposition or maintenance of loads. The effect may be permanent contractions due to, for example: initial drying, shrinkage, or irreversible creep. Other effects are cyclical and depend on service conditions such as environmental differences in humidity and temperature or the application of loads and may result in either expansions or contractions. In addition, abnormal volume changes, usually permanent expansion, may occur in the concrete due to sulfate attack, alkali-aggregate reactions, and other conditions **[ACI-504R, 1997]**.

The results of these changes are movements, both permanent and transient, of the extremities of concrete structural units. If, for any reason, contraction movements are excessively restrained, cracking may occur within the unit. The restraint of expansion movement may result in distortion and cracking within the unit or crushing of its end and the transmission of unanticipated forces of abutting units. In most concrete structures, these effects are objectionable from a structural viewpoint. One of the means of minimizing them is to provide joints at which movement can be accommodated without loss of integrity of the structure **[ACI-504R, 1997]**.

Joints are invariably provided in the water-storage tanks such as movement joints (which are contraction, expansion, or sliding joints) and construction joints. Where the length of structure is very large the construction joints shall be used to achieve continuity of concrete when the whole structure can not be casted at one time**.** Contraction joints in structures are often called control joints because they are intended to control crack location. Expansion joints in structures are often called isolation joints because they are intended to isolate structural units that behave in different ways **[ACI-504R, 1997].**

3.4:STRUCTURAL ANALYSIS OF REINFORCED - CONCRETE TANKS

The method of analysis considered in the present research is the moment distribution method to determine the moments between the wall-wall, floor, and roof junction of the concrete tanks.

The usual procedure of the moment distribution method could be used to take account of the continuity of the walls of concrete tanks with their roofs or bases. A vertical element of the wall is considered together with the horizontal element of the base or the roof. The method involves the calculations of the moments at the ends of the elements under artificial conditions of restraints, then the unbalanced moments are distributed by arithmetical proportion when the artificial restraints are removed. The fixed-end moments per unit length developed at the edges of the concrete wall due to water pressure and those developed at the edges of the roof and base plates are determined; the unbalanced moment is distributed between the connecting elements in proportion to their stiffnesses. The term "stiffness" means the moment needed at the end of the concrete wall or plate to produce a unit rotation of this end. Also, if a moment is distributed to one end of the concrete wall or the plate while the other end is held fixed, a fraction of the distributed moment is carried over to the fixed end of the wall or the plate [**Ghali, 1958]**.

The systematic procedure of any structural design of a reinforced-concrete member virtually consists of calculating the critical forces and moments acting on that member and then determining the respective appropriate practical dimensions and reinforcement accordingly.

As it is previously mentioned, the cases to be considered in this research are the rectangular tanks and the circular tanks, each of which may be on the ground or buried . The design procedure for each of the aforementioned cases is discussed in the following sub-sections.

3.4.1: Moment effects

When the top of the wall and the roof slab are made continuous, as shown in Fig (3.3), the deflection of the roof slab will rotate the top of the wall; this rotation will induce a moment at the top of the wall. The procedure used to determine the amount of the moment transferred from the roof to the wall is similar to moment distribution of continuous frames. The moment at the top of

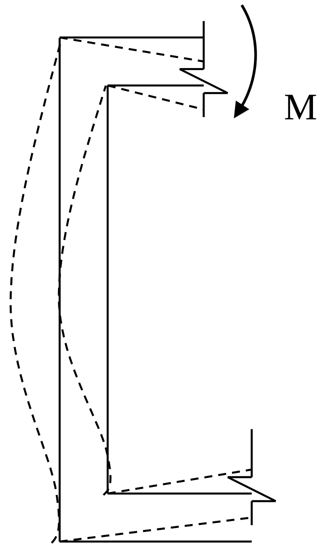


Figure (3.3) Wall with moment applied at top [After **PCA (1993)].**

the wall is first computed on the assumption that the top end of the wall is fixed. A correction is then made for rotation of the top end of the wall caused by the continuity between the roof slab and the wall **[PCA, 1993**].

The fixed end moment at the top end of the wall is determined for the triangular loading whereas the fixed end moment of the roof slab is determined for the uniformly distributed loading. These fixed end moments are as follows **[Case et al. 1999]**:

$$MF_{w1} = (\gamma_w H^3) / 30 \tag{3.1}$$

$$MF_{w2} = (\gamma_w H^3) / 20 \tag{3.2}$$

$$MF_{w3} = (\gamma_w H) L^2 / 12 \tag{3.3}$$

$$MF_{w4} = (\gamma_w H) B^2 / 12 \tag{3.4}$$

$$MF_R = (W_u L^2) / 12 \tag{3.5}$$

where:

MF_{w1}= fixed end moment of the top end of the wall, (KN.m/m);

MF_{w2}= fixed end moment of the bottom end of the wall, (KN.m/m);

MF_{w3}= fixed end moment of the long wall in horizontal direction, (KN.m/m);

MF_{w4}= fixed end moment of the short wall in the horizontal direction, (KN.m/m);

MF_R=fixed end moment of the roof slab, (KN.m/m);

γ_w=weight density of the water, (KN/m^3);

W_u= uniformly distributed load due to (dead + live) of roof slab (KN/m^2);

H = height of the tank, (m); and

L = length of the roof slab, (m).

In many cases, the floor slab and tank wall are one integral unit as shown in Fig. (3.4). Because of continuity, a portion of the bending moments that may be present in the floor slab will be transferred to the tank wall. As long as the base of the wall is artificially fixed against any rotation, it is subject to two moments, one moment is due to the pressure of the liquid and the other is due to the moment at the edge of the floor slab. When the artificial restraint is removed, the joint will rotate and the moments will be redistributed between the base slab and the wall **[PCA, 1993**].

The fixed end moment at the bottom end of the wall is determined for the triangular loading, and the fixed end moment of the floor slab is determined for the uniformly distributed loading. These fixed end moments are as follows **[Case et al. 1999]**:

$$MF_{w2}= (\gamma_w H^3) / 6 \tag{3.6}$$

$$MF_F = (W_u L^2)/ 12 \tag{3.7}$$

where:

MF_{w2}= fixed end moment of 1m width of the bottom end of the wall, (KN.m/m);

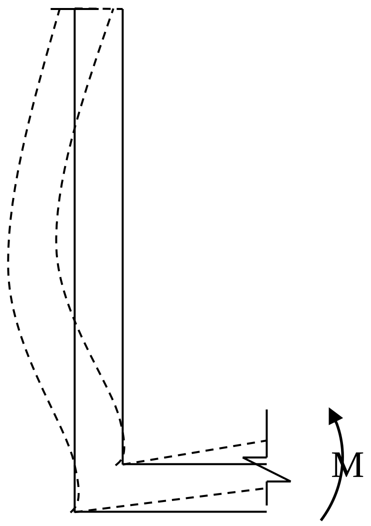


Figure (3.4) Wall with moment applied at base [After **PCA (1993)**].

MF_F=fixed end moment of 1m width of the floor slab, (KN.m/m);

γ_w= weight density of the water, (KN/m^3);

H = height of the tank, (m);

L = length of the base slab, (m); and

W_u= uniformly distributed load (live and dead) on the floor slab(KN/m^2).

3.4.2: The rectangular tank

In a rectangular tank the common side edge of two adjacent wall panels is first considered artificially restrained so that no rotation can take place about the edge. Fixed-edge moments are usually dissimilar in adjacent panels and the differences which correspond to unbalanced moments tend to rotate the edge. When the artificial restraint is removed these unbalanced moments will induce additional moments in the panels. Adding induced and fixed-end moments at the edge gives final end moments, which must be identical on both sides of the common edge [**PCA, 1969]**.

3.4.2.1: Rectangular tank resting on ground

(A) Rectangular tanks have moments distributed in the two directions when the ratio of length (L) to breadth (B) is equal or less than (2). Tanks walls are designed as a continuous frame subjected to pressure varying from zero at top to maximum at (H/4) or (1) m, whichever is more. The bottom portion (H/4) or (1) m, whichever is more is designed as a cantilever as shown in Fig.(3.5). Walls are subjected to bending and direct tension. Bending moments in the walls can be found by any elastic method. The moments at support between the wall-wall junction (Ms) shall be computed by moment distribution method for one-quarter of the tank plan due to symmetry. Bending moment at the center of long and short walls and the vertical bending moment in the portion (h) of the open tank resting on ground due to water pressure are as follows **[Syal et al. 2005]**:

$$B.M_L = \gamma_w (H-h) L^2/8 - Ms_1 \quad (3.8)$$

$$B.M_B = \gamma_w (H-h) B^2/8 - Ms_1 \quad (3.9)$$

$$B.M_V = (\gamma_w H h^2)/6 \quad (3.10)$$

where :

$B.M_L$= bending moment at center of long wall in the horizontal direction, (KN.m/m)

$B.M_B$= bending moment at center of short wall in the horizontal direction, (KN. m/m)

$B.M_V$= vertical bending moment in the bottom portion of the wall (KN. m/m).

Ms_1= the average bending moments induced at support between the walls panel at junction of the tank on the horizontal direction determined by moment distribution, (KN. m);

γ_w= unit weight of water in (KN/m^3)

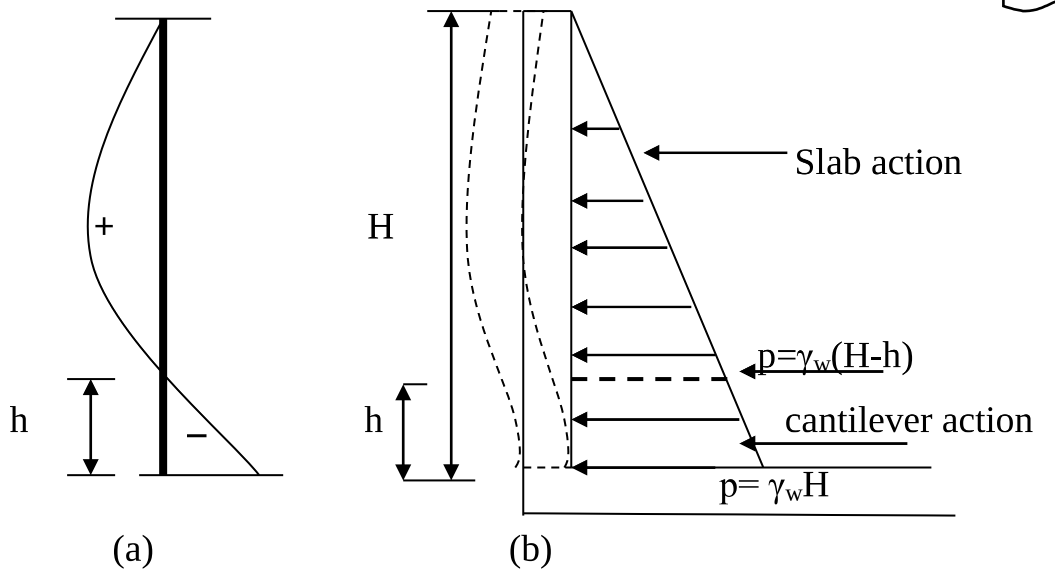


Fig. (3.5): Rectangular concrete tank [After **Syal et al. (2005)]**.

(a) The moment diagram of the wall.

(b)The distribution of loads on the wall of the rectangular tank

If the ratio between the length and the height of the rectangular tank(L/H) is more than (2) the wall of the tank is designed as cantilever **[PCA, 1969].**

Along vertical edges of walls, the shear in one wall will cause axial tension in the adjacent wall and should be combined with the bending moment for the purpose of determining the tensile reinforcements **[Syal et al. 2005]**.

Direct tension in long and short walls are equal to:

$$T_L = \gamma_w \, (H - h) \, B / 2 \qquad (3.11)$$

$$T_B = \gamma_w \, (H - h) \, L / 2 \qquad (3.12)$$

where :

T_L= direct tension of long wall in (KN/m of long wall),

T_B= direct tension of short wall in (KN/m of short wall).

p= water pressure from water surface to h=(H/4 or 1m) whichever is more in (KN/m^2), p= $\gamma_w(H-h)$

When the base slab is rigidly connected to the tank walls (as is generally the case) the bending moment at the junction between the walls and the floor slab shall be taken into account as mentioned in (3.2.2).

In order to determine the design moment in the floor slab of the tank, two cases should be investigated as show in figure (3.6). The first occurs when the tank is empty and subgrade reaction is only applied on the floor slab, while the second case occurs when the tank is full with water**[Syal et al. 2005]**.

The moment at support between the base floor slab and the vertical walls of the tank is determined by moment distribution; while the bending moments at the center of the base floor slab are as follows **[Case et al. 1999]**:

$$B.M_{F1} = (R\ L^2) / 8 - Ms_2 \qquad (3.13)$$

$$B.M_{F2} = (\gamma_w H)L^2/8 - Ms_2 \qquad (3.14)$$

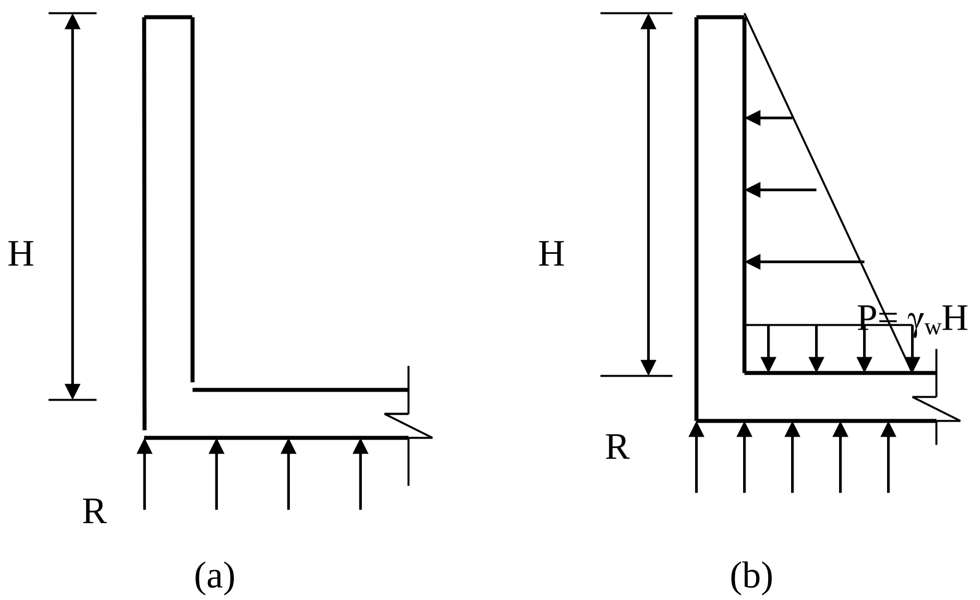


Figure (3.6) Cases of loading of a concrete tank [After **Syal et al. (2005)]**

(a) The tank is empty. (b) The tank is full with water.

where:

$B.M_{F1}$= bending moment at the center of floor slab in case of tanks empty,(KN.m/m);

$B.M_{F2}$= bending moment at the center of floor slab in case of tank full with water, (KN.m/m);

R = Subgrade reaction of soil, (KN/m^2).

Ms_2 = the average bending moments induced at support between the bottom end of the vertical walls of the tank and the floor slab determined by moment distribution, (KN. m).

The floor slab subjected to a direct pull from the base of the walls which equals to:

$$T_F = \gamma_w \ H^3 / 2 \qquad (3.15)$$

where :

T_F= direct tension in the floor slab in (KN).

When the tank is covered with a concrete roof slab, the final bending moment on the roof slab and the vertical bending moments in the tank walls are as follows **[Case et al. 1999]**:

$$B.M_R = W_u \ L^2 / 8 - Ms_3 \qquad (3.16)$$

$$B.M_W = 0.0641 \ \gamma_w H^3 - (Ms_3 + Ms_2)/2 \qquad (3.17)$$

where:

$B.M_R$= the bending moment at the center of roof slab, (KN.m/m).

$B.M_W$= the vertical bending moment of the tank wall which occurs at a distance equal to (0.5477 H) from the top end of the tank wall, (KN. m/m).

$0.0641\ \gamma_w H^3$= the total static moment of member with triangular load which equals to the moment at the center of the member plus the average moment at supports **[Case et al. 1999]**, (KN. m/m).

Ms_3= the bending moments induced at support between the top end of the vertical walls of the tank and the roof slab determined by moment distribution, (KN. m).

(B) For rectangular tanks in which the ratio of length (L) to breadth (B) is more than (2):

The long walls are designed as cantilevers and the short walls as slabs supported on long walls. Bottom portion of short walls within (H/4) or (1) m, whichever is more is designed as cantilever**.** The bending moments in long and short walls of the tank shall be equal to **[Syal et al. 2005]**:

$$B.M_L = \gamma_w H^3 / 6 \tag{3.18}$$

$$B.M_{SB} = \gamma_w (H - h) B^2 / 12 \tag{3.19}$$

$$B.M_B = \gamma_w (H - h) B^2 / 16 \tag{3.20}$$

$$B.M_{BV} = \gamma_w H h^2/6 \quad \text{or} \quad \gamma_w H/6 \quad \text{(whichever is more)} \tag{3.21}$$

where:

$B.M_L$= maximum bending moment at base of long wall, (KN. m/m).

$B.M_B$= maximum bending moment at center of long wall, (KN. m/m).

$B.M_{SB}$= maximum bending moment at support of short wall, (KN. m/m).

$B.M_{BV}$= maximum Cantilever bending moment for bottom portion of short wall, (KN. m/m).

In addition to bending moment, short and long walls are subjected to direct tension on the assumption that the end one meter width of long wall contributes to direct tension on the short wall which shall be equal to **[Syal et al. 2005]**:

$$T_L = \gamma_w (H - h)\, B / 2 \qquad (3.22)$$

$$T_B = \gamma_w (H - h) \qquad (3.23)$$

where:

T_L= direct tension in the long wall, (KN/m of long wall);

T_B= direct tension in the short wall, (KN/ m of short wall).

When the base slab is rigidly connected to the tank walls (as is generally the case) the bending moment at the junction between the walls and the floor slab shall be taken into account as mentioned in (3.2.2).

Two cases should be investigated as shown in figure (3.6) for the floor slab of the tank. The first is when the tank is empty and subgrade reaction is only applied on the floor slab, while the second case is when the tank is full with water **[Syal et al. 2005]**. The bending moment at the center of the base floor slab and the moment at supports between the base slab and the vertical walls are found by the same method of the rectangular tank of ratio of length to breadth less is than (2) .

If the tank is covered with a concrete floor slab as shown in figure (3.4), the roof slab will behave as a one way slab because of the ratio of length to breadth being more than (2). This means that the bending moment acts in the short direction of the slab and minimum steel area for temperature and shrinkage is provided in the long direction **[Nilson et al. 2004].** The final bending moments at roof slab **[ACI-318R, 2005]**.

$$B.M_1 = W_u B^2 / 12 \qquad (3.24)$$

$$B.M_2 = W_u B^2 / 8 - Ms3 \qquad (3.25)$$

where:

$B.M_1$= the bending moment at support of one way roof slab, (KN. m/m).

$B.M_2$= the bending moment of short direction at center of one way roof slab, (KN. m/m)..

The vertical bending moments in the tank walls are as follows:

$$B.M_{WL}= 0.0641\ \gamma_w H^3 - (Ms_3 + Ms_2)/2 \qquad (3.26)$$

where:

$B.M_{WL}$= the vertical bending moment of the long wall of the tank which occurs at distance equal to (0.5477 H) from the top end of the tank wall, (KN. m/m).

(ii): The underground rectangular tank

(A) For rectangular tanks in which the ratio of length (L) to breadth (B) is equal or less than (2).

When the tank is built underground, the tank walls must be investigated for both internal and external pressures. The latter may be due to earth pressure or to a combination of earth and groundwater pressure (i. e., saturated pressure). The principles of analysis of such tank are the same as those for a tank resting on ground. Two cases should be investigated; the first is when the tank is empty , where only earth pressure acting from outside, while the other case is when the tank is full with water and no earth pressure acting from outside as shown in figure (3.7) **[Syal et al. (2005)]**.

In the case when only earth pressure acting from outside, the density of water (γ_w) should be replaced by the density of dry soil (γ_S) in the equations., and the pressure of dry soil as in **[Fredrick et al. 2001]** shall be equal to:

$$P_S= 1/2\ K_A\ \gamma_S\ H^2 \qquad (3.27)$$

where:

P_S= active earth pressure force in KN per linear meter of fill height

γ_S= dry density of soil, (KN/m^3);

H= height of earth fill, (m);

K_A= active earth pressure coefficient which equal to:

$$K_A = \tan^2 (45 - 0.5\,\phi) \tag{3.28}$$

where:

ϕ= angle of repose of soil.

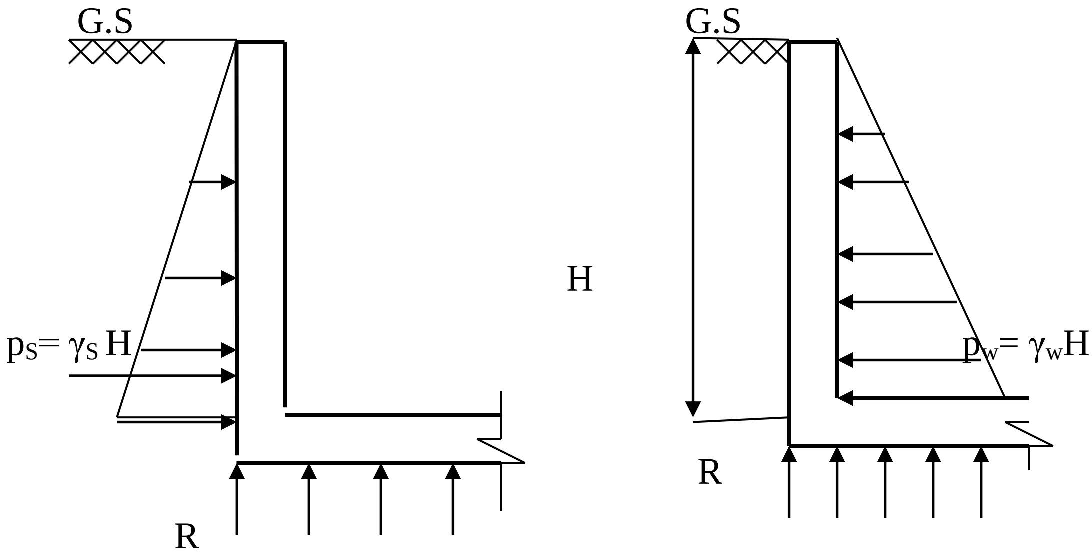


Figure (3.7) Cases of loading of underground concrete tank

(a)Earth pressure only acting on wall (b) Water pressure only acting on wall

[After **Syal et al. (2005)].**

When the soil is saturated with water, the pressure of saturated soil formula acting from outside of the walls shall be equal to **[Syal et al. (2005)]**:

$$p_{SS}=\gamma_w H + (\gamma_S - \gamma_w)\ \frac{1-\sin\phi}{1+\sin\phi} H \qquad (3.29)$$

where:

p_{SS}= pressure due to saturated soil in KN/m^2.

Whenever there is a possibility of ground-water table to rise above the base slab, not only walls are to be designed for saturated soil up to the extent of water above the base slab, but also the base slab is to be designed for the net uplift pressure of water (less weight of slab for tank empty case). In addition, check has to be applied for stability of the tank as a whole against uplift pressure **[Syal et al. 2005].**

When a tank is built below ground with earth covering the roof slab, actual load distribution may not necessarily be triangular as assumed in the internal pressure of water. The earth cover causes a trapezoidal distribution of lateral earth pressure on the walls. In this case it gives a fairly good approximation to substitute a triangle with the same area as the trapezoid representing the actual load distribution; the intensity of load is the same at mid depth in both cases and when the wall is supported at both top and bottom edges, the discrepancy between triangle and trapezoid has relatively little effect at and near the supported edges [**PCA, 1969]**.

(B) For rectangular tanks in which the ratio of length (L) to breadth (B) is more than (2).

The principles of analysis of such tank are same as that for tank resting on ground of ratio of length to breadth more than (2). Two cases should be investigated in this case. The first is when the tank is empty (no water from inside) and only earth pressure acting from outside, while the other case is when the tank is full with water and no earth pressure acting from outside as shown before in figure (3.7) **[Syal et al. 2005]**.

In the case when only earth pressure is acting from outside, the density of water (γ_w) should be replaced by the density of dry soil (γ_S) in the equations, and the pressure force per linear height of dry soil shall be equal to **[Fredrick et al. 2001]**:

$$P_S = 1/2\, K_A\, \gamma_S\, H^2 \qquad (3.30)$$

where:

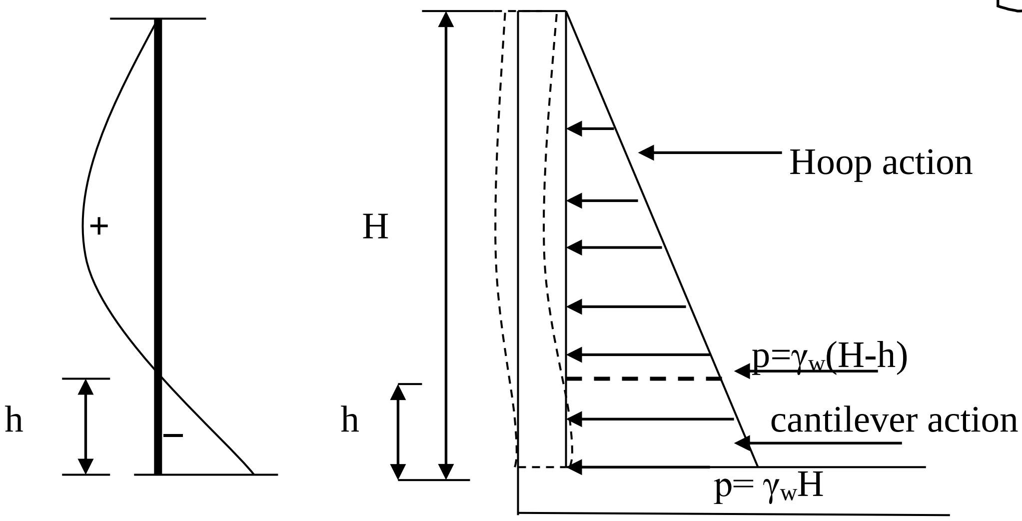
$$K_A = \tan^2 (45 - 0.5\,\phi) \qquad (3.31)$$

When the soil is saturated with water, the pressure of saturated soil formula acting from outside of the wall shall be equal to **[Syal et al. 2005]**:

$$p_{SS} = \gamma_w H + (\gamma_S - \gamma_w)\ \frac{1 - \sin\phi}{1 + \sin\phi} H \qquad (3.32)$$

<u>3.4.3: The circular concrete tank</u>

Due to restraint at the base and roof, the walls of the circular tank are subjected to hoop tension and bending moment. In this case the walls will resist the water pressure partly by hoop action and partly by cantilever action. Elastic behavior of circular tank walls show that for a certain height from base there will be predominate cantilever action, and at higher levels there will be predominate hoop action as shown in Fig.(3.8) **[Syal et al, 2005]**.



