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7th ARCHENG-2012 International Architecture and Engineering Symposiums

Abstract of the Addressing by Board of Trustees' Chairman

Dear Guests, Colleagues, and Friends,

May I take this opportunity, as Chairman of the Board of Trustees, to welcome you to our university, the European University of Lefke, and to our Symposium, the Architecture and Engineering Symposium. Organized and hosted by the European University of Lefke, the 2012 Architecture and Engineering Symposium is the 7th of its kind. This international symposium has been running for 12 years now, and was first organized in the year 2000 to mark 10 years since the establishment of the European University of Lefke. The phrase "Creating the Future" has become our Symposium's motto, and in fulfilling this motto, the Symposium has endeavored to encourage the rapid dissemination of research contributions that push back the frontiers of technology in the areas of both Architecture and Engineering.

I am a Civil Engineer by profession, and so I am well aware of the importance of providing our students with the necessary skills and knowledge for a successful career in both Engineering and Architecture. It is of paramount importance to our local community, as well as to our society.

Since the establishment of the university, over 20 years ago, we have seen a large number of developments in the university, including the addition of Faculties, Schools and Departments, as well as improvements to the infrastructure and faculties of the university. Recently the University's scientific and engineering laboratories have had a major overhaul, and I am pleased to say that EUL is now equipped with some of the most sophisticated and useful scientific equipment our students could wish for.

The Faculty of Architecture and Engineering at EUL has made significant strides in moving this institution forward. Their vision is to compete to be the best faculty of its kind in the country. To this end, it has increased its scientific contributions to the body of knowledge in Engineering and Architecture. This has been achieved by increasing the number of publications of scientific articles in prestigious journals; and through paper presentations by Faculty members in some of the most important international conferences in the world. Our staff are also involved in various computer and electronic engineering projects in the Turkish Republic of Northern Cyprus. Civil engineering professors are involved in TUBITAK projects, and our architecture staff has been involved in a number of architecture workshops.

Many of the undergraduate degree programs offered by our University have been accredited by other Nation's Governments and prominent educational organizations and bodies. This to me is testament to the fact that EUL is a world class university with world class degree programs. Our students can most definitely be assured that an education from the European University of Lefke is one that will stand them in good stead for whatever they decide to do in the future.

I trust that you will enjoy your time with us at this symposium, and I am sure that you will be able to take away with you lots of inspiration and ideas for future research.

Mr. Mehmet ZAFER
Chairman of Board of Trustees
European University of Lefke

7th ARCHENG-2012 International Architecture and Engineering Symposiums

Abstract of Opening Speech by Rector

Dear friends and colleagues from North Cyprus, Turkey, and the rest of the world. On behalf of the Steering, Organizing and Scientific Committees, and the European University of Lefke which is hosting this event, I warmly welcome you to Lefke, and to this Symposium - our 7th Architecture and Engineering Symposiums - "Creating the Future". Our symposiums cover the three major disciplines of our Faculty of Architecture and Engineering, namely Architecture and Civil Engineering, Electrical Engineering, and Computer Engineering.

Since we last met in 2010, there have been many developments within the Faculty of Architecture and Engineering. As well as the faculty growing, with new talented staff, we have passed some major milestones. We have received accreditation from the Pakistan Engineering Council (PEC) for our undergraduate degrees in Computer Engineering and Electrical Engineering. We have also started a PhD program in Computer Engineering, and have our first intake of PhD students this semester. Our university has become an authorized CUDA teaching centre. Our staff is currently engaged in research using the CUDA GPU from NVIDIA in our supercomputer facility. Our objective is to teach students parallelization of computer intensive algorithms using the new generation computing hardware.

Our first symposium was held in the year 2000. Twelve years later, our current symposium deals with the timely and challenging theme, "Creating the Future". Without research and innovation, the economies of the world would stagnate, so as engineers and scientists in the research community, we have our part to play in creating the future world of our nations. As educators, we also create the workforce of tomorrow. Including research in our daily activities at the European University of Lefke allows us to teach at the highest academic level and our students take advantage of this. I would like to offer my sincere congratulations to the staff of our Faculty of Architecture and Engineering for their earnest efforts and contributions to academic and research growth at our university.

I wish for our Faculty a long future with exciting research and much collaboration. I would like to convey a special thank you to all the presenters, distinguished scholars, and younger researchers that are taking part in our symposium. Finally, I would like to express my cordial thanks to the Scientific Committee and the Organizing Committee of each Symposium, for their tireless work in the preparation of the symposium, and the smooth running of our symposium throughout its duration.

Thank you everyone for listening, I hope that you will enjoy our symposium, and take with you some new ideas for your own organizations and your own research.