(a) Bending moment diagram (b) Force diagram

Fig. (3.8) Circular concrete tank resting on ground.

(i): Circular tank resting on ground

The moment at the base of the wall is first computed on the assumption that the base is fixed, and correction is then made for rotation of the base of the wall caused by the continuity between the floor slab and the wall. The fixed-end moment at the base of the wall for the liquid pressure is determined for the triangular loading with the coefficient from table (3.2) according to the value of (H^2/Dt_1). As long as the base of the wall is artificially fixed against any rotation, it is subjected to two moments. One is due to the outward pressure of the liquid, the other is due to the moment at the edge of the base slab. When the artificial restraint is removed, the joint will rotate and the moments will be redistributed **[PCA, 1993]**.

The relative stiffness of the wall of the circular tanks may be determined by multiplying ($E_C t_1^3/H$) by the coefficient k_r from table (3.1) depending on the value of (H^2/Dt_1) **[PCA, 1993]**.

Therefore, the base moment and the relative stiffness of the circular tank are as follows:

Table(3.1): Relative stiffness(k) of cylindrical wall [After **PCA, 1993]**

K=k_r* $E_C\ t_1^3/R$

H^2/Dt_1	**k_r**	H^2/Dt_1	**k_r**
0.4	0.139	10	1.010
0.8	0.270	12	1.108
1.2	0.345	14	1.158
1.8	0.399	16	1.261
2.0	0.445	20	1.430
3.0	0.648	24	1.666
4.0	0.635	32	1.810
5.0	0.713	40	2.025
6.0	0.783	48	2.220
8.0	0.903	56	2.400

Relative Stiffness(K)= $k_r\ E_C\ t_1^3/R$ (3.33)

$M.F_W = k_1\ E_C\ t_1^3/H$ (3.34)

where:

E_C = the modulus of elasticity of concrete. (KN/m^2);
t_1= the thickness of the circular tank wall, (m);
R= the radius of the tank, (m).

The relative stiffness and fixed-end moment ($M.F_F$) of a circular plate are equal to **[PCA, 1993]**:

Relative Stiffness(K)= $0.104\ E_C\ t_3^3/\ R$ (3.35)

$M.F_F = -(0.125\ p_f\ R^2)$ (3.36)

where:

t_3= the thickness of the floor slab, (m).
p_f= the distributed load on the floor slab. KN/m^2;

The ring tension and bending moment throughout the height of the wall are investigated in two steps. First, the base of the wall is assumed fixed, and second, a moment equal to the induced moment is applied at the base. The results are then combined to obtain the actual moment between the wall and the floor slab and actual ring tension along the height of the wall **[PCA, 1993]**.

The moments in the vertical wall strips for the first part are computed by multiplying the lateral load from water pressure ($p_w H^3$) by the coefficient k_1 from table (3.2). The second part of the moment are computed by multiplying the applied moment at the base by the coefficient k_2 from table (3.3) **[PCA, 1993]**. The results are then combined to obtain the actual moments throughout the vertical wall.

$$M.V_1 = k_1 * w H^3 \tag{3.36}$$

$$M.V_2 = k_2 * MR/ H^2 \tag{3.37}$$

$$M.V = M.V_1 + M.V_2 \tag{3.38}$$

where:

$M.V_1$= the vertical moment in the cylindrical wall with (Fixed base- Free top) due to water pressure, (KN.m);

$M.V_2$= the vertical moment in the cylindrical wall with (Moment applied at the base) due to applied moment at the base, (KN.m);

M.V= resultant vertical moment in the cylindrical wall, (KN.m);

p_w = the lateral water pressure ($\gamma_w H$), (KN/m^2);

Table(3.2): Moment in cylindrical wall (fixed base-free top) [After **PCA, 1993]**.
Mom= k_1 * w H^3

	Coefficient(k_1) at point										
$\frac{H^2}{Dt}$	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
0.4	0	+.0005	+.0014	+.0021	+.0007	-.0042	-.0150	-.0302	-.0529	-.0816	-.1205
0.8	0	+.0011	+.0037	+.0063	+.0080	+.0070	+.0023	-.0068	-.0224	-.0465	-.0795
1.2	0	+.0012	+.0042	+.0077	+.0103	+.0112	+.0090	+.0022	-.0108	-.0311	-.0602
1.6	0	+.0011	+.0041	+.0075	+.0107	+.0121	+.0111	+.0058	-.0051	-.0232	-.0505
2.0	0	+.0010	+.0035	+.0068	+.0099	+.0120	+.0115	+.0075	-.0021	-.0185	-.0436
3.0	0	+.0006	+.0024	+.0047	+.0071	+.0090	+.0097	+.0077	+.0012	-.0119	-.0333
4.0	0	+.0003	+.0015	+.0028	+.0047	+.0066	+.0077	+.0069	+.0023	-.0080	-.0268
5.0	0	+.0002	+.0008	+.0016	+.0029	+.0046	+.0059	+.0059	+.0028	-.0058	-.0222
6.0	0	+.0001	+.0003	+.0008	+.0019	+.0032	+.0046	+.0051	+.0029	-.0041	-.0187
8.0	0	.0000	+.0001	+.0002	+.0008	+.0016	+.0028	+.0038	+.0029	-.0022	-.0146
10.0	0	.0000	.0000	+.0001	+.0004	+.0007	+.0019	+.0029	+.0028	-.0012	-.0122
12.0	0	.0000	-.0000	+.0001	+.0002	+.0003	+.0013	+.0023	+.0026	-.0005	-.0104
14.0	0	.0000	.0000	.0000	.0000	+.0001	+.0008	+.0019	+.0023	-.0001	-.0090
16.0	0	.0000	.0000	-.0001	-.0002	-.0001	+.0004	+.0013	+.0019	+.0001	-.0079

	Coefficient(k_1) at point				
$\frac{H^2}{Dt}$	.80H	.85H	.90H	95H	1.00H
20	+.0015	+.0014	+.0005	-.0018	-.0063
24	+.0012	+.0012	+.0007	-.0013	-.0053
32	+.0007	+.0009	+.0007	-.0008	-.0040
40	+.0002	+.0005	+.0006	-.0005	-.0032
48	.0000	+.0001	+.0006	-.0003	-.0026
56	.0000	.0000	+.0004	-.0001	-.0023

Table(3.3):Moment in cylindrical wall (Moment applied at the base) [After **PCA, 1993]**.

Mom= k_2 * M*R/ H^2

	Coefficient(k_2) at point										
$\frac{H^2}{Dt}$	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
0.4	0	+0.013	+0.051	+0.109	+0.196	+0.296	+0.414	+0.547	+0.692	+0.843	+1.000
0.8	0	+0.009	+0.040	+0.090	+0.164	+0.253	+0.375	+0.503	+0.659	+0.824	+1.000
1.2	0	+0.006	+0.027	+0.063	+0.125	+0.206	+0.316	+0.454	+0.616	+0.802	+1.000
1.6	0	+0.003	+0.011	+0.035	+0.078	+0.152	+0.253	+0.393	+0.570	+0.775	+1.000
2.0	0	-0.002	-0.002	+0.012	+0.034	+0.096	+0.193	+0.340	+0.519	+0.748	+1.000
3.0	0	-0.007	-0.022	-0.030	-0.029	+0.010	+0.067	+0.227	+0.426	+0.692	+1.000
4.0	0	-0.008	-0.026	-0.044	-0.051	-0.034	+0.023	+0.150	+0.354	+0.645	+1.000
5.0	0	-0.007	-0.024	-0.045	-0.061	-0.057	-0.015	+0.095	+0.296	+0.606	+1.000
6.0	0	-0.005	-0.018	-0.040	-0.058	-0.065	-0.037	+0.057	+0.252	+0.572	+1.000
8.0	0	-0.001	-0.009	-0.022	-0.044	-0.068	-0.062	+0.002	+0.178	+0515	+1.000
10.0	0	0.000	-0.002	-0.009	-0.028	-0.053	-0.067	-0.031	+0.123	+0.467	+1.000
12.0	0	0.000	0.000	-0.003	-0.016	-0.040	-0.064	-0.049	+0.081	+0.424	+1.000
14.0	0	0.000	0.000	0.000	-0.008	-0.029	-0.059	-0.060	+0.048	+0.387	+1.000
16.0	0	0.000	0.000	+0.002	-0.003	-0.021	-0.051	-0.066	+0.025	+0.354	+1.000

	Coefficient(k_2) at point				
$\frac{H^2}{Dt}$	.80H	.85H	.90H	95H	1.00H
20	-0.015	+0.095	+0.296	+0.606	+1.000
24	-0.037	+0.057	+0.250	+0.572	+1.000
32	-0.062	+0.002	+0.178	+0.515	+1.000
40	-0.067	-0.031	+0.123	+0.467	+1.000
48	-0.064	-0.049	+0.081	+0.424	+1.000
56	-0.059	-0.060	+0.048	+0.387	+1.000

M= the moment applied at the base between the wall and the floor slab determined by the moment distribution,

The ring tension in the horizontal wall strips for the first part is computed by multiplying the lateral load from water pressure (wH^3) by the coefficient k_3 from table (3.4). The second part of the ring tension are computed by multiplying the applied moment at the base by the coefficient k_4 from table (3.5) **[PCA, 1993]**. The results are then combined to obtain the actual rings tension throughout the horizontal direction of the wall.

$$T_1 = k_3 * w H^2 R \quad (3.39)$$

$$T_2 = k_4 * MR/H^2 \quad (3.40)$$

$$T = T_1 + T_2 \quad (3.41)$$

Table(3.4): Tension in circular rings wall (fixed base-free top) [After **PCA, 1993]**
$T=k_3 * w H^2 R$

$\frac{H^2}{Dt}$	Coefficient(k_3) at point										
	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
0.4	+0.149	+0.134	+0.120	+0.101	+0.082	+0.066	+0.049	+0.029	+0.014	+0.004	0
0.8	+0.263	+0.239	+0.215	+0.190	+0.160	+0.130	+0.096	+0.063	+0.034	+0.010	0
1.2	+0.283	+0.271	+0.254	+0.234	+0.209	+0.180	+0.142	+0.099	+0.054	+0.016	0
1.6	+0.265	+0.268	+0.268	+0.266	+0.250	+0.226	+0.185	+0.134	+0.075	+0.023	0
2.0	+0.234	+0.251	+0.273	+0.285	+0.285	+0.274	+0.232	+0.172	+0.104	+0.031	0
3.0	+0.134	+0.203	+0.267	+0.322	+0.357	+0.362	+0.330	+0.262	+0.157	+0.052	0
4.0	+0.067	+0.164	+0.256	+0.339	+0.403	+0.429	+0.409	+0.334	+0.210	+0.073	0
5.0	+0.025	+0.137	+0.245	+0.346	+0.428	+0.477	+0.469	+0.398	+0.259	+0.092	0
6.0	+0.018	+0.119	+0.234	+0.344	+0.441	+0.504	+0.514	+0.447	+0.301	+0.112	0
8.0	-0.011	+0.104	+0.218	+0.335	+0.443	+0.534	+0.575	+0.530	+0.381	+0.151	0
10.0	-0.011	+0.098	+0.208	+0.323	+0.437	+0.542	+0.608	+0.589	+0.440	+0.179	0
12.0	-0.005	+0.097	+0.202	+0.312	+0.429	+0.543	+0.628	+0.633	+0.494	+0.211	0
14.0	-0.002	+0.098	+0.200	+0.306	+0.420	+0.539	+0.639	+0.666	+0.541	+0.241	0
16.0	0.000	+0.099	+0.199	+0.304	+0.412	+0.531	+0.641	+0.687	+0.582	+0.265	0

$\frac{H^2}{Dt}$	Coefficient(k_3) at point				
	.75H	.80H	.85H	90H	.95H
20	+0.716	+0.654	+0.520	+0.325	+0.115
24	+0.746	+0.702	+0.577	+0.372	+0.137
32	+0.782	+0.768	+0.663	+0.459	+0.182
40	+0.800	+0.805	+0.731	+0.530	+0.217
48	+0.791	+0.828	+0.785	+0.593	+0.254
56	+0.763	+0.838	+0.824	+0.636	+0.285

where:

T_1= the ring tension in the circular wall with (Fixed base- Free top) due to water pressure, (KN);

T_2= the ring tension in the circular wall with (Moment applied at the base) due to applied moment at the base, (KN);

T= resultant ring tension in the circular wall, (KN).

Table(3.5): Tension in circular rings (Moment applied at the base) [After PCA, 1993].
$T=k_4 * MR/H^2$

$\frac{H^2}{Dt}$	Coefficient(k_4) at point										
	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
0.4	+2.70	+2.50	+2.30	+2.12	+1.91	+1.69	+1.41	+1.13	+0.80	+0.44	0
0.8	+2.02	+2.06	+2.10	+2.14	+2.10	+2.02	+1.95	+1.75	+1.39	+0.80	0
1.2	+1.06	+1.42	+1.79	+2.03	+2.46	+2.65	+2.80	+2.60	+2.22	+1.37	0
1.6	+0.12	+0.79	+1.43	+2.04	+2.72	+3.25	+3.56	+3.59	+3.13	+2.01	0
2.0	-0.68	+0.22	+1.10	+2.02	+2.90	+3.69	+4.30	+4.54	+4.08	+2.75	0
3.0	-1.78	-0.71	+0.43	+1.60	+2.95	+4.29	+5.66	+6.58	+6.55	+4.73	0
4.0	-1.87	-1.00	-0.08	+1.04	+2.47	+4.31	+6.34	+8.19	+8.82	+6.81	0
5.0	-1.54	-1.03	-0.42	+0.45	+1.86	+3.93	+6.60	+9.41	+11.03	+9.02	0
6.0	-1.04	-0.86	-0.59	-0.05	+1.21	+3.34	+6.54	+10.28	+13.08	+11.41	0
8.0	-0.24	-0.53	-0.73	-0.67	-0.02	+2.05	+5.87	+11.32	+16.52	+16.06	0
10.0	+0.21	-0.23	-0.64	-0.94	-0.73	+0.82	+4.79	+11.63	+19.48	+20.87	0
12.0	+0.32	-0.05	-0.46	-0.96	-1.15	-0.18	+3.52	+11.27	+21.80	+25.73	0
14.0	+0.26	+0.04	-0.28	-0.76	-1.29	-0.87	+2.29	+10.55	+23.50	+30.34	0
16.0	+0.22	+0.07	-0.08	-0.64	-1.28	-1.30	+1.12	+9.67	[illegible]	+34.65	0

$\frac{H^2}{Dt}$	Coefficient(k_4) at point				
	.75H	.80H	.85H	90H	.95H
20	+15.30	+25.9	+36.9	+43.3	+35.3
24	+13.20	+25.9	+40.7	+51.8	+45.3
32	+8.10	+23.2	+45.9	+65.4	+63.6
40	+3.28	+19.2	+46.5	+77.9	+83.5
48	-0.70	+14.1	+45.1	+87.2	+103.0
56	-3.40	+9.2	+42.2	+94.0	+121.0

The final moments at the circular floor slab are determined by adding a quantity equal to the applied moment at the base divided by the term ($p_f R^2$) to the coefficients of circular plate with fixed edges; the final moments are as follows [**PCA, 1993]**:

$$M_{RS}=(-0.125 + M/p_f R^2)\ p_f R^2 \quad (3.42)$$

$$M_{TS}= (-0.025+M/p_f R^2)\ p_f R^2 \quad (3.43)$$

$$M_C= (0.075+ M/p_f R^2)\ p_f R^2 \quad (3.44)$$

where:

M_{RS}=the final radial edge moment of the circular floor slab, (KN.m/m);

M_{TS}= the final tangential edge moment of the circular floor slab, (KN.m/m);

M_{RC}= the final tangential and radial moment at the center of circular floor slab, (KN.m/m).

If the tank is rigidly covered with a concrete roof slab as . The ring tension and bending moment throughout the height of the wall are investigated for three steps. First, the base of the wall is assumed fixed, second, a moment equal to the induced moment is applied at the base, and third, a moment equal to the induced moment is applied at the top between the circular roof slab and the cylindrical wall. The results are then combined to obtain the actual moment between the wall, the roof slab, and the floor slab and actual ring tension along the height of the wall **[PCA, 1993]**.

The determination of the vertical moments and the rings tension in the circular wall due to water pressure and the moment applied at the base are the same fashion of Eqs. (3.36), (3.37), (3.39), and (3.40), respectively, while the vertical moments and the rings tension in the cylindrical wall due to the moment applied at the top are determined by using the coefficients k_2, k_4 from tables

(3.3) and (3.5) considering (0.0H) is the bottom of the wall and (1.0 H) is the top of the wall **[PCA, 1993]**.

$M.V_3 = k_2 * MR/ H^2$ (3.45)

$T_3 = k_4 * MR/H^2$ (3.46)

where:

$M.V_3$= the vertical moment in the cylindrical wall with (Moment applied at the top), (KN.m);

T_3= the ring tension in the cylindrical wall with (Moment applied at the top), (KN);

The results of Eqs. (3.45) and (3.46) are then combined with Eqs. (3.38) and (3.41), respectively, to obtain the actual vertical moments and the actual horizontal rings tension throughout the cylindrical wall.

$M.V= M.V_1 + M.V_2 + M.V_3$ (3.47)

$T= T_1 + T_2 + T_3$ (3.48)

The final moments at the circular roof slab are determined with the same fashion of the moments of the floor slab of Eqs. (3.42), (3.43), and (3.44) by adding a quantity equal to the applied moment at the top divided by the term ($p_r R^2$) to the coefficients of circular plate with fixed edges [**PCA, 1993]**.

where:

p_r= the distributed load (live + dead) on the roof slab. (KN/m^2).

When the circular wall is continuous with the floor slab, the base of the wall is subjected to shear force; the shear force at the base of the wall may be computed as the sum of the product of coefficients depending on the value of

(H^2/Dt_1) taken from table (3.6). The first coefficient k_S is of fixed base with triangular load multiplied by (p_w H^2), and the other coefficient k_m is of moment at edge multiplied by the moment at the base of the circular wall [**PCA, 1993]**.

$$V_{S1}= k_S * w\,H^2 \tag{3.49}$$

$$V_{S2}= k_m * M \tag{3.50}$$

$$V_S= V_1 + V_2 \tag{3.51}$$

where:

V_{S1}= the shear force at the cylindrical wall base due to water pressure, (KN);

V_{S1}= the shear force at the cylindrical wall base due to applied moment, (KN).

V_S= the resultant shear force at the cylindrical wall base, (KN).

Table (3.6): Shear at base of cylindrical wall [After **PCA, 1993].**
Positive sign indicates shear acting inward

	k_S			k_m
H^2/Dt	Triangular loaded Fixed base	Triangular or Rectangular loaded hinged base	Rectangular loaded Fixed base	Moment at adge
0.4	+0.436	+0.755	+0.245	-1.58
0.8	+0.374	+0.552	+0.234	-1.75
1.2	+0.339	+0.460	+0.220	-2.00
1.8	+0.317	+0.407	+0.204	-2.28
2.0	+0.299	+0.370	+0.189	-2.57
3.0	+0.262	+0.310	+0.158	-3.18
4.0	+0.236	+0.271	+0.137	-3.66
5.0	+0.213	+0.243	+0.121	-4.10
6.0	+0.197	+0.222	+0.110	-4.49
8.0	+0.174	+0.193	+0.096	-5.18
10	+0.158	+0.172	+0.087	-5.81
12	+0.145	+0.158	+0.079	-6.36
14	+0.135	+0.147	+0.073	--6.88
16	+0.127	+0.137	+0.066	-7.36
20	+0.114	+0.122	+0.062	-8.20
24	+0.102	+0.111	+0.055	-8.94
32	+0.089	+0.096	+0.048	-10.36
40	+0.080	+0.086	+0.043	-10.62
48	+0.072	+0.079	+0.039	-12.76
56	+0.067	+0.074	+0.038	-13.76

3.4.3.2: The underground circular tank

When the tank is built underground, the tank walls must be investigated for both internal and external pressures.

In the case when only internal water pressure is acting from inside, the ring tension and the vertical moments throughout the cylindrical wall are determined in the same fashion of on- ground tanks, and the Eqs. from (3.33) to (3.50) are used. And in the case when only earth pressure acting from outside, the density of water (γ_w) is replaced by the density of dry soil (γ_S) to determine the ring compression and the vertical moment in the circular wall, and also using the Eqs. from (3.33) to (3.50) [**PCA, 1993]**.

3.4.4: Design of the concrete tank sections.

Typically, in the design of reinforced concrete members, the tensile strength of concrete is ignored. Any significant cracking in a tank containing liquid is unacceptable. For this reason, it must be assured that the stress in concrete from ring tension and from bending is kept at a minimum to prevent excessive cracking. The design of these structures requires that the attention be given not only to strength requirements, but to serviceability requirements as well. A properly designed tank must be able to withstand the applied loads without cracks that would permit leakage. The goal of providing a structurally sound tank that will not leak is achieved by providing the proper amount and distribution of reinforcement, the proper spacing and detailing of construction joints, and the use of quality concrete placed using proper construction practices [**PCA, 1993].**

The design of a tank is based on the environmental provisions and then the structural provisions **[PCA, 1996]**. However, a "good" design should take into consideration the overall cost of the designed structure.

Two approaches currently exist for the design of reinforced concrete members:

1) The strength design.

2) Allowable stress design (referred to in ACI 318-2005 Code as Appendix B, as the alternate design method) **[PCA, 1993]**.

In the present research the allowable (working stress) design method is adopted. In this method, members are designed to carry service loads (load factors and ϕ are taken as unity) under the straight-line (elastic) theory of stress and strain. (Because of creep in the concrete, only stresses due to short-time loading can be predicated with reasonable accuracy by this method) **[Fredrick. et al., 2001].**

According to **[Nilson et al. 2004]**, the concrete stress with maximum value f_C at the outer edge is distributed linearly as shown in figure (3.9), and the entire steel area As is subjected to the stress f_S, correspondingly. The total compression force C and the total tension force T are:

$$C=1/2\ f_C\ b\ kd \tag{3.52}$$

$$T= As\ f_S \tag{3.53}$$

The requirement that these two forces be equal numerically has been taken care of by the manner in which the location of the neutral axis has been determined.

Equilibrium requires that the couple constituted by the two forces C and T be equal numerically to the external bending moment M. Hence, taking moment about C gives:

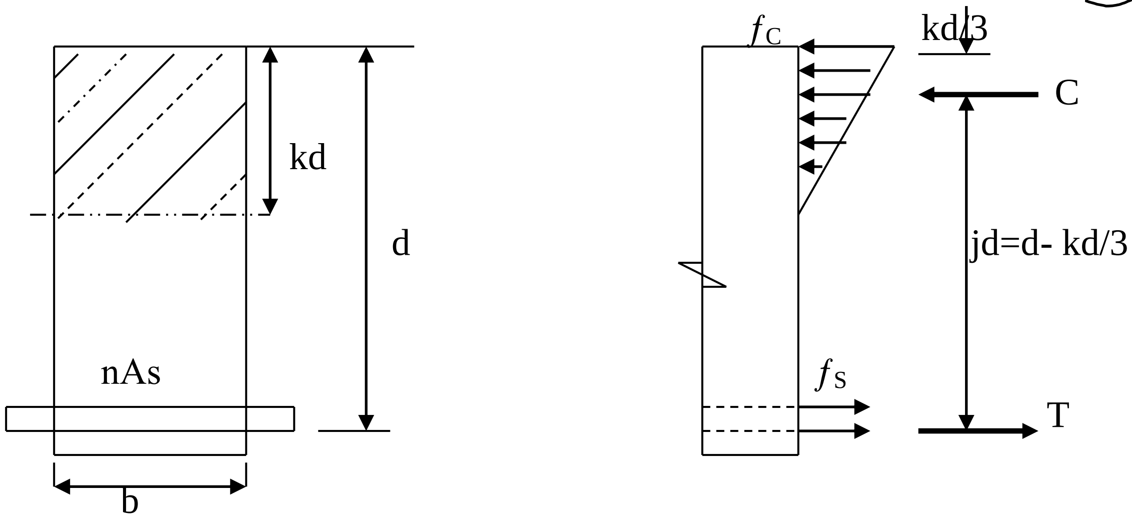


Figure (3.9) cracked transformed section [After **Nilson et al. 2004]**

$$M_S = As\ f_S\ jd \qquad (3.54)$$

Conversely, taking moment about T gives:

$$M_C = 1/2\ f_C\ k\ j\ b\ d^2 \qquad (3.55)$$

$$k = 1 / (1 + (f_S / n\ f_C) \qquad (3.56)$$

$$j = 1 - k/3 \qquad (3.57)$$

$$n = E_S / E_C \qquad (3.58)$$

$$E_C = 4700 \sqrt{f_C'} \qquad (3.59)$$

where:

M_C= compression capacity, (KN.m);

M_S= tension capacity, (KN.m);

jd = the internal lever arm between C and T, (m);

kd= depth of the neutral axis, (m);

f_S= allowable tensile stress in steel, (kN/m^2);

f_C= allowable compression stress in concrete, (kN/m^2);

n = modular ratio;

E_C= modulus of elasticity of concrete, (MN/m^2);

E_S= modulus of elasticity of steel which is equal to (200000*10^3 kN/m^2).

b= unit width of concrete section (m);

d= effective depth of concrete section measured from top to the center of steel reinforcement, (m).