Prof. Dr. Ahmet Bülend Göksel
Rector
European University of Lefke.

7th ARCHENG-2012 International Architecture and Engineering Symposiums

Abstract of Welcome Speech by General Chair

In our efforts to organize our traditional biennial international symposium series on Architecture and engineering now we have moved on to conduct the 7th symposium series in the campus of European University of Lefke. As in the past, this 7th ARCHENG international symposium hosts three symposiums, namely, EEECS'12- International symposium on Electrical and computer Systems, CENG'12- International Symposium on Civil and Environmental Engineering and ARCH'12- International Symposium on Architecture and Interior Architecture. While the first international symposium was conducted when celebrating the 10th anniversary of European University of Lefke, in Nov 2000, the previous 6th ARCHENG international symposium was conducted during Nov 2010 on the celebration of 20th anniversary of EUL.

This symposium series is favoring the research contributions of academics and researchers in the chosen fields of Architecture and Engineering. The objective of each symposium is to bring together the researchers in the emerging fields of engineering or architecture providing them with a forum where they can present their current work leading to innovations for future inventions and thereby contributing to the theme of the symposium, "Creating the Future". We have gained experience in the past symposiums and we trust that we have contributed to science.

Tremendous growth of technology in electronics, computer and communication engineering has been witnessed globally and we have a pleasure to have a tiny share from our symposiums. The growth in building construction technology is also remarkably increasing where improved building materials and newer recycled materials are being launched. Today's research we do will come to market tomorrow and will reach the common man. Therefore, the education must be updated to keep in pace with the research outcomes setting new trends.

In the campus, IEEE chapter and ACM club have been taking active parts in organizing talks, attending special lectures and undertaking projects as to stimulate research minds in the young engineers of EUL. The student community is gaining academic and research maturity by these extra curricular activities. This symposium series is hoping to concede another step in their growth of knowledge and research experience for the challenges they face tomorrow.

On behalf of the organizing committees of the symposium series I express my gratitude to authors who submitted their innovative works based on analogical thinking, case studies and problems that require solutions in the chosen fields of the symposium. We extend our hearty welcome to all participants and contributing authors who have chosen to come over here to take part in the presentations and discussions. We are thankful to the reviewers for their support in exercising their responsibilities to choose the quality papers for presentation and for inclusion in the proceedings of the symposium. We thank all our co-sponsors who have associated with us in organizing this symposium with their contributions. We hope you have an interesting and thought provoking experience with us, and that we will all come together again for future symposiums.

Prof. Dr. K.Balasubramanian
General Chair, 7th ARCHENG-2012 & Dean, FAE

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Theme 3: Construction Management and Education in Civil Engineering

Theme 4: Hydraulics and Environmental Engineering

PERFORMANCE TESTING OF HUNTING SEARCH ALGORITHM IN FINDING THE OPTIMUM SOLUTION OF ENGINEERING DESIGN PROBLEMS

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***Abstract* —This study presents a hunting search based optimum design algorithm for the solution of benchmark problems. Hunting search algorithm is a numerical optimization method inspired by group hunting of animals such as wolves, lions, and dolphins. Each of these hunters performs hunting in a different way. However, they are common in that all of them look for a prey in a group. Prey is encircled and the ring of siege is tightened gradually until it is caught. Hunting search is employed for the automation of optimum design process, during which size variables are selected in such a way that the objective function value of the design problem is the minimum and the design constraints are satisfied. Two numerical design examples namely welded beam design problem and spring design problem are solved by the presented algorithm to demonstrate its efficiency. These results are then compared with the ones obtained with particle swarm and harmony search algorithms. Results reveal that hunting search shows great performance compare to the other stochastic search techniques taken into account in the present study.**

Keywords: : Metaheuristic search techniques, harmony search algorithm, particle swarm method, hunting search algorithm, optimization problems.

1. INTRODUCTION

In recent years, as an alternative to mathematical programming based methods, several meta-heuristic or evolutionary algorithms have been developed, which combine rules and randomness by mimicking natural phenomena, including biological evolutionary processes (evolutionary algorithm and genetic algorithms) [1, 2] animal behavior and intelligence of swarms (ant colony and particle swarm optimizers) [3, 4], the physical annealing process (simulated annealing) [5] and the musical process of searching for a perfect state of harmony (harmony search) [6]. The aim of researchers introducing these methods is to overcome above mentioned shortcomings of traditional mathematical programming techniques in solving optimization problems. What makes these techniques quite robust and simple compared to other classical methods is the fact that they do need neither the gradient information nor the convexity of the objective function and constraints functions. The optimum structural design algorithms based on these techniques are quite effective in finding the solution of discrete programming problems. The common features of these algorithms are that they all employ random number and incorporate a set of parameters that require to be adjusted initially. They show different performance depending on the problem under consideration and the predefined values of these parameters.

One of the recent additions to these novel optimization algorithms is the hunting search algorithm [7], which is inspired by group hunting of animals such as lions, wolves, and dolphins. Hunters involved in the hunting group encircle and catch their prey abiding by the certain strategies. For instance, wolves can hunt animals bigger or faster than themselves by relying on this kind of hunt. One prey is selected and the hunting group gradually moves toward it. The hunters avoid standing in the wind such that the prey senses their smell. This concept is used in the constrained problem to avoid prohibited regions. In optimization process, each of the hunters indicates one solution for a particular problem. Similar to animals cooperate to find and catch the prey, the optimum design process seeks to find the optimum solution.

2. MATHEMATICAL FORMULATION OF AN OPTIMIZATION PROBLEM

One of the most difficult parts encountered in practical engineering design optimizations is the constraint handling. Real-world limitations frequently introduce multiple, non-linear and non-

trivial constraints on a design. A general engineering optimization problem can be defined as follows;

Minimize $f(x)$, $x=\{x_1, x_2,\ldots,x_{Nd}\}$

which is subjected to $g_i(x)\leq 0$, $i=1,2,\ldots,p$ and $h_j(x)=0$, $j=1,2,\ldots,m$ where $Lx_k\leq x\leq Ux_k$, $k=1,2,\ldots,N_d$.

Here, $f(x)$ is the objective function, $\boldsymbol{x}$ denotes the decision solution vector, N_d is the number of decision variables, Lx_k and Ux_k, are the lower and the upper bound of each decision variable, respectively. p is the number of inequality constraints and m is the number of equality constraints.

3. HUNTING SEARCH OPTIMIZATION (HSO)

Hunting search algorithm is instigated by Oftadeh, et. al. [7]. This algorithm is inspired by group hunting of animals such as lions, wolves and dolphins. The common part in the way of hunting of these animals is that they all hunt in a group. They encircle the prey and gradually tighten the ring of siege until they catch the prey. Each member of the group corrects its position based on its own position and the position of other members during this action. If a prey escapes from the ring, hunters reorganize the group to siege the prey again. The hunting search algorithm is based on the way as wolves hunt. The steps of the algorithm are given in the following:

1. Initialize the parameters: The parameters of hunting search algorithm are required to be initialized. These are hunting group size (number of solution vectors in hunting group, HGS), maximum movement toward the leader (MML) and hunting group consideration rate (HGCR) which varies between 0 and 1. The parameters MML and HGCR are used to improvise the hunter position (solution vector).

2. Initialize the hunting group: Based on the number of hunters (HGS), the hunting group matrix is filled with feasible randomly generated solution vectors. The values of objective function are computed for each solution vector and the leader is defined depending on these values.

3. New hunters' positions (new solution vectors) $x'=\{x'_1, x'_2,\ldots\ldots,x'_n\}$ are generated by moving toward the leader (the hunter that has the best position in the group) as follows

$$x'_i = x_i + rand \times MML \times (x_i^{\ell} - x_i)$$

The MML is the maximum movement toward the leader, *rand* is a uniform random number [0,1] and x'_i is the position value of the leader for the i^{th} variable.

For each hunter, if the movement toward the leader is successful, the hunter stays in its new position. However, if the movement is not successful (its previous position is better than its new position) it comes back to the previous position. This provides two advantages. First, the hunter is not compared with the worst hunter in the group to allow the weak members to search for other solutions. They may find better solutions. Secondly, for prevention from rapid convergence of the group the hunter compares its current position with its previous position; therefore, good positions will not be eliminated. The value of MML varies depending on the problem under consideration. The range within 0.05 to 0.4 gives good results.

4. Position correction- cooperation between members: The cooperation among the hunters is required to be modelled in order to conduct the hunt more efficiently. After moving toward the leader, hunters (based on other hunter positions and some random factors) choose another position to find better solutions. Hunters correct their position either following "*real value correction*" or "*digital value correction*". In real value correction, the new hunter's position $x'=\{x'_1, x'_2,\ldots\ldots,x'_n\}$ is generated from HG, based on hunting group considerations or position corrections. For instance, the value of the first design variable for the j^{th} hunter $x_1^{j'}$ for the new vector can be selected as a real number from the specified $HG(x_i^1, x_i^2,\ldots\ldots,x_i^{HGS})$ or corrected using HGCR parameter (chosen between 0 and 1). The variable is updated as follows:

$$x_i^{j'} \leftarrow \begin{cases} x_i^{j'} \in \{x_i^1, x_i^2, \ldots, x_i^{HGS}\} & with\ probability\ HGCR \quad i=1,\ldots,n \\ x_i^{j'} = x_i^{j'} \pm Ra & with\ probability\ (1-HGCR) \quad j=1,.HGS \end{cases}$$