To design a rectangular tank wall subjected to bending moment(M) and direct tension (T) as shown in Fig. (3.10), the net bending moment, either negative or positive, shall be equal to **[Syal et al. 2005]**:

$$B.M_{net} = (M - T.e). \tag{3.60}$$

$$e = (t / 2) - cv \tag{3.61}$$

where:

$B.M_{net}$= net bending moment at the center of reinforcement, (KN.m);

M = the bending moment (negative or positive) at the center of tanks wall, (KN.m);

T = the direct tension in the tank wall, (KN);

e = the effective distance from center of the wall to the center of reinforcement, (m);

t = thickness of the tank wall, (m);

cv = the effective cover of the tank wall, (m).

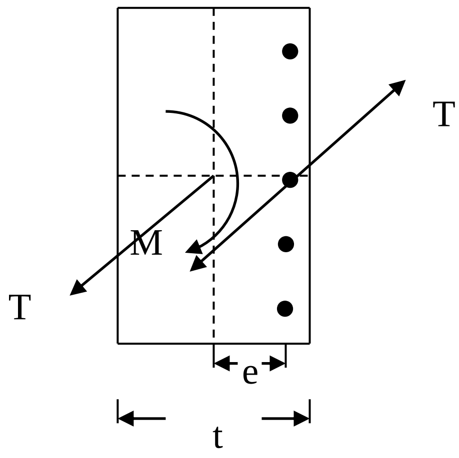


Fig.(3.10). Rectangular tank wall subject to moment and direct tension

[After **Syal et al. 2005]**

CHAPTER FOUR

THE OPTIMAL DESIGN OF REINFORCED-CONCRETE TANKS

4.1: THE CASE STUDY

This research aims at an optimum design of some selected types of reinforced concrete tanks. The following design aspects have been adopted in this research.

[A] With respect to volume :

(i). A relatively small tank, say (V= 50) m^3.

(ii) A moderately medium tank, say (V= 250) m^3.

(iii) A medium tank, say (V= 500) m^3.

(iv) A relatively large tank, say (V= 750) m^3.

(v) A large tank, say (V= 1000) m^3.

[B] With respect to level state :

(i) On-ground tanks.

(ii) Buried tanks.

. (a) Partially buried.

(b) Fully buried.

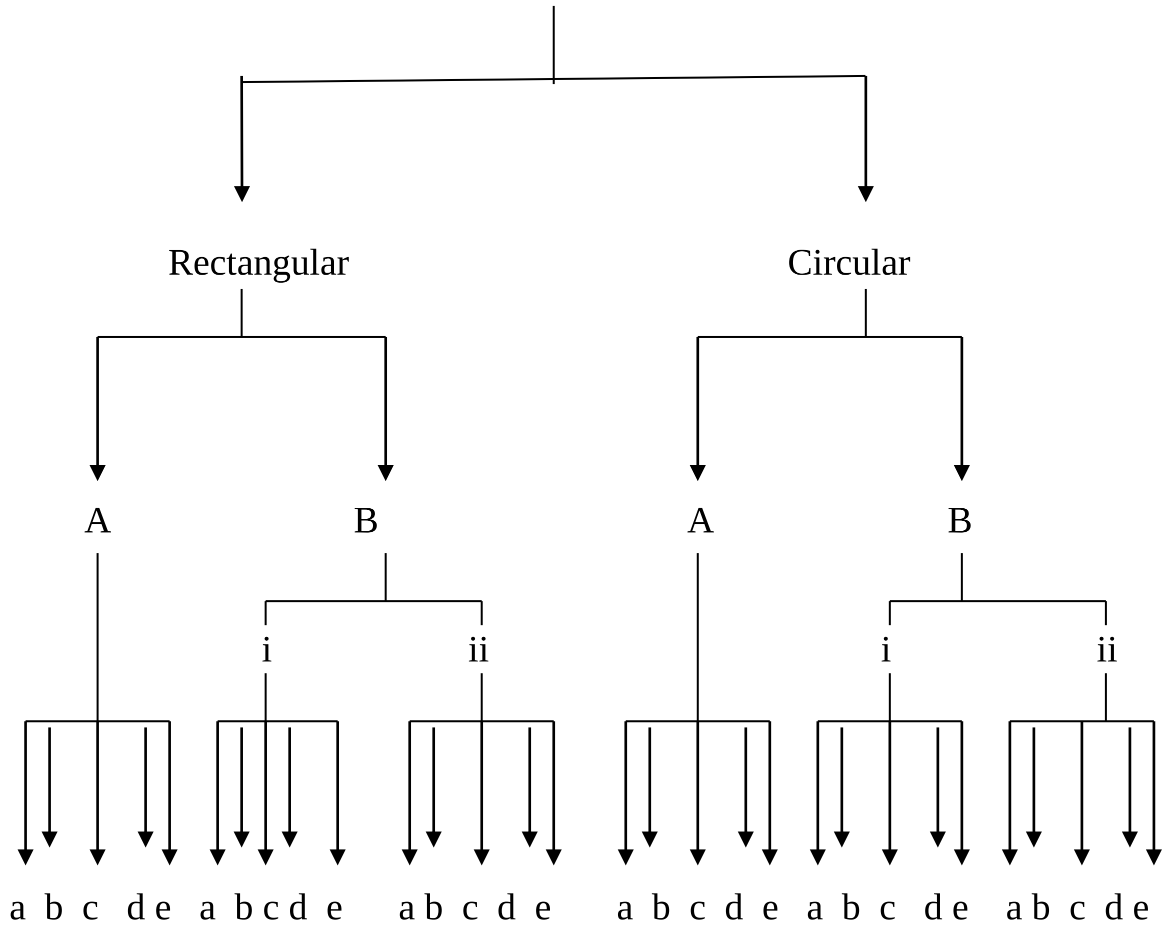
[C] With respect to shape:

(i) Rectangular tanks, with the squared as a special case.

(ii) Circular tanks

Consequently, the investigated cases would be as summarized in Fig. (4.1).

REINFORCED CONCRETE TANKS



A= On-ground

B= Buried

i=Fully buried

ii=Partially buried

a= Tank volume (V) is (50 m^3)

b= Tank volume (V) is (250 m^3)

c= Tank volume (V) is (500 m^3)

d= Tank volume (V) is (750 m^3)

e= Tank volume (V) is (1000 m^3)

Figure (4.1) Investigated cases of the reinforced concrete tanks undertaken

4.2: FORMULATION OF THE OPTIMIZATION MODEL

The general outline of any optimization process constitutes the following basic aspects:

1. Outlining the general objective, recognizing the scope of the involved problem, and identifying all the effective parameters involved.
2. Of the identified parameters, specifying the decision variables; possibly only the basic ones.
3. Formulation of the objective function as a function of the specified decision variables.
4. Formulation of the constraints imposed on the investigated system (problem) as functions of the decision variables.
5. Solving the formulated problem by an appropriate solution procedure in order to identify the optimal solution(s).

4.2.1: The general objective of the optimization process

As mentioned in sec. (4.1), the general objective of the research is to obtain the optimal design of some selected types of reinforced-concrete water-storage tanks.

A reinforced-concrete tank consist of the walls, the floor slab, and the roof slab, when applicable. The length, width, and the height of the tank, the thicknesses of the roof slab, the walls, and the floor slab, are usually the design variables.

4.2.2: The decision variables

[A] **The parallelepiped (rectangular) reinforced-concrete tanks**

A typical reinforced concrete tank in the form of a parallelepiped is shown in Fig. (4.2 - a). The design variables of such a tank are (X_1= L), (X_2= B), (X_3= H), (X_4= t_1), (X_5= t_2), (X_6= t_3), (X_7= t_4), where:

L = length of the tank, (L);

B = width of the tank, (L);

H = height of the tank, (L);

t_1 = thickness of the long walls of the tank, (L);

t_2 = thickness of the short walls of the tank, (L);

t_3 = thickness of the base slab, (L); and

t_4 = thickness of the roof slab for a closed concrete tank, (L).

The optimum solution will give the optimum shape. If (L > B), then the optimum shape shall be denoted as a rectangular shape. A special case of the parallelepiped tank is the one which has its top view as a square

[B] : The cylindrical (circular) reinforced-concrete tanks

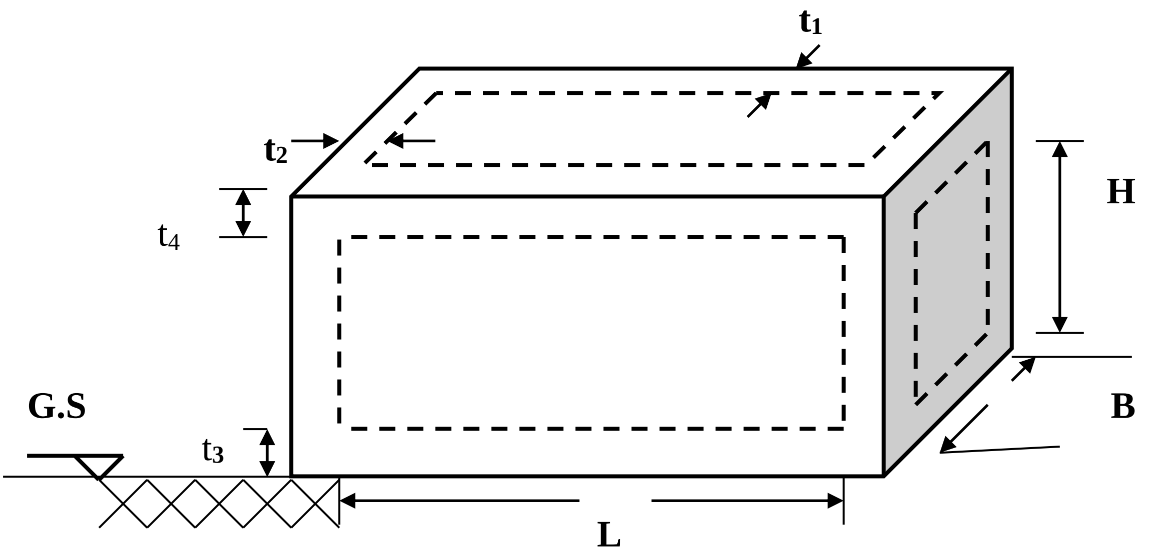
A typical concrete tank of such a form is shown in Fig.(4.2 – b). In this case, the design variables would be : (X_1= D), (X_2= H), (X_3= t_1), (X_4= t_3), (X_5= t_4), where :

D= internal diameter of the tank, (L);

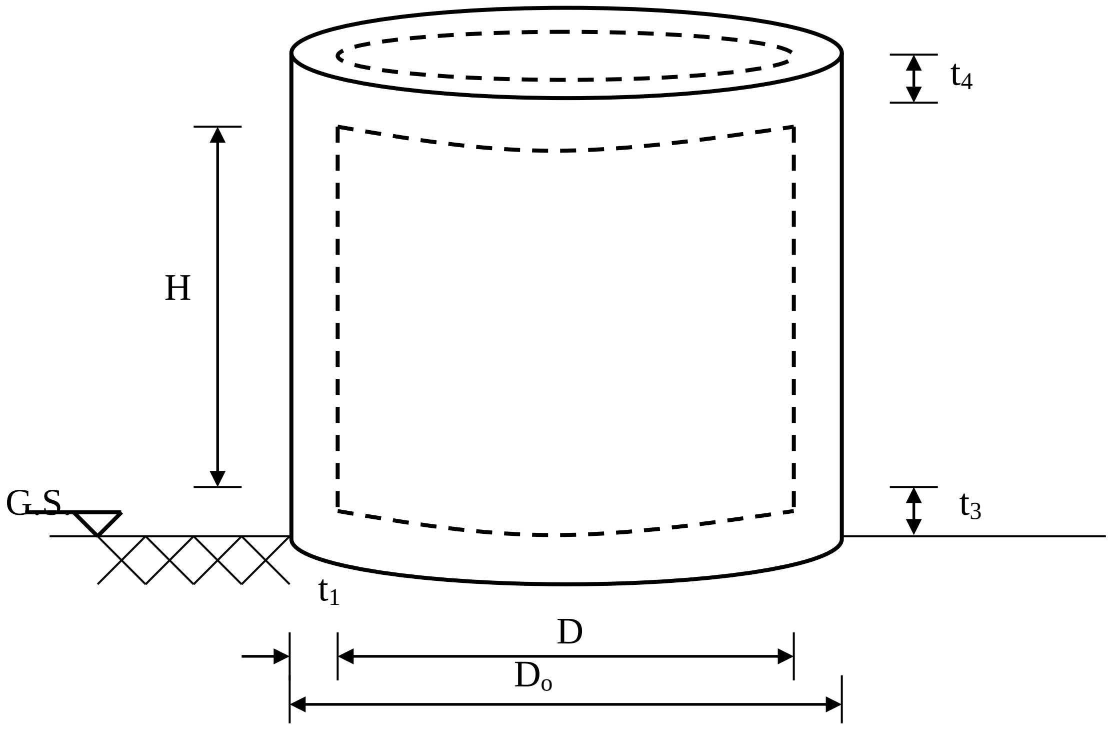
t_1= thickness of the cylindrical shell, (L); other parameter are as defined before.

4.2.3: The objective function

The cost objective function (Z) of the present research involves the cost of bedding, asphalt or water proofing layer, materials of the tank, excavation and fillworks (for buried and partially buried tanks), and water stop material.



[a]:A typical parallelepiped (rectangular) tank.



[b]:A typical cylindrical (circular) tank.

Fig (4.2) The considered shapes of a reinforced concrete tank.

The general procedure to arrive at the aforementioned itemized costs for each of the mentioned types of tanks are discussed hereinafter.

A reasonable and practical survey in the design and construction of a reinforced – concrete tank would delineate the following constituents of its cost:

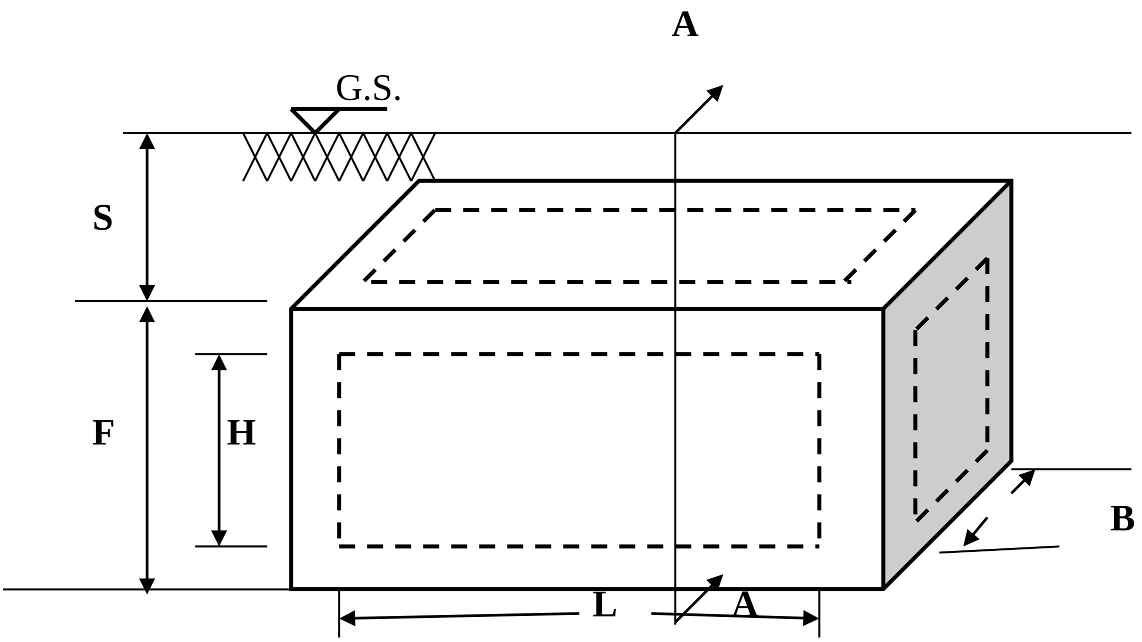
1. Excavation (for buried and partially buried tanks).
2. Bedding (blinding) layer.
3. Asphalt layer.
4. The constituents of the process of constructing the body of the tank, which involves.
   I. Frameworks
   II. Reinforcing
   III. Water stop material
   IV. Concrete materials.
5. Fill work (for buried and partially buried tanks) .

<u>4.2.3.1: The objective function of the rectangular reinforced concrete tank</u>

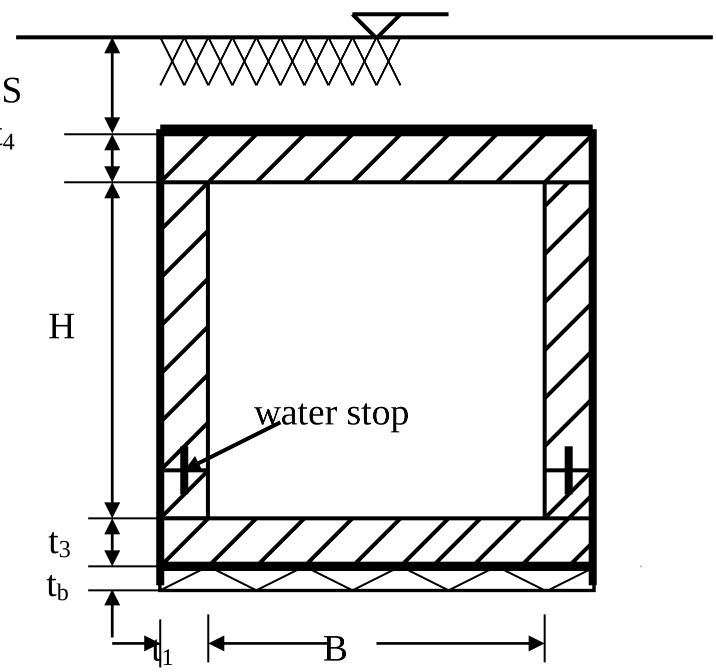
The general case in this respect is the underground rectangular concrete tank. According to the details shown in figure (4.3), the details of the cost calculations are given in the following sub-items.

<u>I):The objective function of rectangular fully-buried reinforced concrete tank</u>

According to details shown in Fig (4.3), the overall cost objective function (ZT_1) of this type of tank shall be as follows:



[a]:The isometric view



[b]:Section(A-A)

Figure(4.3)Typical works of a rectangular fully-buried reinforced concrete tank.

[A] <u>Cost of excavation, (Z_{R1})</u>

The cost of excavation can be, in general, determined as follows.

$$Z_{R1}= C_1V_{R1} \tag{4.1}$$

where :

C_1= cost of excavation per unit volume, (ID/ m^3);

V_{R1}= volume of exaction, (m^3).

The volume of excavation can be calculated as follows:

$$V_{R1} = (B+2t_1)(L+2t_2)(S+F) + 2(B+2t_1)(S+F) + 2(L+2t_2)(S+F) \tag{4.2}$$

where :

S= height of fillworks above the tank, (m);

F= (H + t_3 + t_4 + t_b), (m)

Consequently :

$$Z_{R1}= C_1[(S+F)BL + (2t_2+2)(S+F)B + (2t_1+2)(S+F)L + 4t_1t_2(S+F) + 4(t_1+t_2)(S+F)] \tag{4.3}$$

[B]: <u>Cost of bedding, (Z_{R2})</u>

Bedding is a layer, usually of plain concrete of uniform thickness, under the tank floor to form a base under the tank. The cost of bedding can be determined as:

$$ZR_2= C_2VR_2 \tag{4.4}$$

where:

C_2= cost of plain concrete for bedding per unit volume, (ID/m^3);

VR_2= volume of the bedding materials, (m^3).

The volume of the bedding materials (VR_2) is :

$VR_2 = (B + 2t_1)(L + 2t_2)(t_b)$ (4.5)

where :

t_1= thickness of long wall, (m);

t_2= thickness of short wall, (m)

t_b= thickness of bedding layer, (m).

Then:

$V_{R2} = (BL + 2Bt_2 + 2Lt_1 + 4t_1t_2)\ t_b$ (4.6)

Thus:

$ZR_2 = C_2\ [(t_b)BL + (2\ t_2t_b)B + (2\ t_1t_b)L + 4\ t_1t_2t_b]$ (4.7)

[C]: <u>Cost of water proofing membrane (sliding layer), (Z_{R3})</u>

The sliding layer is usually of a bitumen paper or other suitable material is a layer between the bedding and the tank floor to destroy the bond between them **[syal et al. 2005]**. For buried concrete tanks consideration should be given to installation of an impermeable roof covering_**[SCDPHDWS, 2005].** The cost of the sliding layer can be calculated as follows:

$ZR_3 = C_3AR_3$ (4.8)

where:

C_3= cost of the sliding layer per unit area, (ID/m²);

AR_3= area of sliding layer, (m²).

The area of the sliding layer is:

$AR_3 = (1+X)(B+2t_1)(L+2t_2) + 2X(BH + LH)$ (4.9)

Thus :

$$ZR_3 = C_3[(1+X)(BL + (2t_2)B + (2t_1)L + 4t_1t_2) + 2IBH + 2ILH] \qquad (4.10)$$

Where:

X= constant used to recognize between coverless and closed tanks; its value is (X=1) for a closed tank and (X=0)for a coverlrss tank

[D]: <u>Cost of the body of the tank, (ZR4)</u>

As mentioned in item (4.2.3), the cost of reinforced concrete materials involves three parts which are:

<u>(1):Cost of formworks, (Z_f)</u>.

The cost of the formworks of the concrete tank is:

$$Z_f = C_f A_f \qquad (4.11)$$

where:

C_f= cost of formworks per unit area, (ID/ m^2);

A_f= the area of formworks of rectangular tank, (m^2).

The area of formworks of rectangular concrete tank can be determined as:

$$A_f = [2(B+2t_1)(H+Xt_4+t_3+t_b) + 2(L+2t_2)(H+Xt_4+t_3+t_b) + (2BH) + (2LH) + (XBL)] \qquad (4.12)$$

Thus :

$$Z_f = C_f[4BH + 4LH + XBL + (2(t_3+t_b+Xt_4)L + (2(t_3+t_b+Xt_4)B + 4(t_1+t_2)H + 4(t_1+t_2)(t_b+t_3\ Xt_4)] \qquad (4.13)$$

<u>(2): Cost of reinforcement, (Z_R)</u>.

The cost of reinforcement of the concrete tank would be:

$$Z_R = C_R W_R \qquad (4.14)$$

where:

C_R= cost of reinforcement per unit weight, (ID/ kg);

W_R= weight of reinforcement, (kg).

The weight of reinforcement (W_R) can be calculated as:

$$W_R= 2BH(As_{W1}+As_{W2}) + 2LH(As_{W3} + As_{W4}) + BL(As_F +X\ As_R)w_S \quad (4.15)$$

where:

w_S= weight of reinforcement per cubic meter, (kg/m^3);

As_{W1}= Area of horizontal reinforcement in the short walls , (m^2/m);

As_{W2}=Area of vertical reinforcement in the short walls , (m^2/m);

As_{W3}= Area of horizontal reinforcement in the long walls , (m^2/m);

As_{W2}=Area of vertical reinforcement in the long walls , (m^2/m);

As_F= Area of floor slab reinforcement, (m^2/m).

As_R= Area of roof slab reinforcement, (m^2/m).

Thus :

$$Z_R= C_R w_S[2BH(As_{W1}+As_{W2}) + 2LH(As_{W3} + As_{W4}) + BL(As_F +X\ As_R) \quad (4.16)$$

(3):Cost of concrete materials, (Z_{co})

The cost of concrete materials of the concrete tank is:

$$Z_{co}= C_{co}V_{co} \quad (4.17)$$

where:

C_{co}= cost of concrete materials per unit volume, (ID/ m^3);

V_{co}= volume of concrete materials, (m^3).

The volume of concrete materials (Vco) can be calculated as:

$$V_{co}= [(B+2t_1) (L+2t_2)t_3 + 2(B+2t_1) H\ t_2 + 2LH\ t_1] + (B+2t_1) (L+2t_1)Xt_4] \quad (4.18)$$

Thus:

$$Z_{co}= C_{co} [(2t_1)LH + (Xt_4+t_3)BL + (2t_2)BH +(2t_1(Xt_4+t_3))L + (2t_2(Xt_4+t_3))B + (4t_1t_2)H +4t_1t_2 (Xt_4+t_3)] \quad (4.19)$$

[E] Cost of water stop material, (Z_{R5})

$$Z_{R5} = C_5\, S_{R5} \qquad (4.20)$$

where :

C_5= cost of water stop material per unit meter, (ID/m);

S_{R5}= length of the water stop material, (m).

Then :

$$S_{R5} = 2((B+t_1) + (L+t_2)) \qquad (4.21)$$

Thus :

$$Z_{R5} = 2C_5\, [B + L + t_1 + t_2] \qquad (4.22)$$

[F] Cost of fill work, (Z_{R6})

The cost of fillwork can be in general determined as follows:

$$Z_{R6} = C_6\, V_{R6} \qquad (4.23)$$

where :

C_6= cost of earth fillwork per unit volume, (ID/ m^3);

V_{R6}= volume of the earth fillwork over the concrete tank, (m^3).