The parameter HGCR is the probability of choosing one value from the hunting group stored in the HG. It is reported that selecting values between 0.1 and 0.4 produces better results. *Ra* is an arbitrary distance radius for the continuous design variable. It can be fixed or reduced during optimization process. Several functions can be selected for reducing *Ra*. The following is used in [7].

$$\mathrm{R}a(it) = \mathrm{R}a_{\min}(x_i^{\max} - x_i^{\min}) \exp\left(\frac{\ell n\left(\frac{Ra_{max}}{Ra_{min}}\right) \times it}{itm}\right)$$

where *it* is the iteration number. $x_i^{\max}$ and $x_i^{\min}$ are the maximum and minimum possible values for x_i. Ra_{max} and Ra_{min} are the maximum and minimum of relative search radius of the hunter,

respectively, and *itm* is the maximum number of iterations in the optimization process.

In digital value correction, instead of using real values of each variable, the hunters communicate with each other by the digits of each solution variable. For example, the solution variable with the value of 23.4356 has six meaningful digits. For this solution variable, the hunter chooses a value for the first digit (i.e. 2) based on hunting group considerations or position correction. After the quality of the new hunter position is determined by evaluating the objective function, the hunter moves to this new position; otherwise it keeps its previous position.

5. Reorganizing the hunting group: In order to prevent being trapped in a local optimum they must reorganize themselves to get another opportunity to find the optimum point. The algorithm does this in two independent conditions. If the difference between the values of the objective function for the leader and the worst hunter in the group becomes smaller than a preset constant ε_1 and the termination criterion is not satisfied, then the algorithm reorganizes the hunting group for each hunter. Alternatively, after a certain number of searches the hunters reorganize themselves. The reorganization is carried out as follows: the leader keeps its position and the other hunters randomly choose their position in the design space

$$x_i' = x_i^{\ell} \pm rand \times (x_i^{\max} - x_i^{\min}) \times \alpha \times \exp(-\beta \times EN)$$

where x_i^{ℓ} is the position value of the leader for the i^{th} variable. *rand* is a uniform random number between [0,1]. $x_i^{\max}$ and $x_i^{\min}$ are the maximum and minimum possible values of variable x_i, respectively. EN counts the number of times that the group has been trapped until this step. As the algorithm goes on, the solution gradually converges to the optimum point. Parameters α and β are positive real values.

6. Termination: Steps 3-5 are repeated until maximum number of iterations is satisfied. The pseude code of the algorithm is given in Fig.1

4. DESIGN EXAMPLES

4.1. Welded Beam Design Problem

A rectangular cantilever beam made of low carbon steel is selected as first design example. The geometric view and the dimensions of the beam are illustrated in Fig. 2. The beam is designed to carry a certain P load acting at the free tip with minimum overall cost of fabrication. Design variables of the optimization problem can be listed as in the following.

$h = x_1$: the thickness of the weld

$l = x_2$: the length of the welded joint

$t = x_3$: the width of the beam

$b = x_4$: the thickness of the beam

```
begin;
    Initialize optimization problem and parameters:
        HGS: number of hunters- hunting group size
        MML: maximum movement toward leader
        HGCR: hunting group consideration rate
        EN: number of epochs
        Ra: distance radius
        α and β : reorganization parameters
    Initialize the hunting group- generate random population of HGS solutions (hunters);
    Calculate the fitness values of initial members: fitness(i) = f(x);
    Set leader as the best fitness of all hunters
    while (the termination conditions are not met)
        for each hunter i;
            Change the position- move toward leader in view of MML
            Calculate fitness(i)
            if fitness(i) is better than leader; leader = fitness(i)
        end for;
        for each hunter i;
            Correct position on the basis of group consideration (HGCR) and local search
            Calculate fitness(i)
            if fitness(i) is better than leader; leader = fitness(i)
        end for;
        for each hunter i;
            Update the position- reorganize the hunting group
            Calculate fitness(i)
            if fitness(i) is better than leader; leader = fitness(i)
        end for;
    end while
end procedure;
```

Fig. 1 Pseudo code for hunting search algorithm.