Then:

$$V_{R6} = [(S)BL + (2t_2+2)S + 2F)\, B + (2t_1+2)S + 2F)L + (S)4t_1t_2 + 4(t_1+t_2)(S + F)] \qquad (4.24)$$

Consequently :

$$Z_{R6} = C_6[(S)BL + (2t_2+2)S + 2F)\, B + (2t_1+2)S + 2F)L + (S)4t_1t_2 + 4(t_1+t_2)(S + F)] \qquad (4.25)$$

All the aforementioned sub – objective cost functions are summarized in Table (4.1).

The total cost (ZT_1) is the summation of all aforementioned sub-cost functions calculated in the previous items and given in Table (4.1). That is :

$$ZT_1 = \sum z_{Ri} \qquad i = 1,2,\ldots,6 \qquad (4.26)$$

Thus, the overall cost objective function may be written in the form:

$$ZT_1=[C2\ t_b+(1+X)C_3+Cco(Xt_4+t_3)+(As_F+XAs_R)\ C_Rw_S+XC_f+C_1(S+F)+C_6s]\ \mathbf{BL}+[(2Cco\ t_1+2C_Rw_S(As_{W3}+As_{W4})+4C_f+2C_3)]\ \mathbf{LH}+[(4C_f+2C_3+2C_Rw_S(As_{W1}+As_{W2})+2Ccot_2)]\ \mathbf{BH}+[(2C_2t_bt_2+2(1+X)C_3t_2+Cco(2t_2(t_3+Xt_4))+2C_f(t_3+t_b+Xt_4))+2C_1+C_1(2t_2+2)(S+F)+C_6(2t_2+2)S+2F)]\ \mathbf{B}+[(2C_2t_bt_1+2(1+X)C_3t_1+Cco(2t_1(t_3+Xt_4))+2C_f(t_3+t_b+Xt_4)+2C_5+C_1(2t_1+2)(S+F)+C_6(2t_1+2)S+2F)]\ \mathbf{L}+[(4Ccot_1t_2+4C_f(t_1+t_2))]\ \mathbf{H}+[(C2t_b+(1+X)C_3+Cco(t_3+Xt_4)+(C_6+C_1)S+C_1F]4t_1t_2+[(S+F)(C_6+C_1)+C_f\ (t_3+t_b+Xt_4)]\ 4(t_1+t_2)+[2C4\ (t_1+t_2)] \quad (4.27)$$

Table (4.1) Summary of final cost objective function, (ZT_1).

Cost function	Terms of the decision variables							
	BL	BH	LH	B	L	H	t_1t_2	t_1+t_2
Z_{R1}	$C_1(S+F)$	–	–	$C_1(S+F)(2t_1+2)$	$C_1(S+F)(2t_1+2)$	–	$4C_1(S+F)$	$4C_1(S+F)$
Z_{R2}	C_2t_b	–	–	$2C_2t_b\ t_2$	$2C_2t_b\ t_1$	–	$4C_2t_b$	–
Z_{R3}	$(1+X)\ C_3$	$2C_3$	$2C_3$	$2(1+X)t_2\ C_3$	$2(1+X)t_1\ C_3$	–	$(1+X)\ 4C_3$	–
Z_F	$X\ C_F$	$4\ C_F$	$4\ C_F$	$2C_F\ (t_b+t_3+Xt_4)$	$2C_F\ (t_b+t_3+Xt_4)$	$4C_F\ (t_1+t_2)$	–	$4C_F\ (t_b+t_3+Xt_4)$
Z_R	$(As_F+XAs_R)\ C_Rw_S$	$(AS_{W1}+AS_{W2})\ 2C_Rw_S$	$(AS_{W3}+AS_{W4})\ 2C_Rw_S$	–	–	–	–	–
Z_{co}	$Cco(Xt4+t_3)$	$2\ C_{co}t_2$	$2\ C_{co}t_1$	$C_{co}(2t_2(t_3+Xt_4)$	$C_{co}(2t_1(t_3+Xt_4)$	$4\ C_{co}t_2\ t_1$	$4C_{co}(t_3+Xt_4)$	–
Z_{R5}	–	–	–	$2\ C_5$	$2\ C_5$	–	–	$2\ C_5$
Z_{R6}	C_6S	–	–	$C_6(S(2t_1+2)+2F)$	$C_6(S(2t_2+2)+2F)$	–	$4C_6S$	$4C_6(S+F)$

From the general objective function equation of the rectangular underground concrete tank, (ZT_1), the objective functions of various types of rectangular and squared concrete tanks can be formulated by assigning the respective values for the parameters (X), (S), and (F) in Eq. (4.27). Such an assignment is indicated in table (4.2),

II): The objective function of rectangular on-ground concrete tank (closed)

Generally, similar to the procedure followed in getting the (ZT_1) and according to table (4.2), the terms (S and F) shall have (0) values, and the overall cost objective function (ZT_2) of this type of tank can be written as follows:

$$ZT_2=[C_2\, t_b+C_3+Cco(Xt_4+t_3)+(As_F+XAs_R)\, C_R w_S+XC_f]\,\mathbf{BL}+[(2Ccot1+2C_R w_S(As_{W3}+As_{W4})+4C_f]\mathbf{LH}+[(4C_f+2C_R w_S(As_{W1}+As_{W2})+2Ccot_2)]\,\mathbf{BH}+[(2C_2t_bt_2+2C_3t_2+Cco(2t_2(t_3+Xt_4))+2C_f(t_3+t_b+Xt_4))+2C_5]\,\mathbf{B}+[(2C_2t_bt_1+2C_3t_1+Cco(2t_1(t_3+Xt_4))+2C_f(t_3+t_b+Xt_4)+2C_5+]\mathbf{L}+[(4Ccot1t_2+4C_f(t_1+t_2))]\,H+[(C_2t_b+C_3+Ccot_3+]\,\mathbf{4t_1t_2}+[C_f\,(t_3+t_b+Xt_4)]\,\mathbf{4(t_1+t_2)}+2C_5\,(t_1+t_2) \quad (4.28)$$

The final cost objective function of the on-ground rectangular reinforced concrete closed tank is given in Table (4.3).

Table (4.2): Cases of the rectangular tank

Type of tank	Type of cover	Term X	Term S	Term F
On-ground	Closed	1	0	0
On-ground	Open	0	0	0
Partially buried	Closed	1	0	(*)
Partially buried	Open	0	0	(*)
Fully-buried	Closed	1	(*)	(*)

(*) A value according to the site condition

Table (4.3) Summary of final cost objective function, (ZT_2).

Cost function	Terms of the decision variables							
	BL	BH	LH	B	L	H	t_1t_2	t_1+t_2
Z_{R1}	–	–	–	–	–	–	–	–
Z_{R2}	C_2t_b	–	–	$2C_2t_b\ t_2$	$2C_2t_b\ t_1$	–	$4C_2t_b$	–
Z_{R3}	C_3	–	–	$2t_2\ C_3$	$2t_1\ C_3$	–	C_3	–
Z_F	$X\ C_F$	$4\ C_F$	$4\ C_F$	$2C_F\ (t_b+t_3+Xt_4)$	$2C_F\ (t_b+t_3+Xt_4)$	$4C_F\ (t_1+t_2)$	–	$4C_F\ (t_b+t_3+Xt_4)$
Z_R	$(As_F+XAs_R)\ C_Rws$	$(A_{SW1}+A_{SW2})\ 2C_Rws$	$(A_{SW3}+A_{SW4})\ 2C_Rws$	–	–	–	–	–
Z_{co}	$Cco(Xt_4+t_3)$	$2\ C_{co}t_2$	$2\ C_{co}t_1$	$C_{co}(2t_2(t_3+Xt_4)$	$C_{co}(2t_1(t_3+Xt_4)$	$4\ C_{co}t_2\ t_1$	$C_{co}(t_3+Xt_4)$	–
Z_{R5}	–	–	–	$2\ C_5$	$2\ C_5$	–	–	$2\ C_5$
Z_{R6}	–	–	–	–	–	–	–	–

<u>III): The objective function of rectangular on-ground reinforced concrete tank (open)</u>

The formulation of the objective function of this type is the same as that of the rectangular aboveground closed tank except giving the term (X) zero value. According to table (4.2) the overall cost objective function(ZT_3) shall be:

$$ZT_3=[C_2\ t_b+C_3+Ccot_3+ (As_F)\ C_Rw_S +]\ BL+[(2Ccot_1+2C_Rw_S(As_{W3}+As_{W4})+ 4C_f]LH+[(4C_f+2C_Rw_S(As_{W1}+As_{W2})+2Ccot_2)]\ BH+[(2C_2t_bt_2+2C_3t_2+ Cco(2t_2t_3)+2C_f(t_3+t_b))+2C_5]\ B + [(2C2t_bt_1+2C_3t_1+2Ccot_1t_3+ 2C_f(t_3+ t_b)+2C_5+]\ L + [(4Ccot_1t_2+4C_f(t_1+t_2))]\ H +[(C2t_b+C_3+Ccot_3+]\ 4t_1t_2 + [C_f\ (t_3+t_b)]\ 4(t_1+t_2) + 2C5\ (t_1+t_2) \quad (4.29)$$

The final cost objective function of the on-ground rectangular reinforced concrete open tank is given in Table (4.4).

Table (4.4) Summary of final cost objective function, (ZT_3).

Cost function	Terms of the decision variables							
	BL	BH	LH	B	L	H	t_1t_2	t_1+t_2
Z_{R1}	–	–	–	–	–	–	–	–
Z_{R2}	C_2t_b	–	–	$2C_2t_b\ t_2$	$2C_2t_b\ t_1$	–	$4C_2t_b$	–
Z_{R3}	C_3	–	–	$2t_2\ C_3$	$2t_1\ C_3$	–	C_3	–
Z_F	$X\ C_F$	$4\ C_F$	$4\ C_F$	$2C_F\ (t_b+ t_3)$	$2C_F\ (t_b+ t_3)$	$4C_F\ (t_1+ t_2)$	–	$4C_F\ (t_b+ t_3)$
Z_R	$(As_F)\ C_Rw_S$	$(As_{W1}+As_{W2})\ 2C_Rw_S$	$(As_{W3}+As_{W4})\ 2C_Rw_S$	–	–	–	–	–
Z_{co}	$Ccot_3$	$2\ C_{co}t_2$	$2\ C_{co}t_1$	$2\ C_{co}t_2\ t_3$	$2\ C_{co}t_1\ t_3$	$4\ C_{co}t_2\ t_1$	$C_{co}t_3$	–
Z_{R5}	–	–	–	$2\ C_5$	$2\ C_5$	–	–	$2\ C_5$
Z_{R6}	–	–	–	–	–	–	–	–

IV):The objective function of rectangular partially-buried reinforced concrete tank (closed)

According to table (4.2), the final formulation of the overall cost objective function of rectangular partially buried closed tank (ZT_4) is :

ZT_4=[$C_2\, t_b + C_3 + C_{co}(Xt_4+t_3) + (As_F + XAs_R)\, C_R w_S + XC_f + C_1 F) +$] **BL** +[($2C_{co}\, t_1 + 2C_R w_S(As_{W3}+As_{W4}) + 4C_f + 2IC_3$)] **LH** + [($4C_f + 2C_3 + 2C_R w_S(As_{W1}+As_{W2}) + 2C_{co}t_2$)] BH + [($2C_2 t_b t_2 + 2C_3 t_2 + C_{co}(2t_2(Xt_4+t_3)) + 2C_f(t_3+t_b+Xt_4)) + 2C_5 + C_1(2t_2+2)\, F + C_6 2F$)] **B** + [($2C_2 t_b t_1 + 2C_3 t_1 + 2C_{co} t_1(Xt_4+t_3)) + 2C_f(t_3+t_b+Xt_4) + 2C_5 + C_1(2t_1+2)F + C_6 2F$)]**L**+[($4C_{co}t_1 t_2 + 4C_f(t_1+t_2)$)]H+[($(C_2 t_b + C_3 + C_{co}(Xt_4+t_3) + C_1 F$] $\mathbf{4t_1t_2}$ + [$F\,(C_6 + C_1) + C_f\,(t_3+t_b+Xt_4)$] $4\mathbf{(t_1+t_2)}$ + $2C_5\,(t_1+t_2)$ (4.30)

The final cost objective function of the partially buried rectangular reinforced closed concrete tank is given in Table (4.5).

Table (4.5) Summary of final cost objective function, (ZT_4).

Cost function	Terms of the decision variables							
	BL	BH	LH	B	L	H	t_1t_2	t_1+t_2
Z_{R1}	$C_1\,F$	–	–	$C_1F\,(2t_2+2)$	$C_1F\,(2t_1+2)$	–	C_1F	$4C_1F$
Z_{R2}	C_2t_b	–	–	$2C_2t_b\,t_2$	$2C_2t_b\,t_1$	–	$4C_2t_b$	–
Z_{R3}	C_3	$2IC_3$	$2C_3$	$2t_3\,C_2$	$2t_1\,C_3$	–	C_3	–
Z_F	$X\,C_F$	$4\,C_F$	$4\,C_F$	$2C_F\,(t_b+t_3+Xt_4)$	$2C_F\,(t_b+t_3+Xt_4)$	$4C_F\,(t_1+t_2)$	–	$4C_F\,(t_b+t_3+Xt_4)$
Z_R	$(As_F+XAs_R)\,C_Rw_S$	$(A_{SW1}+A_{SW2})\,2C_Rw_S$	$(A_{SW3}+A_{SW4})\,2C_Rw_S$	–	–	–	–	–
Z_{co}	$C_{co}(Xt_4+t_3)$	$2\,C_{co}t_2$	$2\,C_{co}t_1$	$C_{co}(2t_2(t_3+Xt_4)$	$C_{co}(2t_1(t_3+Xt_4)$	$4\,C_{co}t_2\,t_1$	$C_{co}(t_3+Xt_4)$	–
Z_{R5}	–	–	–	$2\,C_5$	$2\,C_5$	–	–	$2\,C_5$
Z_{R6}	–	–	–	C_62F	$C_6\,2F$	–	–	$4C_6F$

V): The objective function of rectangular partially-buried reinforced concrete tank (open)

The final formulation of the overall cost objective function of rectangular partially buried open concrete tank (ZT_5)is :

ZT_5=[$C_2\,t_b$+C_3+$C_{co}t_3$+ (As_F) $C_R w_S$ + C_5F)+] **BL** +[(2$C_{co}\,t_1$+ 2$C_R w_S$(As_{W3}+As_{W4})+4C_f+2C_3)] **LH** + [(4C_f+2C_3+2$C_R w_S$(As_{W1}+ As_{W2})+ 2$C_{co}t_2$)] **BH** + [(2$C_2 t_b t_2$+2$C_3 t_2$+C_{co}(2$t_2 t_3$)+2C_f(t_3+t_b))+2C_5+ C_1(2t_2+2) F+ $C_6$2F)] **B** + [(2$C_2 t_b t_1$+2$C_3 t_1$+2$C_{co} t_1 t_3$+2C_f(t_3+ t_b)+ 2C_5+ C_1(2t_1+2) F+$C_6$2F)] **L** +[(4$C_{co} t_1 t_2$+4C_f(t_1+t_2))] **H** + [($C_2 t_b$+C_3+$C_{co}t_3$+ C_1F] 4$t_1 t_2$ + [F (C_6+ C_1)+C_f (t_3+t_b)] 4(t_1+t_2) + 2C5 (t_1+t_2) (4.31)

The final cost objective function of the partially buried rectangular reinforced concrete open tank is given in Table (4.6).

Table (4.6) Summary of final cost objective function, (ZT_5).

Cost function	Terms of the decision variables							
	BL	BH	LH	B	L	H	t_1t_2	t_1+t_2
Z_{R1}	C_1 F	–	–	C_1F (2t_2+2)	C_1F (2t_1+2)	–	C_1F	4C_1F
Z_{R2}	C_2t_b	–	–	2C_2t_b t_2	2C_2t_b t_1	–	4C_2t_b	–
Z_{R3}	C_3	2C_3	2C_3	2t_2 C_3	2t_1 C_3	–	C_3	–
Z_F	X C_F	4 C_F	4 C_F	2C_F (t_b+ t_3)	2C_F (t_b+ t_3)	4C_F (t_1+ t_2)	–	4C_F (t_b+ t_3)
Z_R	(As_F) $C_R w_S$	(A_{SW1} + A_{SW2}) 2$C_R w_S$	(A_{SW3} + A_{SW4}) 2$C_R w_S$	–	–	–	–	–
Z_{co}	$C_{co}t_3$	2 $C_{co}t_2$	2 $C_{co}t_1$	2 $C_{co}t_2$ t_3	2 $C_{co}t_1$ t_3	4 $C_{co}t_2$ t_1	$C_{co}t_3$	–
Z_{R5}	–	–	–	2 C_5	2 C_5	–	–	2 C_5
Z_{R6}	–	–	–	$C_6$2F	C_6 2F	–	–	4C_6F

4.2.3.2 The objective function of the circular reinforced concrete tank

I): The objective function of fully-buried circular concrete tank

According to the detailed shown in Fig (4.4), the overall cost objective function (ZT_6) of this type of tank is as follows:

[A] Cost of excavation, (Z_{C1})

The cost of excavation is the same as for the rectangular tank but replacing the volume of excavation of the rectangular by the circular, that is:

$$Z_{C1} = C_1 V_{C1} \qquad (4.32)$$

where :

V_{C1}= volume of exaction of the circular concrete tank, (m^3).

The volume of excavation can be calculated as follows:

$$V_{C1} = \pi/4\ (D+2t_1)^2\ (S+F) \qquad (4.33)$$

Consequently :

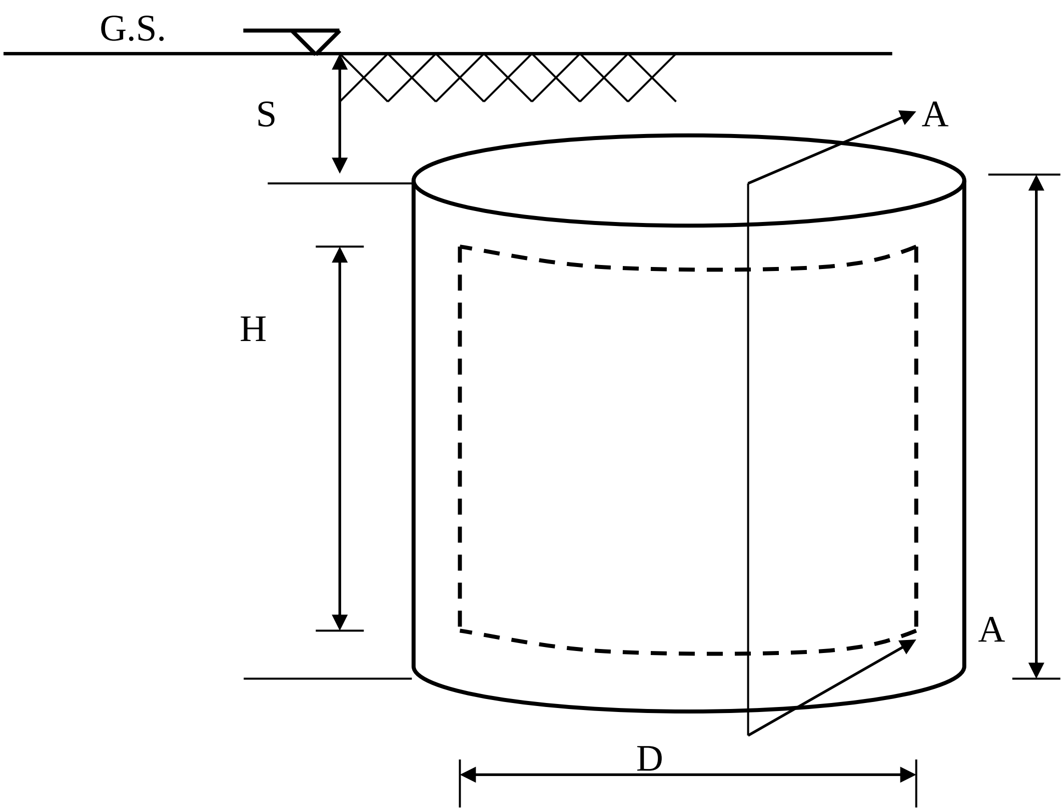
$$Z_{C1} = C_1 \pi/4\ [(S+F)\ D^2 + 2D\ t_1\ (S+F) + t_1^2\ (S+F)] \qquad (4.34)$$

[B] Cost of bedding, (Z_{C2})

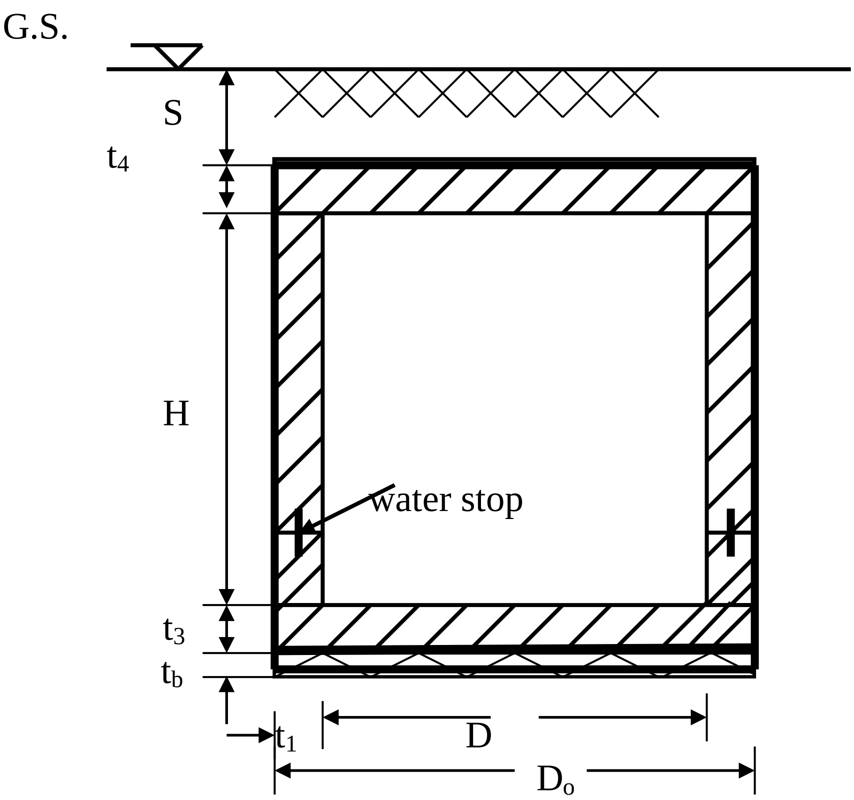
The cost of bedding of the circular concrete tank can be determined as:

$$Z_{C2} = C_2 V_{C2} \qquad (4.35)$$

where:



[a]:The isometric view.



[b]:Section(A-A)

Figure(4.4):Typical works of a circular fully-buried reinforced concrete tank.

V_{C2}= volume of the bedding materials of the circular tank shape, (m^3).

$$V_{C2} = \pi/4\,(D + 2t_1)^2\,(t_b) \qquad (4.36)$$

where :

t_1= thickness of wall of the circular concrete tank, (m);

D= internal diameter of the circular concrete tank, (m)

Then:

$$V_{C2} = \pi/4\,(D^2 + 4t_1 D + 4t_1^2)t_b \qquad (4.37)$$

Thus:

$$Z_{C1} = \pi/4\,C_1\,t_b\,[D^2 + 4t_1 D + 4t_1^2] \qquad (4.38)$$

[C] <u>Cost of water proofing, (Z_{C3})</u>

The cost of the water proofing layer can be calculated as follows:

$$Z_{C3} = C_3 A_{C3} \qquad (4.39)$$

where:

A_{C3}= area of sliding layer, (m^2).

The area of the water proofing layer is:

$$A_{C3} = \pi/4\,(1+X)\,(D+2t_1)^2 + \pi\,I(D+2t_1) \qquad (4.40)$$

Thus :

$$Z_{C3} = C_3\,[\pi/4\,(1+X)(D^2 + 4t_1 D + 4t_1^2) + \pi DH + 2\pi t_1 H] \qquad (4.41)$$

[D] <u>Cost of the body of the tank, (Z_{C4})</u>

As mentioned in item (4.2.3) the cost of reinforced concrete materials involves three parts which are:

(1) <u>Cost of formworks, (Z_f)</u>.

The cost of the formworks of the circular concrete tank is:

$$Z_f = C_f\, A_f \qquad (4.42)$$

where:

A_f= the area of formworks of the circular tank, (m^2).

The area of formworks of the circular concrete tank can be determined as follows:

$$A_f = [\,\pi\,(D+2t_1)\,(H+Xt_4+t_3+t_b) + \pi\, D\, H + \pi/4\, X\, D^2] \qquad (4.43)$$

Thus :

$$Z_f = C_f\,[\,\pi/4\, X\, D^2 + 2\pi\, DH + 2\pi\, t_1 H + \pi\, D\,(t_3+t_b+Xt_4) + 2\pi\, t_1\,(t_3+t_b+Xt_4)] \qquad (4.44)$$

(2)Cost of reinforcement, (Z_{CR}).

The cost of reinforcement of the circular concrete tank can be calculated as follow:

$$Z_{CR} = C_R W_R \qquad (4.45)$$

The weight of reinforcement (W_R) can be calculated as:

$$W_{CR} = \pi\, DH(As_{W1}+As_{W2})\, w_S + \pi/4\,(D+2t1)^2\,(As_F + X\, As_R) \qquad (4.46)$$

where:

w_S= unit weight of reinforcement per cubic meter, (kg/m^3):

As_{W1}= Area of horizontal hoop reinforcement in the tank wall , (m^2/m);

As_{W2}=Area of vertical bending reinforcement in the tank wall , (m^2/m);

As_F= Area of floor slab reinforcement of the tank, (m^2/m).

As_R= Area of roof slab reinforcement of the tank, (m^2/m).

Thus :

$$Z_R = C_R w_S[\,\pi\, D\, H(As_{W1}+As_{W2}) + \pi/4\,(D^2 + 4t_1\, D + 4t_1^2)\,(As_F + X\, As_R)] \qquad (4.47)$$

(3)Cost of concrete materials, (Z_{co})

The cost of concrete materials of the circular concrete tank is:

$$Z_{co}= C_{co}V_{co} \tag{4.48}$$

where:

V_{co}= volume of concrete materials of the circular tank, (m^3).

The volume of concrete materials (V_{co}) can be calculated as:

$$V_{co}= \pi/4\ (D+2t_1)^2(t_3+ Xt_4) + \pi\ DH\ t_1] \tag{4.49}$$

Thus:

$$Z_{co}= C_{co}\ [\pi/4\,(Xt_4+t_3)\ (D^2 + 4t_1\ D + 4t_1^{\ 2}) + (\pi\ DH\ t_1)] \tag{4.50}$$

[E] Cost of water stop material, (Z_{c5})

$$Z_{C5}= C_5 S_{C5} \tag{4.51}$$

where :

S_{C5}= length of the water stop material, (m).

Then :

$$S_{C5} = \pi\ (D+t_1) \tag{4.52}$$

Thus :

$$Z_{C5}= \pi\, C_5\ [D+ t_1] \tag{4.53}$$

[F] Cost of fill work, (Z_{C6})

The cost of fillwork can be in general determined as follows:

$$Z_{C6}= C_6\ V_{C6} \tag{4.54}$$

where :

V_{C6}= volume of the earth fillwork over the circular concrete tank, (m^3).

Then:

$$V_{C6} = [(\pi\ (D+2t_1)(S+F) + \pi/4\ (D+2t_1)^2 S] \quad (4.55)$$

Consequently :

$$Z_{C6} = C_6[(\pi + \pi\, t_1)S + \pi F)\, D + \pi/4\ S\, D^2 + 2\ \pi\, t_1 (S+F) + \pi\, t_1^2 S] \quad (4.56)$$

All the aforementioned sub – objective cost functions are summarized in Table (4.7).

The total cost (ZT_6) is the summation of all aforementioned sub-cost functions calculated in the previous items and given in Table (4.7). That is :

$$ZT_6 = \sum Z_{ci} \qquad i = 1,2,\ldots,6 \quad (4.57)$$

Thus, the overall cost objective function may be written in the form:

$$\begin{aligned} ZT_6 = {} & [\pi/4\ (C_2\, t_b + (1+X)C_3 + Cco(Xt_4+t_3) + (A_{SF} + XA_{SR})\, C_R w_S + XC_f + C_1(S+F) + C_6 S]\ \mathbf{D^2} + [\pi\, (Ccot_1 + C_3 + C_R w_S(A_{SW1} + A_{SW2}) + 2C_f]\ \mathbf{DH} + \\ & [\pi\, t_1\, (C2t_b + (1+X)C_3 + Cco(t_3+Xt_4) + C_1(S+F) + C_6 F + C_R w_S(A_{SF} + XA_{SR})) \\ & + \pi\, C_f(t_3 + t_b + Xt_4) + \pi\, C_5 + \pi\, (C_1 + C_6)(S+F)]\ \mathbf{D} + [2\pi\, t_1\, (C_3 + C_f)]\ \mathbf{H} + \\ & [\pi\, (C_2 t_b + (1+X)C_3 + C_{c0}\, (t_3 + Xt_4) + C_6 s + C_1(S+F)]\ \mathbf{t_1^2} + [\pi\, (C_f\, (t_3 + t_b + Xt_4) + \\ & C_5 + 2\, (C_1 + C_6)\, (S+F)]\ \mathbf{t_1} \end{aligned} \quad (4.58)$$

The formulations of the sub-objective functions of (ZT_6) are summarized in Table (4.7).

Table (4.7) Summary of final cost objective function, (ZT_6)

Cost function	Terms of the decision variables					
	D^2	DH	H	D	t_1^2	t_1
Z_{C1}	$\pi/4\, C_1(S+F)$	–	–	$C_1\,(S+F)\,(\pi\, t_1+\pi)$	$\pi\, C_1(S+F)$	$2\pi\, C_1(S+F)$
Z_{C2}	$\pi/4\, C_2 t_b$	–	–	$\pi\, C_2 t_b\, t_1$	$\pi\, C_2 t_b$	–
Z_{C3}	$(1+X)\, C_3\, \pi/4$	$\pi\, C_3$	$2\pi\, C_3\, t_1$	$\pi\,(1+X) t_1\, C_3$	$\pi\,(1+X) C_3$	–
Z_F	$\pi/4\, X\, C_F$	$2\pi\, C_F$	$2\pi\, C_F t_1$	$\pi\, C_F\,(t_b+t_3+Xt_4)$	–	$2\pi\, C_F\,(t_b+t_3+Xt_4)$
Z_R	$\pi/4\,(As_F+XAs_R)\, C_R w_S$	$(A_{SW1}+A_{SW2})\,\pi\, C_R w_S$	–	$(As_F+XAs_R)\,\pi\, t_1\, C_R w_S$	–	–
Z_{co}	$Cco(Xt_4+t3)\,\pi/4$	$\pi\, C_{co} t_1$	–	$(t_3+Xt_4)\,\pi\, C_{co} t_1$	$\pi\, C_{co}(t_3+Xt_4)$	–
Z_{C5}	–	–	–	$\pi\, C_5$	–	$\pi\, C_5$
Z_{C6}	$\pi/4\, C_6 S$	–	–	$((\pi\, t_1+\pi)S+\pi\, F)\, C_6$	$\pi\, C_6 S$	$2\pi\, C_6(S+F)$

II): The objective function of circular on-ground concrete tank (open)

The overall cost objective function (ZT_7) of this type of tank is as follows:

$$ZT_7=[\pi/4\,(C_2\, t_b+C_3+C_{co}t_3+(A_{SF})\, C_R w_S]\, \mathbf{D^2}+[\pi\,(C_{co}t_1+C_R w_S(A_{SW1}+A_{SW2})+2C_f]\,\mathbf{DH}+[\pi\, t_1\,(C_2 t_b+C_3+C_{co}t_3+C_R w_S(A_{SF}))+\pi\, C_f(t_3+t_b)+\pi\, C_5]\,\mathbf{D}+[2\pi\, t_1\, C_f]\,\mathbf{H}+[\pi\,(C_2 t_b+C_3+C_{c0}\, t_3]\,\mathbf{t_1^2}+[\pi\,(C_f\,(t_3+t_b)+C_5]\,\mathbf{t_1} \qquad (4.59)$$

The summary of the formulated respective cost objective function is given in Table (4.8).

Table (4.8) Summary of final cost objective function, (ZT_7)

Cost function	Terms of the decision variables					
	D^2	DH	H	D	t_1^2	t_1
Z_{C1}	–	–	–	–	–	–
Z_{C2}	$\pi/4\, C_2 t_b$	–	–	$\pi\, C_2 t_b\, t_1$	$\pi\, C_2 t_b$	–
Z_{C3}	$C_3\, \pi/4$	–	–	$\pi\, t_1\, C_3$	$\pi\, C_3$	–
Z_F	–	$2\pi\, C_F$	$2\pi\, C_F t_1$	$\pi\, C_F\, (t_b + t_3)$	–	$2\pi\, C_F\, (t_b + t_3)$
Z_R	$\pi/4\, (As_F)\, C_R ws$	$(A_{SW1} + A_{SW2})\, \pi\, C_R ws$	–	$\pi\, t_1\, C_R ws (As_F)$	–	–
Z_{co}	$C_{co} t_3\, \pi/4$	$\pi\, C_{co} t_1$	–	$t_3\, \pi\, C_{co} t_1$	$\pi\, C_{co}(t_3 + Xt_4)$	–
Z_{C5}	–	–	–	$\pi\, C_5$	–	$\pi\, C_5$
Z_{C6}	–	–	–	–	–	–

<u>III): The objective function of circular on-ground concrete tank (closed)</u>

The overall cost objective function (ZT_8) of this type of tank is as follows:

$$ZT_8 = [\pi/4\ (C_1\, t_b + C_2 + C_{co}(Xt_4 + t_3) + (A_{SF} + XA_{SR})\, C_R w_S + XC_f]\, \mathbf{D^2} + [\pi\, (C_{co} t_1 + C_R w_S (A_{SW1} + A_{SW2}) + 2C_f]\, \mathbf{DH} + [\pi\, t_1\, (C_1 t_b + C_2 + C_{co}(t_3 + Xt_4) + C_R w_S (A_{SF} + XA_{SR})) + \pi\, C_f (t_3 + t_b + Xt_4) + \pi\, C_4]\, \mathbf{D} + [2\pi\, t_1\, C_f]\, \mathbf{H} + [\pi\, (C_1 t_b + C_{c0}\, (t_3 + Xt_4)]\, \mathbf{t_1^2} + [\pi\, (C_f\, (t_3 + t_b + Xt_4) + C_4]\, \mathbf{t_1} \qquad (4.60)$$

The summary of the formulation of (ZT_8)is given in Table (4.9).

Table (4.9) Summary of final cost objective function, (ZT_8)

Cost function	Terms of the decision variables					
	D^2	DH	H	D	t_1^2	t_1
Z_{C1}	–	–	–	–	–	–
Z_{C2}	$\pi/4\, C_2 t_b$	–	–	$\pi\, C_2 t_b\, t_1$	$\pi\, C_2 t_b$	–
Z_{C3}	$C_3\, \pi/4$	–	–	$\pi\; t_1\, C_3$	$\pi\; C_3$	–
Z_F	$\pi/4\, X\, C_F$	$2\pi\, C_F$	$2\pi\, C_F t_1$	$\pi\, C_F\, (t_b + t_3 + Xt_4)$	–	$2\pi\, C_F\, (t_b + t_3 + Xt_4)$
Z_R	$\pi/4\; (As_F + XAs_R)\, C_R w_S$	$(A_{SW1} + A_{SW2})\; \pi\, C_R w_S$	–	$(As_F + XAs_R)\; \pi\; t_1\, C_R w_S$	–	–
Z_{co}	$C_{co}(Xt_4+t_3)\; \pi/4$	$\pi\; C_{co} t_1$	–	$(t_3 + Xt_4)\; \pi\, C_{co} t_1$	$\pi\, C_{co}(t_3 + Xt_4)$	–
Z_{C5}	–	–	–	$\pi\, C_5$	–	$\pi\; C_5$
Z_{C6}	–	–	–	–	–	–

IV): The objective function of circular partially-buried concrete tank (closed)

The overall cost objective function (ZT_9) of this type of tank would be:

$$ZT_9 = [\pi/4\;(C_2\, t_b + C_3 + C_{co}(Xt_4 + t_3)) + (A_{SF} + XA_{SR})\, C_R w_S + XC_f + C_1 F]\, \mathbf{D^2} + [\pi\,(C_{co} t_1 + C_3 + C_R w_S (A_{SW1} + A_{SW2}) + 2C_f]\, \mathbf{DH} + [\pi\, t_1\,(C_2 t_b + C_3 + C_{co}(t_3 + Xt_4) + C_1 F + C_6 F + C_R w_S (A_{SF} + XA_{SR})) + \pi\, C_f (t_3 + t_b + Xt_4) + \pi\, C_5 + \pi\,(C_1 + C_6) F]\, \mathbf{D} + [2\pi\, t_1\,(C_3 + C_f)]\, \mathbf{H} + [\pi\,(C_2 t_b + C_3 + C_{c0}\,(t_3 + Xt_4) + C_1 F]\, \mathbf{t_1^2} + [\pi\,(C_f\,(t_3 + t_b + Xt_4) + C_5 + 2\,(C_1 + C_6)\, F]\, \mathbf{t_1} \qquad (4.61)$$

The summary of the formulation of (ZT_9)is given in Table (4.10).

Table (4.10) Summary of final cost objective function, (ZT_9)

Cost function	Terms of the decision variables					
	D^2	DH	H	D	t_1^2	t_1
Z_{C1}	$\pi/4\, C_1F$	–	–	$C_1 F (\pi t_1+\pi)$	πC_1F	$2\pi C_1F$
Z_{C2}	$\pi/4\, C_2t_b$	–	–	$\pi C_2t_b t_1$	πC_2t_b	–
Z_{C3}	$C_3\pi/4$	πIC_3	$2\pi C_3 t_1$	$\pi t_1 C_3$	πC_3	–
Z_F	$\pi/4\, X C_F$	$2\pi C_F$	$2\pi C_Ft_1$	$\pi C_F (t_b+t_3+Xt_4)$	–	$2\pi C_F (t_b+t_3+Xt_4)$
Z_R	$\pi/4\, (As_F+XAs_R) C_Rw_S$	$(A_{SW1}+A_{SW2})\pi C_Rw_S$	–	$(As_F+XAs_R)\pi t_1 C_Rw_S$	–	–
Z_{co}	$C_{co}(Xt_4+t_3)\pi/4$	$\pi C_{co}t_1$		$(t_3+Xt_4)\pi C_{co}t_1$	$\pi C_{co}(t_3+Xt_4)$	–
Z_{C5}	–	–	–	πC_4	–	πC_4
Z_{C6}	–	–	–	πFC_6	–	$2\pi C_6F$

<u>V): The objective function of circular partially-buried concrete tank (open)</u>

The overall cost objective function (ZT_{10}) of this type of tank is:

$$ZT_{10}= [\pi/4\ (C_2 t_b+C_3+ C_{co}t_3+(A_{SF})\ C_Rw_S +C_1F]\ \mathbf{D^2} +[\pi (C_{co}t_1+ C_3+C_Rw_S (A_{SW1}+A_{SW2})+2C_f]\ \mathbf{DH} +[\pi t_1 (C_2t_b+C_3 +C_{co}t_3 + C_Rw_S(A_{SF}))+ \pi C_f(t_3+ t_b)+ \pi C_5 + \pi (C_1+ C_6)F]\ \mathbf{D} + [2\pi t_1 (C_3I+C_f)]\ \mathbf{H} + [\pi (C_2t_b+C_3+C_{c0}t_3+C_1F]\ \mathbf{t_1^2} + [\pi (C_f (t_3+ t_b)+ C_5+2 (C_1+ C_6) F]\ \mathbf{t_1} \quad (4.62)$$

The summary of the formulated respective cost objective function(ZT_{10}) are given in Table (4.11).

Table (4.11) Summary of final cost objective function, (ZT_{10})

Cost function	Terms of the decision variables					
	D^2	DH	H	D	t_1^2	t_1
Z_{C1}	$\pi/4\, C_1F$	–	–	$C_1 F$ $(\pi t_1+\pi)$	πC_1F	$2\pi C_1F$
Z_{C2}	$\pi/4\, C_2t_b$	–	–	$\pi C_2t_b t_1$	πC_2t_b	–
Z_{C3}	$C_3\pi/4$	πC_3	$2\pi C_3 t_1$	$\pi t_1 C_3$	πC_3	–
Z_F	–	$2\pi C_F$	$2\pi C_Ft_1$	$\pi C_F (t_b+ t_3)$	–	$2\pi C_F (t_b+ t_3)$
Z_R	$\pi/4\ (As_F)\ C_{RWS}$	$(A_{SW1}+ A_{SW2})\ \pi C_{RWS}$	–	$(As_F)\ \pi t_1\ C_{RWS}$	–	–
Z_{co}	$Ccot_3\ \pi/4$	$\pi C_{co}t_1$		$t_3\pi C_{co}t_1$	$\pi C_{co}(t_3)$	–
Z_{C5}	–	–	–	πC_5	–	πC_5
Z_{C6}	–	–	–	πFC_6	–	$2\pi C_6F$

4.3 THE CONSTRAINTS

The objective function is minimized subject to a set of constraints. The basic controlling constraints are discussed hereinafter.

4.3.1 Dimensions

[1]: Thickness of reinforced concrete walls at least (3) m high should have a minimum thickness of (30) cm. Usually, the minimum thickness of any minor structural member in environmental engineering concrete structures is (20) cm will be required where a (50) mm concrete cover is desired **[ACI 350R, 1989]**. According to **[ACI 318, 1963]** minimum thickness of one way and two way slabs is (B/28 and Peremeter/180), respectively, i.e.:

$t_4 \geq B / 28$ for one way slab (4.63)

$t_4 \geq 2(B+ L) / 180$ for two way slab (4.64)

$t_1,t_2 \geq 20$ cm for tank wall less than 3m in height (4.65)

$t_1,t_2 \geq 30$ cm for tank wall more than 3m in height (4.66)

$t_3, t_4 \geq 20$ cm for tank floor and roof (4.67)

$L \leq L$ limit (4.68)

$B \leq B$ limit (4.69)

$H \leq H$ limit (4.70)

$D \leq D$ limit (4.71)

[2] The reinforced concrete sections should safely resist the critical applied moments, shear forces, and hoop tension. These conditions can be stated in general as **[Hamdi, 1998]**:

$$|M| - As. f_S. jd \leq 0 \tag{4.72}$$

$$(T \text{ or } F)/ ((b\ t)+(n-1)As) - f_{ct} \leq 0 \tag{4.73}$$

$$(F \text{ or } T) / As - f_S \leq 0 \tag{4.74}$$

$$V / b\ d - V_C \leq 0 \tag{4.75}$$

where :

M= the maximum negative or positive bending moment.

F= the maximum hoop force;

T= the maximum direct tension force;

V= the maximum shear force;

f_{ct}= the allowable tensile stress of concrete, table (4.11);

d = effective depth of concrete section;

As= the steel reinforcement area in general;

j = the internal lever arm;

V_C= the allowable shear stress of concrete.

where:

$$V_C = 0.16\ (\sqrt{f_C^{\,-}})\ b\ d \qquad (4.76)$$

Table (4.11) Permissible concrete stresses in relation to resisting cracking **[**After **Syal et al. (2005)]**

Grade of concrete	Permissible stresses kN/m^2		Shear *(10^3)
	Direct Tension*(10^3)	Tension Due to Bending*(10^3)	
M15	1.1	1.5	1.5
M20	1.2	1.7	1.7
M25	1.3	1.8	1.9
M30	1.5	2,0	2.2
M35	1.6	2.2	2.5
M40	1.7	2.4	2.7

4.3.2: Minimum reinforcement

[ACI 350R, 1989] suggested that the minimum reinforcement in walls, roofs, and floors in each of two directions at right angles shall have an area of 0.28 percent of the concrete section. However, **[ACI 318R, 2005]** suggested a limit of minimum reinforcement depending upon the grade of reinforcement as follows:

For grade 280000 KN/m^2 and 350000 KN/m^2 the minimum reinforcement shall have an area of 0.2 percent of the concrete section, and for grade 420000 KN/m^2 the minimum reinforcement shall have an area of 0.18 percent of the concrete section. The former restriction has been considered in the present study **[ACI 350R, 1989]**, that is:

$$As_{min} = 0.0028\ b\ d \tag{4.77}$$

Therefore the bending, tension, and hoop reinforcement should be not less than the minimum steel area. These conditions can be stated as:

$$As_{min} / As - 1 \leq 0 \tag{4.78}$$

where:

$$As\ bending = M_{max} / f_S\ j\ d \tag{4.79}$$

$$As\ tension = T_{max} / f_S \tag{4.80}$$

$$As\ hoop = T / f_S \tag{4.81}$$

4.3.3: Durability and impermeability of the concrete

Durability of concrete is influenced by many factors such as concrete grade, concrete cover, and crack width.

[ACI 350R, 1989] recommended that the concrete grade should be not less than (28 $*10^3$ KN/m^2) and nominal cover of concrete for all steel should be not less than (40 mm). To control crack width for concrete surfaces exposed to different condition of environment, the permissible concrete stresses as suggested by **[Syal et al. 2005]** and given in table **(** 4. 11**)** should be consulted**.**

As stated in **[Fredrick et al. 2001]**, the tensile stress in the reinforcement must not be greater than the following.

Grade 280 and 350 : $140*10^3$ KN/m^2 (4.82)

Grade 420 : $168*10^3$ KN/m^2 (4.83)

However, **[Hamdi, 1998]** stated that the allowable tensile stresses in steel reinforcement due to hoop tension and direct tension should not exceed f_{Smax} which is about (100 $*10^3$ KN/m^2) in order that crack width do not exceed the limit. Moreover, the allowable tensile stresses in steel reinforcement due to bending of (138 $*10^3$ KN/m^2) was suggested by **[PCA, 1996]**. These limits shall be adopted in the present research.

The compressive stress in the extreme surface of the concrete must not exceed (0.45 f_c'**)**, where the f_c' is the 28-day compressive strength of the concrete **[Fredrick et al. 2001]**.

To control permeability, the thickness of wall of circular and rectangular tanks in relation to the height as in **[Hamdi, 1998]** and [**Uraiby, 1997]**is:

$$t_1, t_2 \geq 0.025\ H + 0.05 \qquad (4.84)$$

4.3.4: Concrete tanks storage capacity

A prespecified constant storage capacity is assumed for rectangular, square, and circular tanks as follows, respectively:

$$V_{OR} \geq B.L.H \quad (4.85)$$

$$V_{OS} \geq L^2.H \quad (4.86)$$

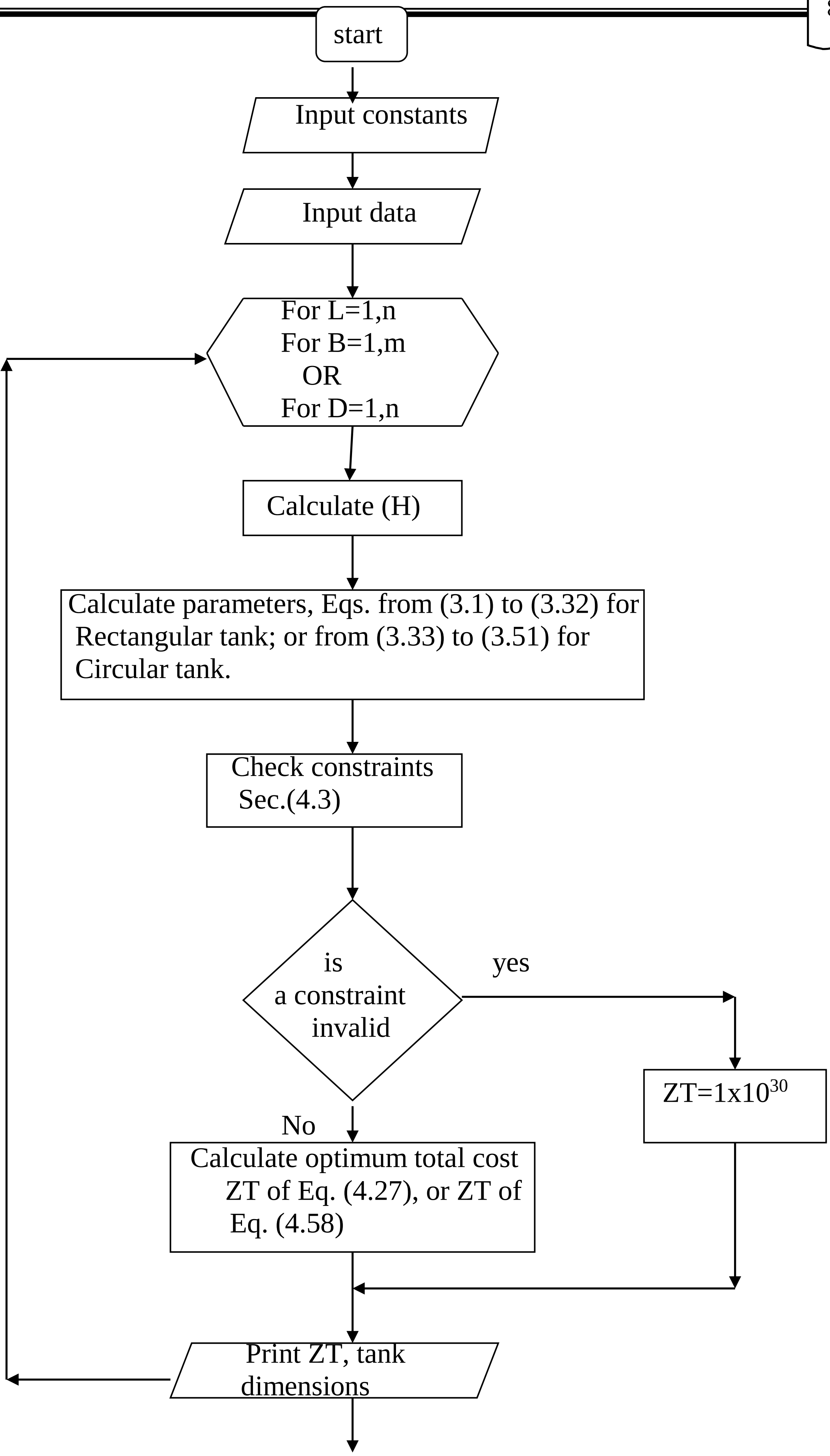
$$V_{OC} \geq (\pi/4)\, D^2.H \quad (4.87)$$

4.4.: The adopted method of optimization

The method of optimization used in this research is based on the iterative method and may be summarized in the following steps:

1. Set loops for the horizontal dimensions (L, B, or D) of the tank with a step length.
2. Calculate the vertical dimension (H) of the tank.
3. It has been suggested that merely giving the objective function a very large positive value (in a minimization problem) (usually $Z_{min}=1\times10^{30}$), whenever the constraints are violated **[Bundy, 1984]**.
4. Calculate the total cost objective function (ZT) of the given dimensions and constraints.
5. If the calculated (ZT) is less than (Z_{min}), set (Z_{min}= ZT). Otherwise continue with new dimensions and new (ZT).