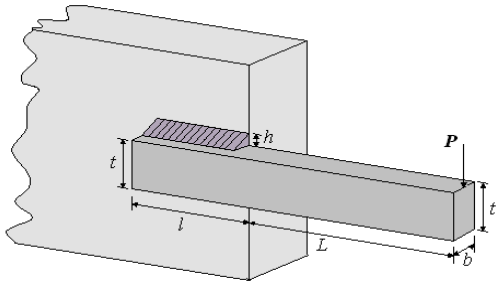

Fig. 2 Welded beam design.

The optimization problem can be stated as follows:

Minimize the cost function;

$$f(x) = 1.10471 x_1^{\ 2} x_2 + 0.04811 x_3 x_4 (14.0 + x_2)$$

Subject to:

$g_1(x) = \tau(x) - \tau_{\max} \le 0$: shear stress

$g_2(x) = \sigma(x) - \sigma_{\max} \le 0$: bending stress in the beam

$g_3(x) = x_1 - x_4 \le 0$: side constraint

$g_4(x) = 0.10471 x_1^{\ 2} + 0.04811 x_3 x_4 (14.0 + x_2) - 5 \le 0$: side constraint

$g_5(x) = 0.125 - x_1 \le 0$: side constraint

$g_6(x) = \delta(x) - \delta_{\max} \le 0$: end deflection of the beam

$g_7(x) = P - P_c(x) \le 0$: buckling load on the bar

Where

$$\tau(x) = \sqrt{(\tau')^2 + 2\,\tau'\tau''\,\frac{x_2}{2R} + (\tau'')^2}$$

$$\tau' = \frac{P}{\sqrt{2}\,x_1 x_2}$$

$$\tau'' = \frac{MR}{J} \quad , \quad M = P\left(L + \frac{x_2}{2}\right)$$

$$R = \sqrt{\frac{x_2^{\,2}}{4} + \left(\frac{x_1 + x_3}{2}\right)^2}$$

$$J = 2\left\{\sqrt{2}\; x_1 x_2 \left[\frac{x_2^{\,2}}{12} + \left(\frac{x_1 + x_3}{2}\right)^2\right]\right\}$$

$$\delta(x) = \frac{4PL^3}{E x_3^{\,3} x_4} \quad , \qquad \sigma(x) = \frac{6PL}{x_4 x_3^{\,2}}$$

$$P_c(x) = \frac{4.013\,E\sqrt{\frac{(x_3^2 x_4^6)}{36}}}{L^2}\left(1 - \frac{x_3}{2L}\sqrt{\frac{E}{4G}}\right)$$

$P = 6000\ lb, \quad L = 14\ in.,$

$E = 30\times10^6\ psi, \quad G = 12\times10^6\ psi$

$\tau_{max} = 13{,}600\ psi, \quad \sigma_{max} = 30{,}000\ psi,$

$\delta_{max} = 0.25\ in.$

The side constraints for the design variables are given as follows:

$0.1 \le x_1 \le 2.0, \quad 0.1 \le x_2 \le 10,$

$0.1 \le x_3 \le 10, \quad 0.1 \le x_4 \le 2.0$

The optimum designs obtained by the metaheuristic techniques are tabulated in Table 1. The optimum solution belongs the hunting search algorithm which is 1.724941. This is followed by particle swarm algorithm as 1.724966 and the harmony search algorithm found the third best value as 1.77192. The design histories of each algorithm are shown in Fig. 3. It is clear from Fig. 3 that hunting search has the best convergence rate and the best objective function value obtained is 1.724941.

Table 1. Optimum designs of welded beam problem obtained by three metaheuristic techniques.

	Meta- heuristic Search Techniques		
	Hunting Search	**Harmony Search**	**Particle Swarm**
f	1.724941	1.77192	1.724956
x_1	0.205731	0.18461	0.205730
x_2	3.47112	3.95998	3.471079
x_3	9.036624	9.10543	9.036796
x_4	0.205730	0.20626	0.205729

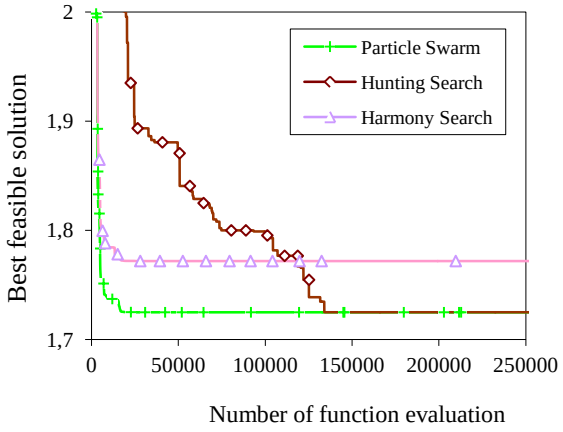

Fig. 3 Design histories of three algorithms for welded beam problem.

4.2 Spring Design Problem

Spring design problem aims to minimize the weight of tension/compression spring and is subjected to four constraints. These are minimum deflection, shear stress, surge frequency constraints and limits on outside diameter and design variables. The design problem shown in Fig. 4 has three design variables which are;

$d = x_1$: the wire diameter

$D = x_2$: the mean coil diameter

$N = x_3$: the number of active coils

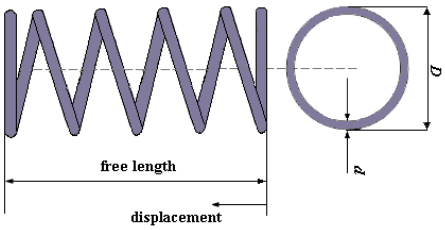

Fig. 4. Spring design.

The mathematical model of the problem can be expressed as follows;

Minimize; $f(x) = (x_3 + 2)\,x_2\,x_1^{\,2}$

Subjected to;

$$g_1(x) = 1 - \frac{x_2^{\,3} x_3}{71785 x_1^{\,4}} \le 0$$

$$g_2(x) = \frac{4x_2^{\,2} - x_1 x_2}{12566(x_2 x_1^{\,3} - x_1^{\,4})} + \frac{1}{5108 x_1^{\,2}} - 1 \le 0$$

$$g_3(x) = 1 - \frac{140.45 x_1}{x_2^{\,2} x_3} \le 0$$

$$g_4(x) = \frac{x_1 + x_2}{1.5} - 1 \le 0$$

And side constraints;

$$0.05 \le x_1 \le 2, \qquad 0.25 \le x_2 \le 1.3, \qquad 2 \le x_3 \le 15$$

The optimum solutions obtained by there metaheuristic algorithms considered in these review are tabulated in Table 2. It is clear from the table that except harmony search algorithm the remaining metaheuristic algorithms has obtained the optimum solutions that are very close to each other. The best solution for the spring design problem is determined by the hunting search algorithm which is equal to 0.012665.

Table 2. Optimum designs obtained by three metaheuristic techniques.

	Meta- heuristic Search Techniques		
	Hunting Search	Harmony Search	Particle Swarm
f	0.012665	0.012835	0.012666
x_1	0.051695	0.054665	0.051935
x_2	0.356869	0.432354	0.362666
x_3	11.27982	7.934532	10.94829

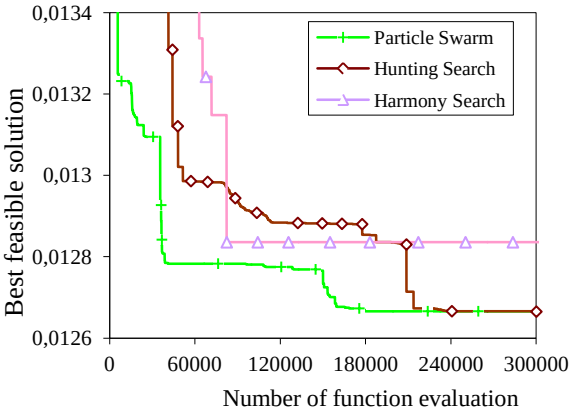

Fig. 5 Design histories of three algorithms for spring design problem.

5. CONCLUSION

The highly promising outcome of this research suggests that hunting search algorithm is an effective altenative for solving engineering optimization problems. In view of the results obtained, it can be concluded that hunting search algorithm can be extended to other real-world optimization problems in manufacturing and design area. Spring design problem and the welded beam design problem taken from the literature are solved by using three metaheuristic techniques namely harmony search, particle swarm and hunting search and their performance is evaluated and compared. It is noticed that hunting search algorithm shows great performance in finding the solution of design optimization problems.

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CARBON FIBER REINFORCED POLYMER SHEAR STRENGTHENING OF RC BEAMS SUBJECTED TO CYCLIC LOAD

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***Abstract*—An experimental investigation was conducted to study the effect of composite carbon fabric shear reinforcement on the ultimate strength and behaviour of reinforced concrete beam. Shear deficient specimens were strengthened by using side-bonded and wrapped CFRP straps. Eight beams were fabricated and tested under the cyclic loads. The main objective of the study is to obtain ductile flexural behaviour from the shear deficient RC beams. To verify the reliability of shear design equations and guidelines, experimental results were compared with all common guidelines and published design equations.**

Keywords: Shear strengthening, CFRP strip, Cyclic Load, Reinforced concrete beam, Anchorage.