The main program of the optimization process is schematically shown in the flow chart given in Fig. (4.5)


start
Input constants
Input data
For L=1,n
For B=1,m
OR
For D=1,n
Calculate (H)
Calculate parameters, Eqs. from (3.1) to (3.32) for Rectangular tank; or from (3.33) to (3.51) for Circular tank.
Check constraints Sec.(4.3)
is a constraint invalid
yes
ZT=1x10^30
No
Calculate optimum total cost ZT of Eq. (4.27), or ZT of Eq. (4.58)
Print ZT, tank dimensions

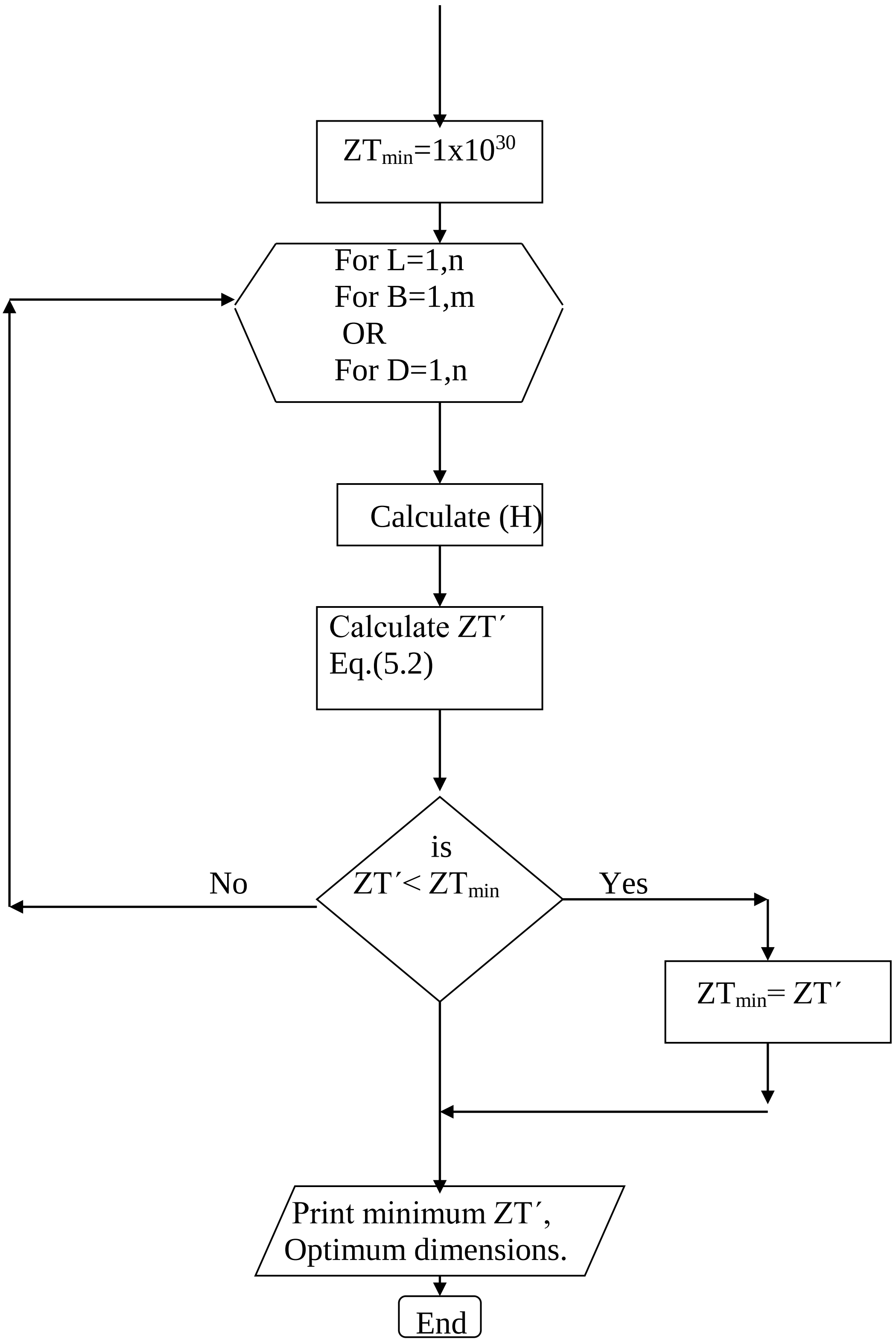


Figure (4.5):Flow chart of the main program of the optimization process.

CHAPTER FIVE

APPLICATIONS AND ANALYSIS OF RESULTS

In this chapter, unit prices which are mentioned in chapter Four are specified; a generalized case study is suggested, which includes all parameters for analysis, design, and constructing the concrete tanks. These parameters are introduced in the constructed optimization model. Finally, the obtained results for the considered cases are analyzed.

5.1: UNIT PRICES

The unit prices [C_1, C_2, C_3, ,C_F, C_R, C_{CO}, C_5, and C_6] in Eqs. from (4.1) through (4.62) are given in Table (5.1).

Table (5.1):Applied unit prices[After KM, 2009]

Symbol and units	Item	Value (1000 I.D.)
C_1 (ID/m^3)	Excavation (including finishing).	20
C_2 (ID/m^3)	Blinding concrete (materials).	25
C_3 (ID/m^2)	Sliding layer (materials, labor)	5
C_F (ID/m^2)	Formworks and constructing (reinforced and plain concrete).	25* 65**
C_R (ID/ kg)	Steel reinforcement (materials)	1.5
C_{CO} (ID/ m^3)	Concrete materials	300
C_5 (ID/m)	Water stop materials	18
C_6 (ID/ m^3)	Compacted frillworks	10

*For rectangular tanks.
**For circular tanks.

5.2: GENERAL DATA

In addition to unit prices, the following general data are used in the applications.

1. Modulus of elasticity of steel, E_S= 200000*10^3 KN/m^3.
2. Characteristic cube strength of concrete at 28 days, f_C=30 *10^3 KN/m^3.
3. Modulus of elasticity of concrete, E_C= 26000 *10^3 KN/m^3.
4. Allowable tensile stress in steel due to bending, f_{Sb}=138 *10^3 KN/m^3.
5. Allowable tensile stress in steel due to hoop and direct tension, f_{St}=100 *10^3 KN/m^3.
6. Tensile stress in concrete, f_{ct}=1.5 *10^3 KN/m^3.
7. Depth of earth filling above the roof of fully-buried tanks, S=0.5 m.
8. Minimum cover to tension steel, Cv= 0.05 m.
9. Concrete weight density, γ_C= 24 KN/m^3.
10. Water weight density, γ_w = 9.81 KN/m^3.
11. Dry soil weight density, γ_S= 16 KN/m^3.
12. Steel weight density, w_R = 78.5 KN/m^3.
13. Angle of repose, ϕ_S= 30°.
14. Thickness of bedding layer, t_b=0.1 m.
15. Bearing capacity of soil, qa=50 KN/m^2.
16. Storage tank capacity, V_O= 50, 250, 500, 750, and 1000 m^3.

5.3: THE APPLIED OBJECTIVE FUNCTION

The final total objective functions, given by Eqs. (4.27) for rectangular tanks and (4.58) for circular tanks could be represented by the general form:

$$ZT_i = C_i + f\{X_i\} \qquad i= 1, 2,\ldots, n \qquad (5.1)$$

In the optimization program, the process will consider minimizing ($Z'T_i$) given by:

$$Z'T_i = ZT_i - C_i \qquad (5.2)$$

Then the following matrices could be prepared:

$$[Z'T]_{i \times j} = [a]_{i \times k} . [X]_{k \times j}$$

where:

$Z'T$= final total objective function used in the optimization process;

a = respective cost coefficient;

X = design variables.

5.4: THE COMPUTER PROGRAM

As part of this work, a computer program has been written in Fortran language to embody the formulations of the analysis and optimization. The program consists of two parts as follows:

1. The structural part: After calculating the respective moments and forces by analyze the tanks by moment distribution method, then determined the concrete tank sections and reinforcement.
2. Optimization: In this part, optimization is carried out to find the optimal design dimensions and cost [as discussed in Sec (4.4.4)].

The flow chart of the main program has been shown in Fig. (4.5) before.

5.5: RESULTS OF APPLICATIONS

The results of applications are presented to show the following:

1. Variation of cost with different tank volumes for rectangular and circular reinforced concrete tanks for different situations as follows:
   a. The rectangular and circular on-ground open tanks, ***Fig. (5.1)***.
   b. The rectangular and circular on-ground closed tanks, ***Fig. (5.2)***.
   c. The rectangular and circular partially-buried open tanks, ***Fig. (5.3).***
   d. The rectangular and circular partially-buried closed tanks, ***Fig. (5.4).***
   e. The rectangular and circular fully-buried open tanks, ***Fig. (5.5).***

2. Relation between the optimal dimensions and thicknesses with different tank volumes for rectangular and circular reinforced concrete tanks for different situations as follows:
   a. The rectangular and circular on-ground open tanks, ***Fisg. (5.6)*** and ***(5.7)***.
   b. The rectangular and circular on-ground closed tanks, ***Figs. (5.8)*** and ***(5.9)***.
   c. The rectangular and circular partially-buried open tanks, ***Figs. (5.10)*** and ***(5.11)***.
   d. The rectangular and circular fully-buried tanks, ***Figs. (5.12)*** and ***(5.13)***.
   e. The rectangular and circular fully-buried closed tanks, ***Figs. (5.14)*** and ***(5.15)***.

3. Optimum dimensions, optimum thicknesses, and minimum costs for rectangular and circular reinforced concrete tanks with different tank volumes and different situations as follows:
   a. The rectangular and circular on-ground open tanks, ***Tables (5.2)*** and ***(5.7)***.

b. The rectangular and circular on-ground closed tanks, ***Tables (5.3)*** and ***(5.8)***.

c. The rectangular and circular partially-buried open tanks, ***Tables (5.4)*** and ***(5.9)***.

d. The rectangular and circular partially-buried closed tanks, ***Tables (5.5)*** and ***(5.10)***.

e. The rectangular and circular fully-buried tanks, ***Tables (5.6)*** and ***(5.11)***.

4. Effect of bearing capacity of the soil on the tanks dimensions ***Table (5.12)***.

5. A condensed comparison of the obtained results is given in ***Table (5.13)***. The comparison considers two aspects:

(a): The necessary volume of reinforced concrete. This is indicated by the relative reinforced concrete volume, (RRCV), measured in [cubic meters of reinforced concrete per one cubic meter of stored water]. The table shows the ranking of (RRCV) in an ascending order, i.e., rank (1) denotes the least (RRCV) which virtually denotes the most efficient one in this respect.

(b): The overall involved cost: This is indicated by the economic efficiencies, (EE), measured in [I.D. per one cubic meter of stored water]. The table shows the ranking of (EE) in an ascending order of magnitude, giving rank (1) to the least (EE) which, virtually, would be the most efficient in this regard.

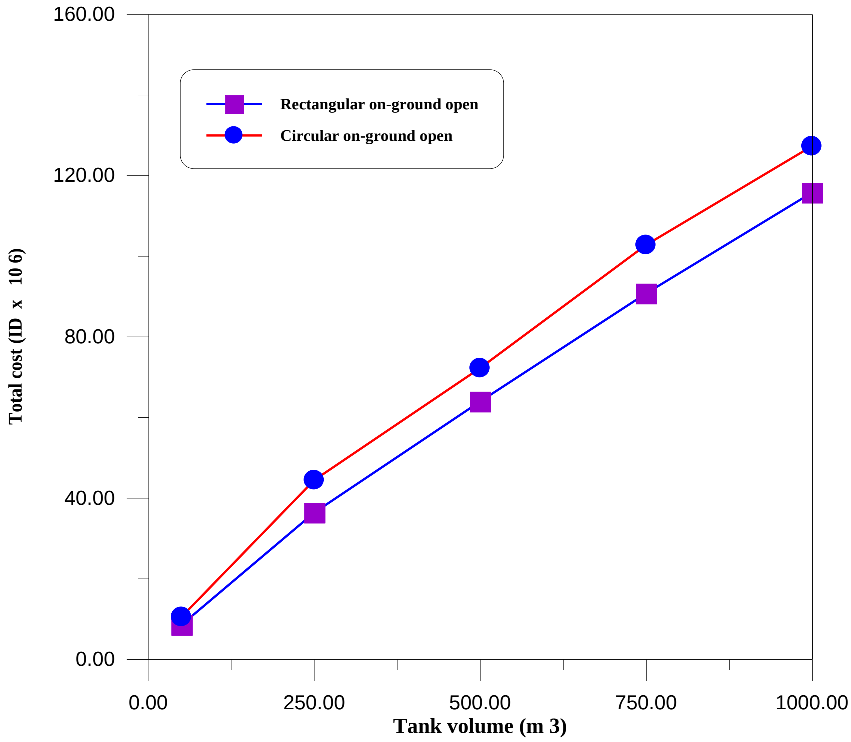


Fig.(5.1) Variation of cost with tank volume for rectangular and circular on-ground open tanks.

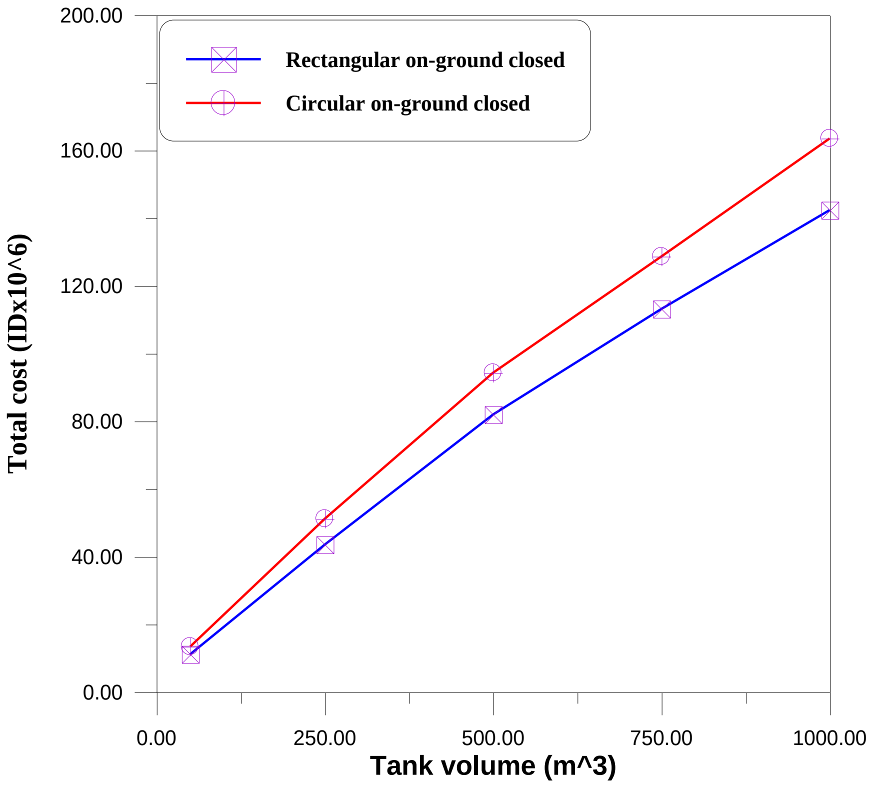


Fig.(5.2) Variation of cost with tank volume for rectangular and circular on-ground closed tanks.

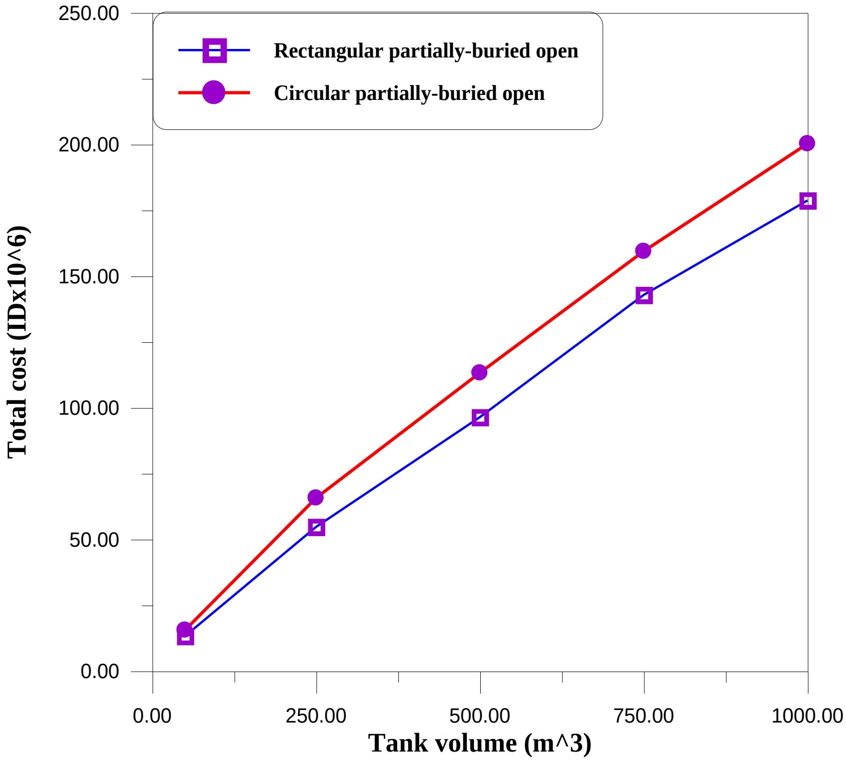


Fig.(5.3) Variation of cost with tank volume for rectangular and circular partially-buried open tanks.

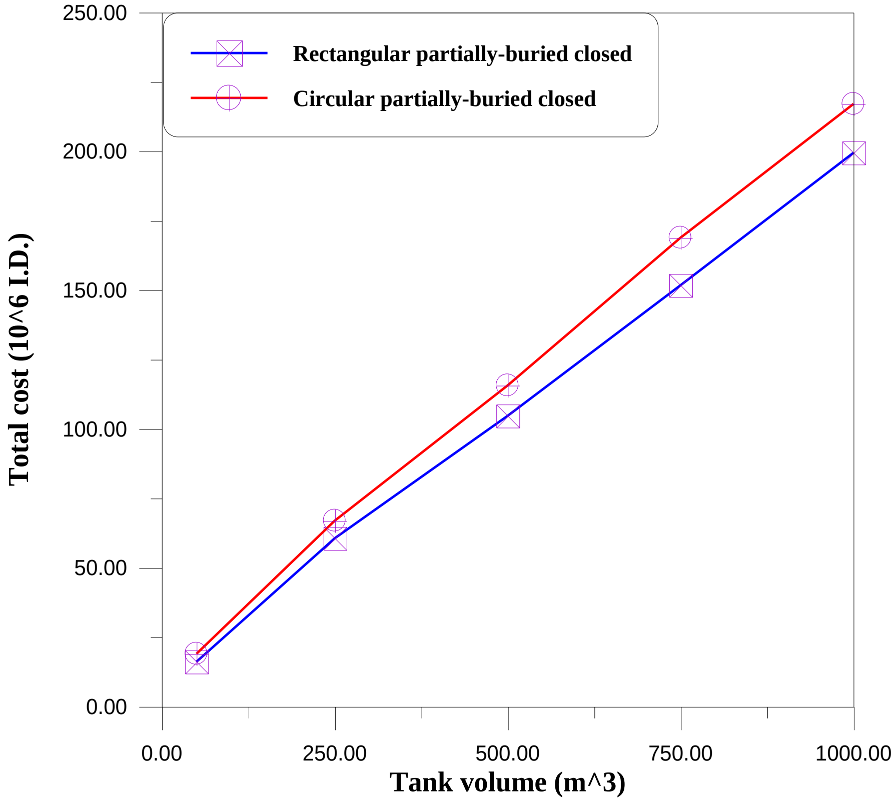


Fig.(5.4) Variation of cost with tank volume for rectangular and circular partially-buried closed tanks.

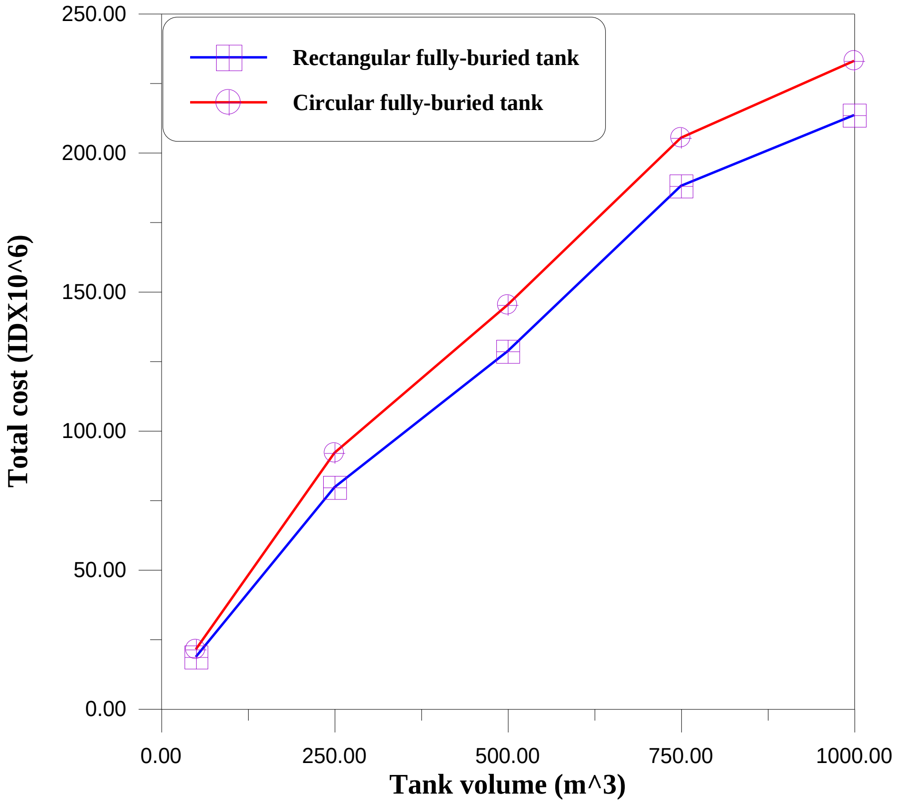


Fig.(5.5) Variation of cost with tank volume for rectangular and circular fully-buried tanks.

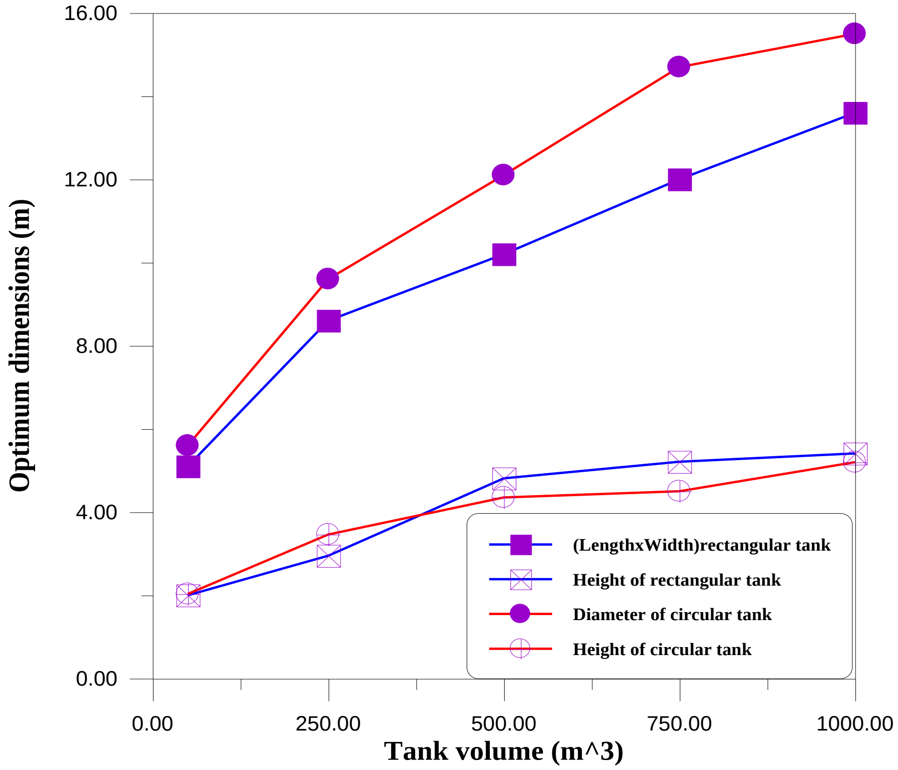


Fig.(5.6) Relation between the optimum dimensions with different tank volumes for rectangular and circular on-ground open tanks.

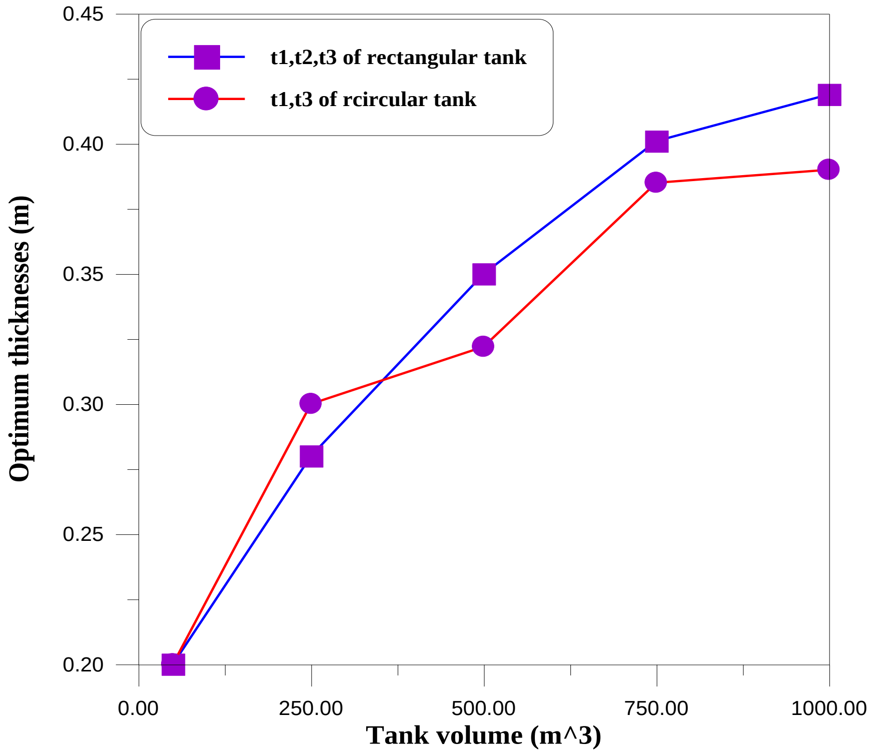


Fig.(5.7) Relation between the optimum thicknesses and the considered tanks volumes for rectangular and circular on-ground open tanks.