1. INTRODUCTION

Many of the earlier studies about shear strengthening were concerned with the proof of effectiveness of CFRP as shear strengthening material. These studies indicated that using CFRP for shear strengthening is an effective and convenient method for improving member's strength and/or stiffness. In these studies, the behaviour of shear deficient beams strengthened with CFRP was also investigated [1].

The shear capacity of the strengthened RC beam depends on the compressive strength of concrete, the yield strength of shear and longitudinal reinforcement, the shear span to depth ratio and the area of composite fabrics [2, 3].

Bonding is critical for the effectiveness of CFRP strengthening. The effectiveness of strengthening techniques employing FRP relies on the adhesion between the FRP strap/sheet and the concrete surface of the element to be strengthened. One important aspect, peculiar to this technique, concerns the anchorage failure that occurs in a strap/sheet bonded to a concrete surface. Many studies, both theoretical and experimental, have been carried out on FRP-concrete adhesion [4]. For preventing debonding failure, anchorage details must be developed and applied at the end of the CFRP strips. But, limited number of studies is currently encountered about developed anchorage details [5, 6].

However, behaviour of the CFRP was not examined under cyclic load as monotonic loading was applied in all these studies. Experimental results were compared with the analytical approaches that were suggested by ACI-440 Committee Report [7], Concrete Society TR55 [8] and Fib Bulletin 14 [9].

2. EXPERIMENTAL PROGRAMME

Eight rectangular RC cantilever beams with different CFRP orientations were tested subjected to cyclic load. All specimens had the same geometry and materials as shown in Fig. 1. Longitudinal reinforcement consists of four 20 mm diameter steel rebars in the bottom and top of the beam section. Beams had no internal shear reinforcing rebars in the shear span. Stirrups were, however, utilized outside the shear span to block local cracks that might occur under the applied load. A single concrete mix was used for all specimens to achieve a similarity in strength. The concrete was a mixture of water, cement, sand and aggregate with the ratio of 0.68:1:2:3 by mass. The measured average cylinder compressive strengths of concrete cylinder were approximately 25 MPa. The yield stress of the longitudinal reinforcing steel was 414 MPa.

Beam-1 was the reference of shear deficient beams, tested without strengthening, and used as a baseline comparison to evaluate the enhancement in strength provided by the strengthening material. The remaining shear deficient RC beams were strengthened with CFRP strips.

Beam-2 was strengthened with 50 mm wide CFRP strips spaced at 80 mm. Strips were applied to the both sides of the beam symmetrically. Beam-

3 was strengthened with 50 mm wide CFRP strips spaced at 120 mm. The beam-4 strengthening scheme was identical to beam-3. But the top and bottom ends of each of the strips were anchoraged to the beam with two anchorages. Beam-5 was strengthened with 50 mm wide strips spaced at 160 mm and anchorages were applied to both the top and bottom ends of the beam web. Strengthening scheme of beam-6 was identical to beam-5 but without any anchorage applied for strengthening material. Beam-7 was strengthened with 100 mm wide CFRP strips spaced at 160 mm. Beam-8 was strengthened with 50 mm wide CFRP strips spaced at 120 mm. CFRP was wrapped around the entire cross-section. CFRP strips were applied to both sides for all except beam-8.

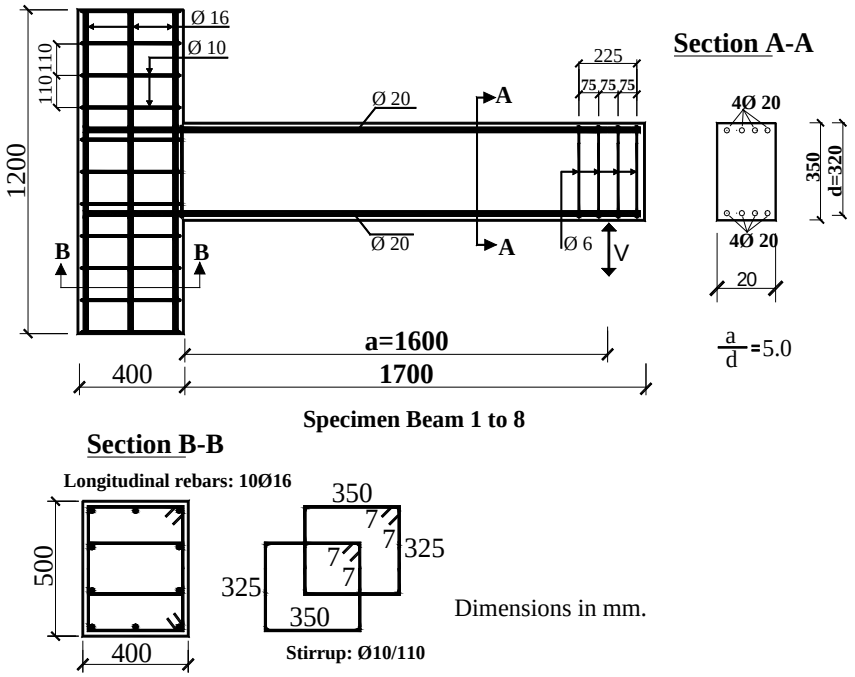

Fig. 1 Reinforcement details of specimen beams.