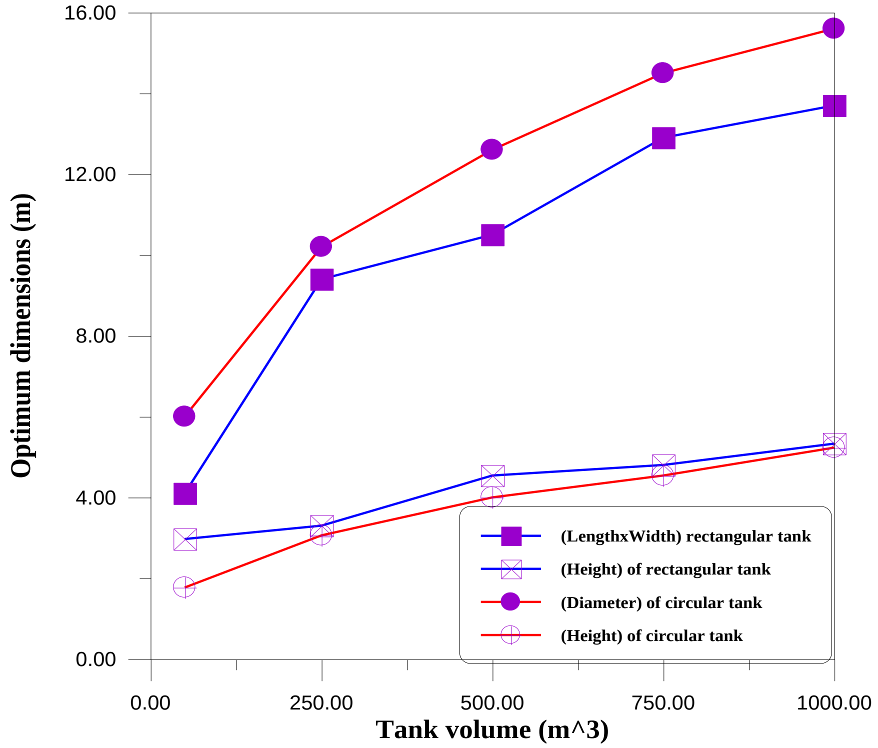


Fig.(5.8) Relation between the optimum dimensions and the considered tanks volumes for rectangular and circular on-ground closed tanks.

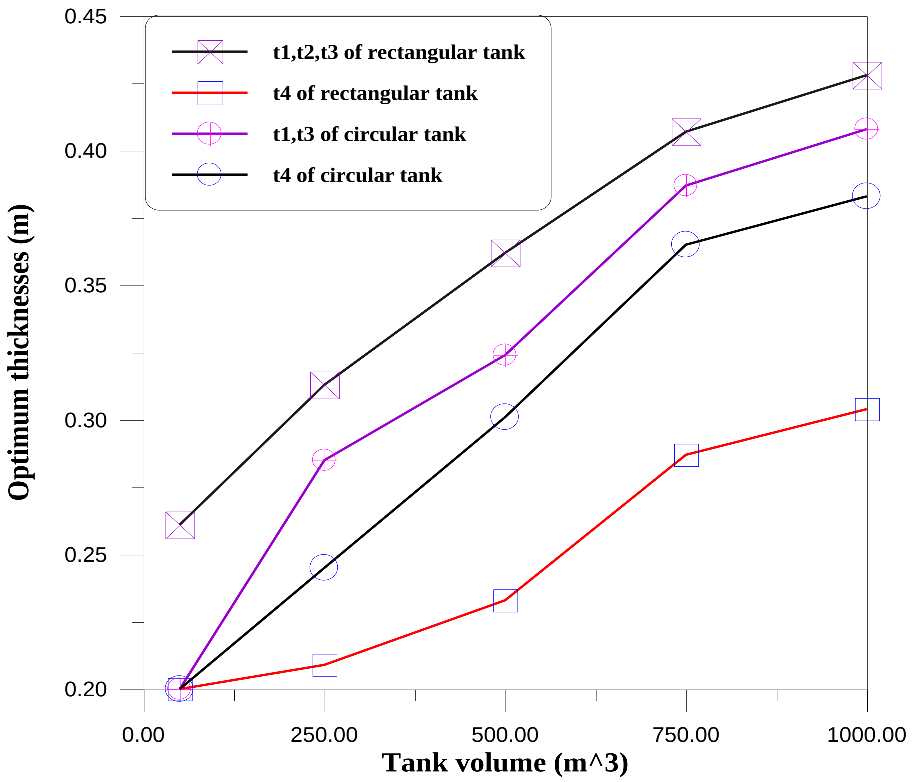


Fig.(5.9) Relation between the optimum thicknesses and the considered tanks volumes for rectangular and circular on-ground closed tanks.

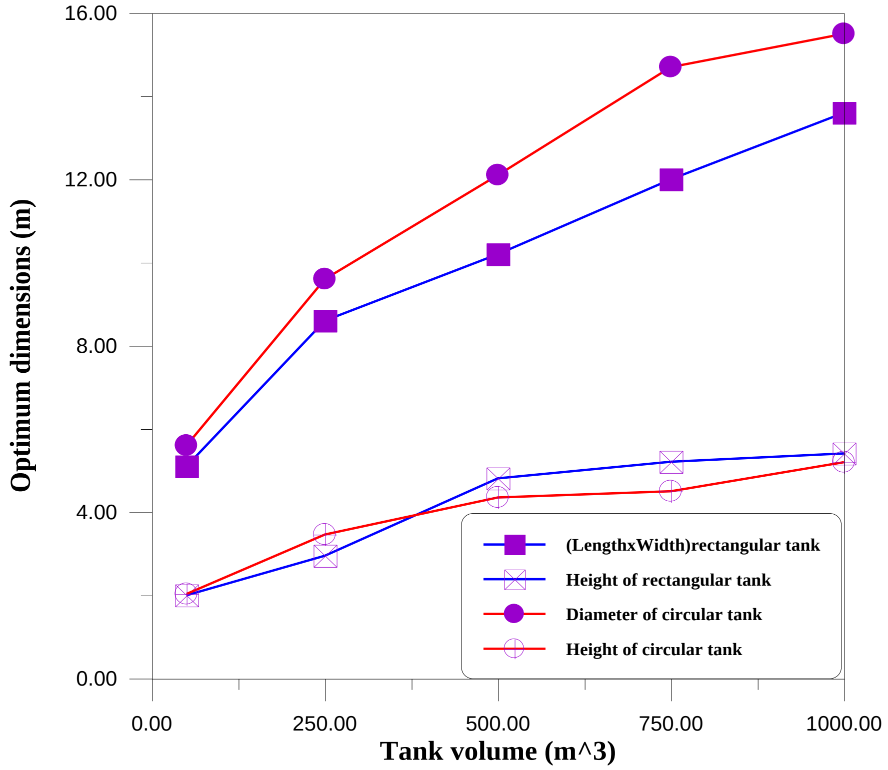


Fig.(5.10) Relation between the optimum dimensions and the considered tanks volumes for rectangular and circular partially-buried open tanks.

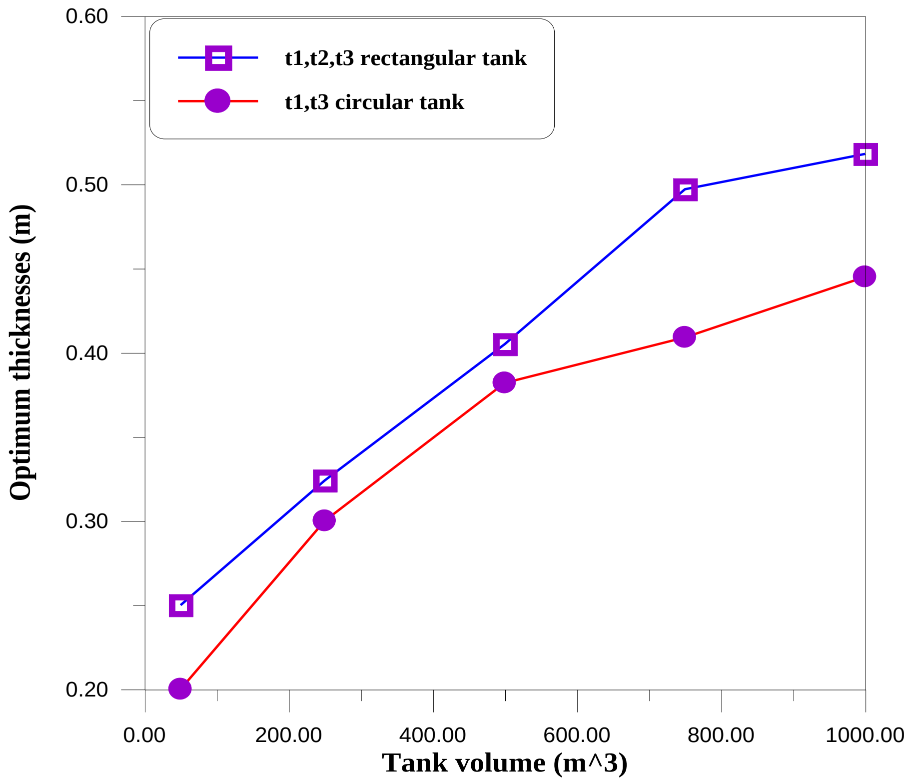


Fig.(5.11) Relation between the optimum thicknesses and the considered tanks volumes for rectangular and circular partially-buried open tanks.

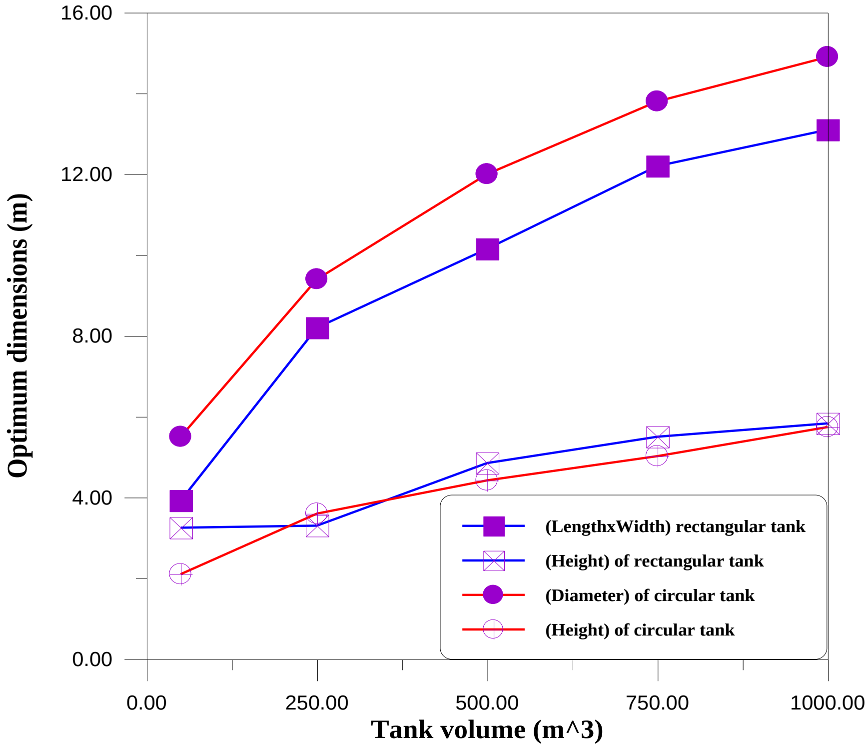


Fig.(5.12) Relation between the optimum dimensions and the considered tanks volumes for rectangular and circular fully-buried tanks.

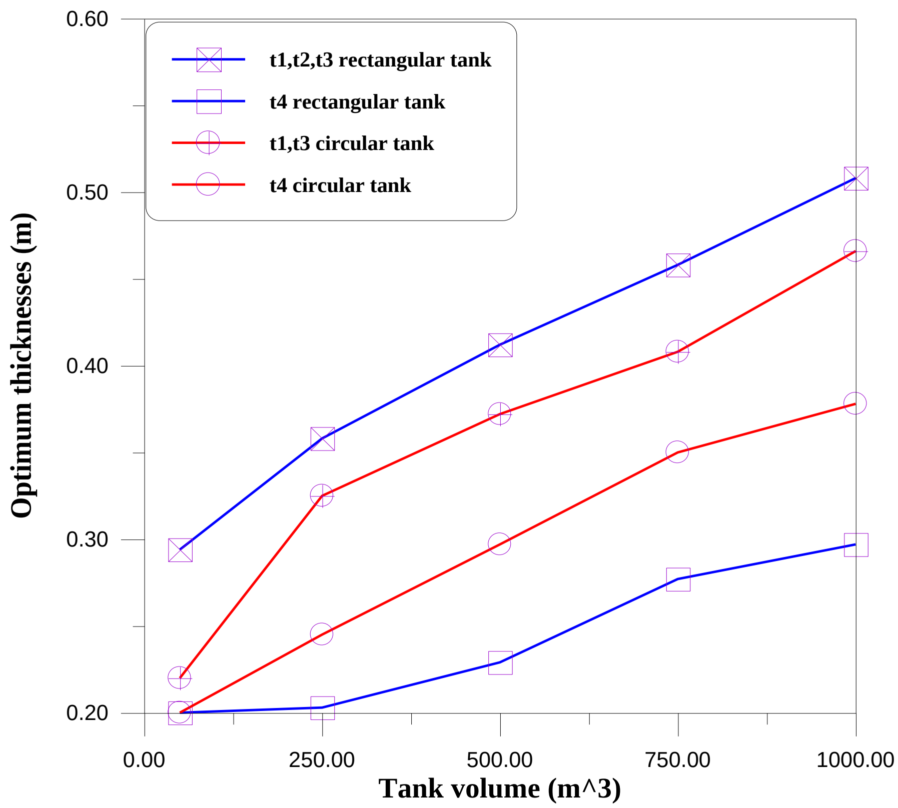


Fig.(5.13) Relation between the optimum thicknesses and the considered tanks volumes for rectangular and circular partially-buried closed tanks.

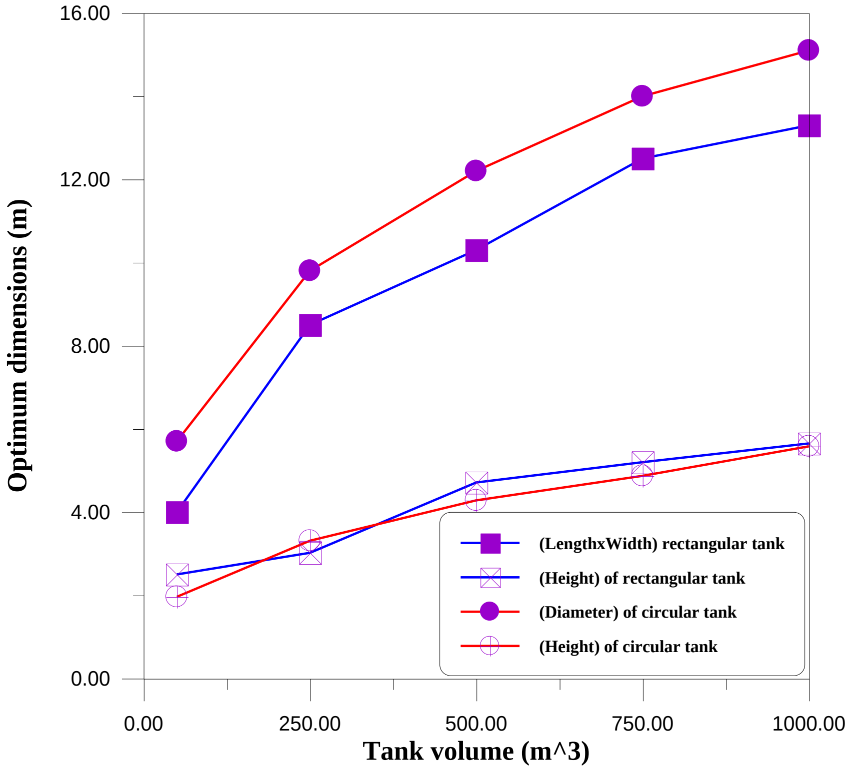


Fig.(5.14) Relation between the optimum dimensions and the considered tanks volumes for rectangular and circular partially-buried closed tanks.

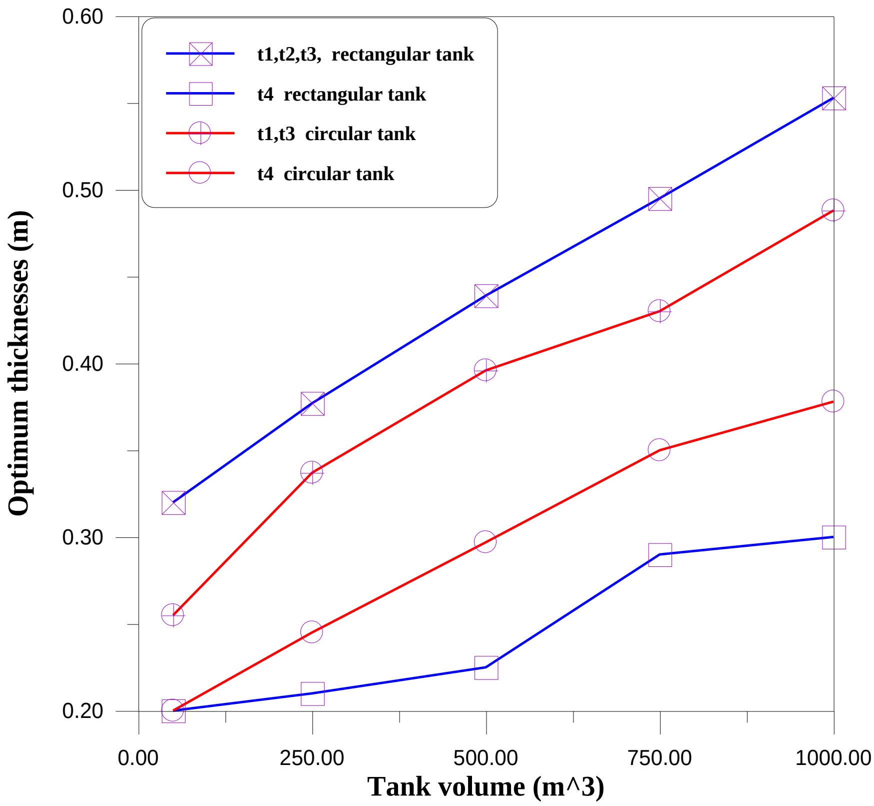


Fig.(5.15) Relation between the optimum thicknesses and the considered tanks volumes for rectangular and circular fully-buried tanks.

Table(5.2):Optimum dimensions, thicknesses, and costs for (Rectangular, on-ground, open) tanks.

Tank volume (m^3)	Length L (m)	Width B (m)	Height H (m)	Wall Thicknesses t_1, t_2 (mm)	Floor Thickness t_3(mm)	Cost (10^6 ID)
50	**5.1**	**4.9**	**2.0**	**200**	**200**	**8.443**
250	**9.2**	**9.2**	**2.95**	**280**	**280**	**36.268**
500	**10.2**	**10.2**	**4.81**	**351**	**351**	**63.817**
750	**12**	**12**	**5.21**	**403**	**403**	**90.614**
1000	**13.6**	**13.6**	**5.41**	**419**	**419**	**115.642**

Table(5.3):Optimum dimensions, thicknesses, and costs for (Rectangular, on-ground, closed) tanks.

Tank volume (m^3)	Length L (m)	Width B (m)	Height H (m)	Wall Thicknesses t1,t2,(mm)	Floor Thickness t3(mm)	Roof Thickness t4(mm)	Cost (10^6 ID)
50	**4.1**	**4.1**	**2.97**	**250**	**250**	**200**	**13.024**
250	**9.4**	**9.4**	**2.82**	**313**	**313**	**209**	**46.653**
500	**10.5**	**10.5**	**4.54**	**362**	**362**	**233**	**80.326**
750	**12.9**	**12.9**	**4.51**	**410**	**410**	**287**	**113.816**
1000	**13.7**	**13.7**	**5.33**	**438**	**438**	**304**	**146.910**

Table(5.4):Optimum dimensions, thicknesses, and costs for (Rectangular, partially-buried, open) tanks.

Tank volume (m^3)	Length L (m)	Width B (m)	Height H (m)	Wall Thickness t_1, t_2 (mm)	Floor Thickness t_3(mm)	Cost (10^6 ID)
50	4.9	4.9	2.08	261	250	12.925
250	9.0	9.0	3.09	324	324	47.476
500	10.5	10.5	4.54	405	405	88.445
750	11.7	11.7	5.84	479	479	130.325
1000	13.4	13.4	5.74	518	518	171.75

Table(5.5):Optimum dimensions, thicknesses, and costs for (Rectangular, partially-buried, closed) tanks.

Tank volume (m^3)	Length L (m)	Width B (m)	Height H (m)	Walls Thickness t_1,t_2,(mm)	Floor Thickness t_3(mm)	Roof Thickness t_4(mm)	Cost (10^6 ID)
50	4	4	3.125	294	294	200	16.1
250	9.1	9.1	3.02	358	358	203	60.595
500	10.3	10.3	4.71	412	412	229	104.581
750	12.5	12.5	4.8	485	485	277	151.733
1000	13.4	13.4	5.56	533	533	297	199.45

Table(5.6):Optimum dimensions, thicknesses, and costs for (Rectangular, fully-buried) tanks.

Tank volume (m^3)	Length L (m)	Width B (m)	Height H (m)	Wall Thicknesses t_1, t_2 (mm)	Floor Thickness t_3 (mm)	Cost (10^6 ID)
50	3.92	3.92	3.25	320	200	18.67
250	8.7	8.7	3.3	377	210	70.626
500	10.15	10.15	4.85	439	255	119.544
750	12.2	12.2	5.04	495	290	176.887
1000	13.1	13.1	5.83	552	300	220.288

Table(5.7):Optimum dimensions, thicknesses, and costs for (Circular, on-ground, open) tanks.

Tank volume (m^3)	Diameter D (m)	Height H (m)	Wall Thickness t_1 (mm)	Floor Thickness t_3 (mm)	Cost (10^6 ID)
50	5.6	2.03	200	200	10.4
250	9.6	3.46	300	300	42.319
500	12.1	4.35	322	322	72.119
750	14.7	4.42	375	375	102.646
1000	15.5	5.20	390	390	127.160

Table(5.8):Optimum dimensions, thicknesses, and costs for (Circular, on-ground, closed) tanks.

Tank volume (m^3)	Diameter D (m)	Height H (m)	Wall Thickness t_1 (mm)	Floor Thickness t_3 (mm)	Roof Thickness t_4 (mm)	Cost (10^6 ID)
50	**6**	**1.77**	**200**	**200**	**200**	**14.968**
250	**10.2**	**3.06**	**285**	**285**	**270**	**55.895**
500	**12.6**	**4.00**	**334**	**334**	**320**	**92.195**
750	**14.5**	**4.54**	**387**	**387**	**353**	**135.499**
1000	**15.6**	**5.23**	**408**	**408**	**375**	**168.354**

Table(5.9):Optimum dimensions, thicknesses, and costs for (Circular, partially-buried, open) tanks.

Tank volume (m^3)	Diameter D (m)	Height H (m)	Wall Thickness t_1 (mm)	Floor Thickness t_3 (mm)	Cost (10^6 ID)
50	**5.8**	**1.89**	**200**	**200**	**14.332**
250	**9.9**	**3.25**	**300**	**300**	**53.773**
500	**12.4**	**4.14**	**385**	**385**	**99.280**
750	**14.2**	**4.74**	**403**	**403**	**147.455**
1000	**15.7**	**5.23**	**445**	**445**	**191.284**

Table(5.10):Optimum dimensions, thicknesses, and costs for (Circular, partially-buried, closed) tanks.

Tank volume (m^3)	Diameter D (m)	Height H (m)	Wall Thickness t_1 (mm)	Floor Thickness t_3 (mm)	Roof Thickness t_4 (mm)	Cost (10^6 ID)
50	**5.7**	**1.96**	**220**	**220**	**200**	**18.976**
250	**9.8**	**3.31**	**325**	**325**	**245**	**66.957**
500	**12.2**	**4.28**	**372**	**372**	**297**	**115.653**
750	**14**	**4.87**	**410**	**410**	**350**	**168.842**
1000	**15.1**	**5.58**	**466**	**466**	**378**	**217.050**

Table(5.11):Optimum dimensions, thicknesses, and costs for (Circular, fully-buried) tanks.

Tank volume (m^3)	Diameter D (m)	Height H (m)	Wall Thickness t_1 (mm)	Floor Thickness t_3 (mm)	Roof Thickness t_4 (mm)	Cost (10^6 ID)
50	**5.5**	**2.10**	**255**	**255**	**200**	**21.337**
250	**9.4**	**3.60**	**337**	**337**	**245**	**77.596**
500	**12**	**4.42**	**396**	**396**	**301**	**128.992**
750	**13.8**	**5.02**	**416**	**416**	**365**	**181.357**
1000	**14.9**	**5.74**	**488**	**488**	**383**	**238.794**

Table(5.12):Optimum dimensions, thicknesses, and costs for (Rectangular, on-ground, open) tanks [using: qa=70 KN/m^2].

Tank volume (m^3)	**Length L (m)**	**Width B (m)**	**Height H (m)**	**Wall Thicknesses t_1, t_2,(mm)**	**Floor Thickness t3(mm)**	**Cost (10^6 ID)**
50	**4.9**	**4.9**	**2.05**	**200**	**200**	**7.473**
250	**9.2**	**9.1**	**2.99**	**223**	**223**	**32.635**
500	**9.6**	**9.6**	**5.43**	**311**	**311**	**56.447**
750	**11**	**11**	**6.2**	**370**	**370**	**79.113**
1000	**12.1**	**12.1**	**6.83**	**401**	**401**	**99.268**

Table(5.13):Relative effectiveness of the considered designs of the reinforced-concrete water-storage tanks.

Location and state		Storage volume (m^3)	Shape	Volume of reinforced concrete		Cost (1000 I.D.)	
				RRCV*	Rank**	EE***	Rank**
a	On-ground open	50	Rectangular	0.279	20	168.860	12
			Circular	0.256	13	208.000	26
		250	Rectangular	0.229	11	145.144	7
			Circular	0.221	7	177.276	16
		500	Rectangular	0.222	8	127.634	4
			Circular	0.189	2	144.238	6
		750	Rectangular	0.226	10	120.819	2
			Circular	0.196	3	136.861	5
		1000	Rectangular	0.208	4	115.642	1
			Circular	0.185	1	127.160	3
b	On-ground closed	50	Rectangular	0.460	48	260.480	41
			Circular	0.390	45	299.680	45
		250	Rectangular	0.336	38	186.540	19
			Circular	0.315	32	223.580	31
		500	Rectangular	0.290	24	160.652	10
			Circular	0.280	21	184.390	18
		750	Rectangular	0.291	25	151.182	9
			Circular	0.287	23	180.665	17
		1000	Rectangular	0.282	22	146.910	8
			Circular	0.270	18	168.354	11
c	Partially-buried closed	50	Rectangular	0.321	35	258.500	40
			Circular	0.259	15	286.640	44
		250	Rectangular	0.271	19	189.904	20
			Circular	0.225	9	215.092	28
		500	Rectangular	0.257	14	176.890	15
			Circular	0.231	12	198.560	23

		750	Rectangular	0.262	16	173.767	14
			Circular	0.219	6	196.607	22
		1000	Rectangular	0.268	17	171.750	13
			Circular	0.210	5	191.284	21
d	Partially-buried closed	50	Rectangular	0.461	49	322.000	47
			Circular	0.403	46	379.500	49
		250	Rectangular	0.377	44	248.380	38
			Circular	0.344	40	267.828	42
		500	Rectangular	0.319	34	209.162	27
			Circular	0.301	29	231.306	33
		750	Rectangular	0.318	33	202.310	25
			Circular	0.292	26	225.123	32
		1000	Rectangular	0.362	42	199.450	24
			Circular	0.295	28	217.050	29
e	Fully-buried	50	Rectangular	0.487	50	372.000	48
			Circular	0.443	47	426.740	50
		250	Rectangular	0.368	43	282.504	43
			Circular	0.329	36	310.384	46
		500	Rectangular	0.338	39	239.088	36
			Circular	0.310	31	257.984	39
		750	Rectangular	0.335	37	235.585	34
			Circular	0.293	27	241.809	37
		1000	Rectangular	0.346	41	220.288	30
			Circular	0.302	30	238.794	35

*RRCV=relative reinforced concrete volume(cubic meter of reinforced concrete per one cubic meter of stored water).

** Rank = out of (2x5x5=50 cases under investigation).

***EE = economic efficiency =(I.D. per one cubic meter of stored water).

5.6: ANALYSIS OF THE RESULTS

Recalling that all the designed tanks are structurally safe, the results contained in Tables (5.2) through (5.13) and illustrated in Figs. (5.1) through (5.15) indicate the following:

[A] With respect to volume

1. It is clear from Figs. (5.1) through (5.5) that the optimum total cost (ZT) of all selected volumes of the case study of the reinforced concrete tanks increases progressively and almost linear with increasing the tank volume (V). In fact, this is a logical and an expected result.
2. Table (5.13) indicates that the economic efficiencies (EE) of a specified shape and state decreases with the increasing of the tank volume (V).
3. Table (5.13) indicates also that the relative reinforced concrete volume (RRCV) of a specified shape and state decreases with the increasing of volume (V).

[B] With respect to location

1. Tables (5.2) through (5.11) indicate that the dimensions and thicknesses of the fully-buried tanks are greater than that of the partially-buried and of the on-ground reinforced concrete tanks. This result is due to the higher loads and moments applied on the fully- buried tanks.
2. It is clear from Figs. (5.1) through (5.5) and from Tables (5.2) through (5.11), that the optimal total cost (ZT) of the buried tanks (partially and fully) is greater than that of the on-ground tanks for the same tank volume (V). This result is due to the additional cost of excavation and fillworks, beside the increments in the dimensions and thicknesses of the tank constituents.

3. Table (5.13) shows that the effective efficiencies (EE) of the fully-buried tanks are greater than those of the corresponding partially-buried ones which, in turn, are greater than those of the corresponding on-ground tanks.
4. Table (5.13) also shows that the relative reinforced concrete volumes (RRCV) of the fully-buried tanks are greater than those of the partially-buried and the on-ground tanks.

[C] With respect to shape

1. For all volumes considered in the case study, all the assumed rectangular tank sections came to be square in all optimum results for all considered states of the reinforced concrete tanks, as given in Tables (5.2) through (5.6).
2. For all volumes considered in the case study, the optimum total cost (ZT) and the effective efficiencies (EE) of the circular tanks are greater than those of the rectangular tanks as shown in Figs. (5.1) through (5.5), Tables (5.2) through (5.11), and Table (5.13).
3. It is clear from Table (5.13) that the relative reinforced concrete volumes (RRCV) of the circular tanks are less than those of the rectangular tanks.

[D] A compact summary in this respect is:

1. The optimal total cost (ZT) for all volumes and locations considered in the case study went to the square shape because of the low constructional cost than the circular tank.
2. The optimization process revealed that the circular tanks come always lag as compared to the rectangular ones. For example, ($V=250\ m^3$) , the optimum sections of the circular on-ground open tank were (D=9.60 m, H=3.46 m) for which the total cost(ZT= $44.319\text{x}10^6$ I.D.), whereas for the same volume and location, the optimum sections of the rectangular shape were (L=9.20 m, B=9.20 m, H=2.95 m) for which the total cost (ZT= $38.286\text{x}10^6$ I.D.).

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1: INTRODUCTION

In this research, a total of (50) cases for reinforced-concrete tanks for water-storage have been taken as the case study, namely, (5) cases for location and state, each with (5) proposed storage volumes, each with two tanks shapes, aiming at attaining the optimum design. The results revealed the following.

6.2: CONCLUSIONS

1. The computer program for the optimization processes has been found suitable in giving the final safe results.

(2) The optimal results for the considered cases of water-storage, reinforced concrete tanks are summarized in Tables (6.1) and (6.2).

Table (6.1): Optimum dimensions of rectangular tanks.

Location and state	Storage volume	Optimum dimensions					
		Length L,(m)	Width B,(m)	Height H,(m)	**Walls Thickness t_1,t_2,(mm)**	**Floor Thickness t_3(mm)**	**Roof Thickness t_4(mm)**
On-ground open	50	5.20	4.90	2.0	200	200	
	250	9.20	9.20	2.95	280	280	
	500	10.20	10.20	4.81	351	351	
	750	12.00	12.00	5.21	403	403	
	1000	13.60	13.60	5.41	419	419	

Location and state	Storage volume	Optimum dimensions					
		Length L,(m)	Width B,(m)	Height H,(m)	**Walls Thickness t_1,t_2,(mm)**	**Floor Thickness t_3(mm)**	**Roof Thickness t_4(mm)**
On-ground closed	50	4.10	4.10	2.97	250	250	200
	250	4.90	4.90	2.82	313	313	209
	500	10.50	10.50	4.54	362	362	233
	750	12.90	12.90	4.51	410	410	287
	1000	13.70	13.70	5.33	438	438	304
Partially-buried open	50	4.90	4.90	2.08	261	250	
	250	9.00	9.00	3.09	324	324	
	500	10.50	10.50	4.54	405	405	
	750	11.70	11.70	5.84	479	479	
	1000	13.40	13.40	5.74	518	518	
Partially-buried closed	50	4.00	4.00	2.50	294	294	200
	250	9.10	9.10	3.02	358	358	203
	500	10.30	10.30	4.71	412	412	229
	750	12.50	12.50	4.80	485	485	277
	1000	13.40	13.40	5.56	533	533	297
Fully-buried	50	3.92	3.92	3.25	320	320	200
	250	8.70	8.70	3.30	377	377	210
	500	10.15	10.15	4.85	439	439	255
	750	12.20	12.20	5.04	495	495	295
	1000	13.10	13.10	5.83	552	552	310

Table (6.2): Optimum dimensions of circular tanks.

Location and state	Storage volume	Optimum dimensions				
		Diameter D,(m)	Height H,(m)	**Wall Thickness t_1, (mm)**	**Floor Thickness t_3(mm)**	**Roof Thickness t_4(mm)**
On-ground open	50	5.60	2.03	200	200	
	250	9.60	3.46	300	300	
	500	12.10	4.35	322	322	
	750	14.70	4.42	375	375	
	1000	15.50	5.20	390	390	
On-ground closed	50	6.00	1.77	200	200	200
	250	10.20	3.06	285	285	270
	500	12.60	4.00	334	334	320
	750	14.50	4.54	387	387	353
	1000	15.60	5.23	408	408	375
Partially-buried open	50	5.80	1.89	200	200	
	250	9.90	3.25	300	300	
	500	12.40	4.14	385	385	
	750	14.20	4.74	403	403	
	1000	15.70	5.23	445	445	
Partially-buried closed	50	5.70	1.96	220	220	200
	250	9.80	3.31	325	325	245
	500	12.20	4.28	372	372	297
	750	14.00	4.87	410	410	350
	1000	15.10	5.58	466	466	378

Location and state	Storage volume	Optimum dimensions				
		Diameter D,(m)	Height H,(m)	**Walls Thickness t_1, (mm)**	**Floor Thickness t_3(mm)**	**Roof Thickness t_4(mm)**
Fully-buried	50	5.50	2.10	255	255	200
	250	9.40	3.60	337	337	255
	500	12.00	4.42	396	396	301
	750	13.80	5.02	416	416	365
	1000	14.90	5.74	488	488	383

3. The optimal total cost (ZT) for all volumes and locations considered in the case study went to the square shape
4. Increasing the allowable bearing capacity of the soil will decrease the horizontal dimensions and leads to decrease the tank cost for the same volume (V).
5. The optimization processes automatically excluded the circular tanks shape For example, (V=250 m^3) , the optimum sections of the circular on-ground open tank were (D=9.30 m, H=3.46 m) for which the total cost(ZT= 44.319x10^6 I.D.), whereas for the same volume and location, the optimum sections of the rectangular shape were (L=9.20 m, B=9.20 m, H=2.95 m) for which the total cost (ZT= 38.286x10^6 I.D.).

6.The effective efficiency (EE) of a specified state decreases with the increasing of the reinforced concrete tanks volumes (V).

6.3: RECOMMENDATIONS

The following are recommended for further studies:

1. Investigating the effect of ground-water on the buried tanks.
2. Studying the optimum design of multi-cell reinforced concrete tanks.
3. Solving the optimization problem by techniques other than the one used in this research.
4. Studying the design of tapered wall reinforced concrete tanks.

REFERENCES

Abdul-Hussain, K. [1985]: "Optimum Design of Structures". M.Sc. Thesis, University of Technology, Iraq.

ACI-318R (American Concrete Institute) [1963]: ”Building Code Requirements For Structural Concrete ", Structural Bureau, Chicago. USA.

ACI-350R (American Concrete Institute) [1989]: ”Environmental Engineering Concrete Structures", Structural Bureau, Chicago. USA.

ACI-504R (American Concrete Institute) [1997]: ”Guide to Sealing Joints in Concrete Structures ", Structural Bureau, Chicago. USA.

ACI-318R (American Concrete Institute) [2005]: ”Building Code Requirements For Structural Concrete ", Structural Bureau, Chicago. USA.

Adidam, S. R, and Subramanyam, A.V.[1982]: "Optimum Design of Reinforced Concrete Water Tanks". Journal of The Structural Division, ASCE, Vol. 108, No.ST6 .p.p.(1219-1231), June.

Ajaam, A. H. A. [2006]: "Optimal Design of Steel Rectangular Tanks on Elastic Foundation Based on Non-Linear Analysis". M.Sc. Thesis, University of Babylon, Iraq.

Anson, M. and Parker, T.F. [1965]: " A Numerical Technique for Analysis of Reinforced Concrete Circular Tank Walls of Various Cross Sections", Journal of Institution Engineers, April-May, Vol. No. 23, pp.(103-109).

Barenberg, J. and Allison, D. [2003]: " Design and Construction Of Liquid-Tight Concrete Structures", Technical Brief of Improving Performace, Colorado. Internet at www.google.com.

Buchi, J. E, [1938]: "Design of Walls of Open Rectangular Tanks". Concrete and Construction Engineering, Vol. 13, p.p (183-191), July.

Bundy, B. D. [1984]: "Basic Optimization Methods". Edward Arnold, London.

Carpenter, W. C. and Smith, E. A. [1975]: "Computational Efficiency in structural Optimization". Journal of Engineering Optimization., Vol.1, p.p.(169-188).

Case, J., Carls, T. F., and Chilver, L. [1999]: " Strength of Materials and Structures". 4th edition, John Wiley & sons Book Comp., ISBN(0 470 37980 6).

Cheung, Y. K. and Davies, J. D. [1967 (a)]: " The Analysis of Rectangular Tanks use of Finite Element Method Technique". Concrete and Construction Engineering, Vol. 26, p.p. (169-174), May.

Cheung, Y. K. and Davies, J. D. [1967 (b)]: " Bending Moment in the Long Walled Tanks". Journal of the American Concrete Institute, p.p.(685-689), October.

Cheung, Y. K. and Zeinkiewicz, O. C. [1965]: "Plates and Tanks on Elastic Foundation: An Application of Finite Element Method". International Journal of Solids and Structures, Vol.1,p.p.(451-461). May.

Craemer, H. [1953]: "Bending Moments in Box-Shaped Reservoirs". The Structural Engineering, p.p.(177-178), July.

DAAF (Department of the Army and the Air Force) [1985]: "Water Supply, Water Storage". Technical Manual, Vol.4, September.

Dantzing, C. B. [1963]: "Linear Programming and Extensions". Princeton University Press, New York, USA.

Davies, J. D. [1961]: " Cylindrical Concrete Tanks on Elastic Media". ASCE, Vol.56, p.p.(61-64), January.

Davies, J. D. [1962]: "The Influence of Support Condition on the Behavior of Cylindrical Concrete Tanks". Proceeding of the Institution of Civil Engineering, Paper No. 6590, p.p.(379-388), London, England.

Davies, J. D., Cheung, Y. K., and Gorecki, A. [1970]: " Analysis of Long Rectangular Tanks". Water and Water Engineering, p.p.(510-516), December.

Duffin, R., Peterson, E., and Zener, C. [1967]: "Geometric Programming: Theory and Application". John wiley & Sons, New York, USA.

Fredrick, S. M., Jonathan, T. R., and Mark, S. R. [2001]:"Building Design and Construction Hand Book ". 6th edition, Mc Graw hill, New York.

Fiacco, A. V., and McCormick, G. P. [1968]:"Non-linear programming Sequential Unconstrained Minimization Technique ". John wiley & Sons.

Gallagher, R. H., and Zeinkiewicz, O. C. [1973]: "Optimum Structural design, Theory and Applications" John Wiley and Sons, London.

Ghali, A. [1957]: "The Structural Analysis of Circular and Rectangular Concrete tanks", Ph.D. Thesis. University of Leeds, England.

Ghali, A. [1958]: "Analysis of Cylindrical Tanks with Flat Bases by Moment Distribution Method" The Structural Engineering,vol.36, No.5, pp.(165-176), May.

Ghali, A. [1979]: "Circular Storage Tanks and Silos". Spon LTD Co. USA.

Hamdi, M. J.[1998]: "'Optimum Design of R/C Cylindrical Tanks Resting on Elastic Media". M.Sc. Thesis, University of Baghdad, Iraq.

Hooke, R. and Jeeves, T. A. [1961]: "Direct Search Solution of Numerical and Statistical Problems". Jour. of Ass. Computer Mathematical, Vol.8,p.p.(212-229), USA.

Hussain, H. M. [1964]: " The Analysis of Rectangular Plates and Cellular Structures". Ph.D, Thesis, University of Leeds, England.

Iyengar, N.G.R. [2000]: "Optimization in Structural Design", Indian Institute of Technology, Kanpur, India. Internet at www.google.com.

Jaiswal, O. R., Rai, D. C., and Jain, S. K. [2003]: "Review of Cod Provisions on Seismic Analysis of Liquid Storage Tanks", Department of Civil Engineering, Indian Institute of Technology, Kanpur, India,.

Al-Kenany, M.S. [1996]: "Analysis of Cylindrical Cellular Storage Tanks by the Method of Grillage" M.Sc. Thesis, University of Baghdad, Iraq.

Kimence, B. and Erguven, M. E. [1992]: "Interaction between Cylindrical and Shallow Spherical Shells on the Vlasov Model Foundation", Computer and Structures, vol.45, No.1, pp.(41-51), USA.

KM (Karbala market), [2009]: {Personal connections}.

Lightfoot, E. and Ghali, A. [1958]: "The Analysis of Rectangular Tanks". Proceeding to the Institution of Structural Engineering, London, U.K.

Lightfoot, E. and Sawko, F. [1960]: "The Analysis of Grid Frameworks and Floor Systems by the Electronic Computer". The Institution of Structural Engineer, March, p.p.(79-87), London, U.K.

Mahdi, T.H. [1989]:"Improved Analysis Methods for Soil-Raft Interaction", M.Sc. Thesis, University of Baghdad, Iraq.

Meichoers, R. and Roznavy, G. [1970]:"Optimum Design of R/C Tanks". Journal of The Engineering Mechanics Divisions, ASCE, Vol.96, No. EM. p.p. (1093-1106), December.

Al-Muhaidi, Riadh S. [1986]: "Stiffness Analysis of Cylindrical Concrete Tanks Resting on Elastic Media". Proceeding of Fourth Scientific Conference, Scientific Research Council, Vol 4, Part 1, Baghdad, Iraq.

Al-Ne'aimi, A. A. H, and Al-Sabah, A. I. [1988]: " Optimum Design of R/C Cylindrical Tanks With Tapered Walls". Proceeding of the 2nd Iraqi Engineering Conference, University of Mosul, Engineering College, 1-3 November, Iraq.

Al-Ne'aimi, A. A. H, and Al-Sabah, A. I. [1989]: " Minimum Cost Design of Underground R/C Cylindrical Tanks ". Proceeding of the 5th Iraqi Engineering Conference, Vol. 4, Part2, 7-11October, Iraq.

Nilson, A. H., Darwin, D., and Dolan, W. C. [2004]: "Design of Concrete Structures", 14th Edition, Mc Graw hill Book Comp. Inc, New York, USA.

PCA (Portland Cement Association) [1951]: "Rectangular concrete tanks " . USA.

PCA. (Portland Cement Association). [1969]: "Rectangular concrete tanks " .ISBN(0-89312-125-8), USA.

PCA (Portland Cement Association) [1993]: "Circular concrete tanks without prestressing" . USA.

PCA. (Portland Cement Association). [1996]: "Underground concrete tanks " . USA.

Philips, D. T, **Ravindran, A., and James, J. [1976]:** "Operation Research Principles and Practice". John Wiley and Sons, Canada.

Rosen, J. B. [1960]: "The Gradient Projection Method of Non – Linear Programming "SIAM Journal, Vol.8, PP.(181-217),USA.

Rosenbrock, H. H. [1960]: "An Automatic Method for Finding the Greatest or Least Value of a Function". Comput. Journal, Vol. 3, .p.p.(175-184), USA.

Al- Sarrai, Basheer A. M. [1988]: "Analysis of Non-Prismatic Cylindrical Concrete Tanks Resting on Elastic Media". M.Sc, Thesis, University of Baghdad, Iraq.

SCDPHDWS. **(State of Connecticut Department of Public Health Drinking Water Section) [2005]**:"Storage Tank Design and Construction Guidelines" . USA. Internet at ww.google.com.

Slaby, A. A. H. [1987]: "Optimum Design of Reinforced Concrete Frames". M.Sc. Thesis, College of Engineering, University of Baghdad, Iraq.

Starczewski, M. [1981]: " Non-Circular Pressure Vessels: Some Guidance Notes for Designers". British Engineering Technical Report, Vol XIV, England.

Sultan, B.A. [1988]: "Analysis of Non-Prismatic Cylindrical Tanks Resting on Elastic Media", M.Sc. Thesis, Department of Civil Engineering, University of Baghdad, Iraq.

Syal, L. C., Goel, A. K., and Saha, M. B. [2005]: "Reinforced Concrete Structures'. 5th edition, Jhon Wiley & Sons, New York.

Thevendran, V. and Thambiratnam, D.P [1986]: "Minimum Weight Design of Cylindrical Water Tanks", International Journal for Numerical Methods in Engineering, Vol.23, No.3, pp.(379-383), USA.

Timoshenko, S. and Woinowsky-Kriger. S. [1959]: "Theory of Plates and Shells". Mc Graw hill book comp. Inc, New York, USA.

Tin-Loi, F., Pulmano, V.A., and Thambiratnam, D.P. [1990]: "Analogy for Axisymmetrically Loaded Cylindrical Shells", Int. J., Comput. Struct., Vol34, No.2, pp.(281-285), USA.

Uraiby, S. A. [1997]: "'Optimum Design of R/C Rectangular Tanks on Elastic Foundation". M.Sc. Thesis, University of Baghdad, Iraq.

USEPA. (United States Environmental Protection Agency) [2002]: "Finished Water Storage Facilities ".Ground Water and Drinking Water Standards and Risk Management Division, Prepared by (AWWA) with Assistance from Economic and Engineering Services, Inc, Washington, USA.

Young, D. [1943]: "Bending Moment in the Walls of Rectangular Tanks". Transactions, ASCE, Vol.108, Paper No. 2176, p.p.(217-230), USA.

التصميم الامثل الامين للخزانات الكونكريتية

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الخلاصة

نظرا لأهمية الخزانات من الخرسانة المسلحة في حياة البشر ولما تمثله من كلفة كبيرة, فقد تم اختيار التصميم الأمثل الأمين لهذه الخزانات موضوعا للبحث, وتم استخدام طريقة توزيع العزوم في التحليل, اخذين بالاعتبار كلفة كل من الحفر وطبقة التعمية وطبقة الإسفلت العازل والمواد الإنشائية المتكون منها جسم الخزان والتي تشتمل على (أعمال القالب وأعمال حديد التسليح والمواد المانعة للتسرب والخرسانة المتكون منها جسم الخزان) وأخيرا أعمال الدفن.

بالنظر لتعدد أنواع وأشكال الخزانات فقد تم اختيار الأوسع انتشارا واستعمالا منها وهي الخزانات المستطيلة الشكل (منها المربع كحالة خاصة) والخزانات الدائرية. إما بالنسبة للموقع فهي أما أن تكون فوق الأرض أو مدفونة (كليا أو جزئيا). وكل هذه الأنواع المذكورة آنفا أما أن تكون مفتوحة او مغلقة.

لربط الحالة العملية بالنظرية فقد انتخبت الحجوم (50, 250, 500, 750, 1000 م^3) لتمثل الحجوم الصغيرة والمتوسطة والكبيرة.

ولغرض تهيئة نموذج الامثلية. فقد تم إعداد دالة الهدف لتغطي الكلف المذكورة. وقد اعتمد نموذج الامثلية حجم المنشأ كدالة للهدف واعتبر الأبعاد هي متغيرات القرار مع استخدام محددات هندسية. وتم حل نموذج الامثلية اللاخطية باعتماد طريقة المحاولة والتكرار. حيث تم تطوير برنامج حاسوب متضمنا كل ما تم ذكره لغرض الحصول على النتائج.

تم تحليل النتائج في ضوء ما يلي:

1. امثل الأبعاد لمختلف الحجوم للخزانات.
2. امثل الأبعاد بالنسبة لمستوى قعرالخزان من سطح الأرض.
3. امثل الأبعاد بالنسبة لشكل الخزان.

لقد ثبتت النتائج ما يلي

1. طريقة حل مسألة الامثلية كفوءة في إعطاء النتائج الأمينة.
2. استبعدت عملية الامثلية الخزانات الدائرية حيث لنفس الحجم نجد أن الكلفة للخزان المربع اقل من كلفة الخزان الدائري, فمثلا للخزانات الجالسة فوق الارض المفتوحه ولحجم ($250\ M^3$) نجد ان

الابعاد المثلى للخزان الدائري هي (القطر= 9.6 M) و(الارتفاع= 3.46 M) اما الكلفة االكلية فهي ($44.319*10^6$ دينارعراقي) بينما للخزان المربع ولنفس الحجم فكانت الابعاد المثلى هي (الطول=9.2 M) و (العرض=9.2 M) و (الارتفاع=2.95 M) اما الكلفة الكلية فكانت ($38.286*10^6$ دينارعراقي)

3. الكلفة النسبية لكل خزان بالنسبة لمستوى قعر الخزان من الأرض تقل مع زيادة الحجم للخزان.
4. الشكل المربع للخزان هو الأمثل لكل الحجوم ولكل المواقع بالنسبة لقعر الخزان.

Curriculum Vitea

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