Ancorage Details of Specimens

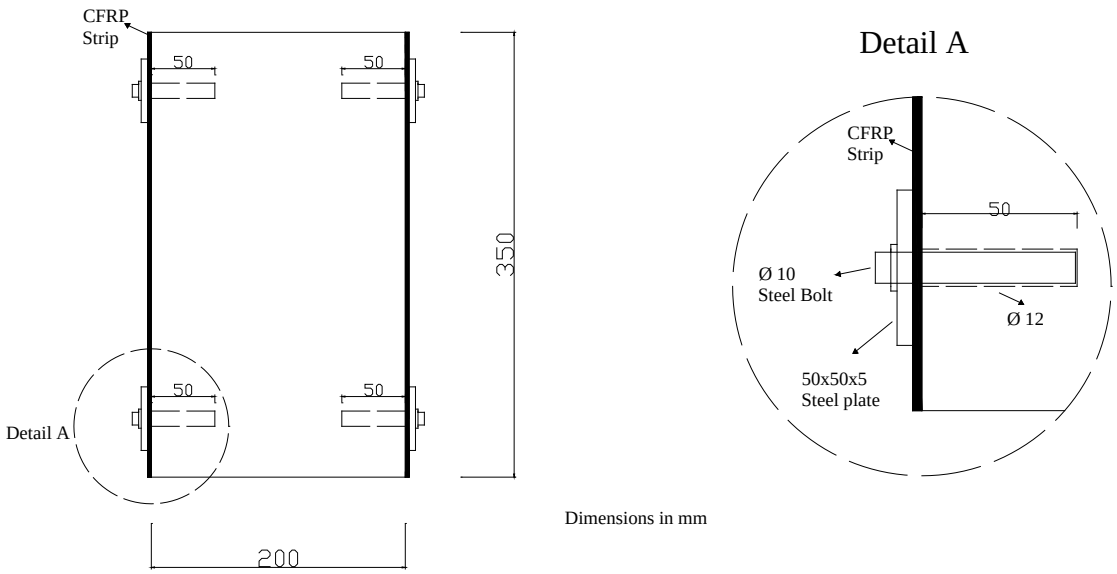

Fig. 2 Anchorage details for specimen beam.

Different mechanical anchorage details were developed in the study. Anchorages were included as 50x50x5 mm square steel plates and 10 mm diameter threaded rods with the anchorage details presented in Fig. 2. The location at which threaded rods were to be placed was marked for the anchorage application. The bottom and top marks at the beams' web were drilled with 12 mm diameter drills up to 50 mm depth. All the drilled holes were cleaned with pressured air.

Threaded rods of 10 mm diameter were placed at the bottom and top drilled holes by taking the bonding procedure of CFRP into account. While placing threaded rods, extra care was taken not to damage the continuity of fibre directions. After curing at room temperature, developed anchorage details were performed on the beam specimens.

Before applying to composite, some pre-treatment steps for the beams were necessary. First, the beams were sand-blasted to remove outer weak surface of the concrete and then the surfaces were vacuum cleaned to remove loose particles and dust. Epoxy primer was coated all throughout the beam according to the manufacturer's suggestions. CFRP strips were then placed on the coated epoxy primer and constant pressure was applied on the sheet surface by a roller. Another layer of epoxy was then put on top of the fabric and the extreme resin was cleaned. Specimens were cured for a minimum of 15 days under laboratory conditions. The properties of CFRP and the epoxy resin are presented in Table 1.

Table 1. Properties of CFRP and Resin.

Properties of CFRP	**Remarks of SikaWrap 230C**
Fiber Orientation	0^0 (unidirectional)
Construction	99% Wrap, 1% weft
Thickness (mm)	0.12
Tensile Strength (MPa)	4 100
Elastic Modulus (MPa)	231 000
Ultimate Tensile Strain (%)	1.7 %
Properties of Resin	Remarks of Resin
Tensile Strength (MPa)	30
Elastic Modulus (MPa)	3800

3. TEST SETUP

All cantilever specimens were tested under cyclic load. To perform cyclic load to the specimen, a loading column was designed with hinges by the beam's free end. The loading column contained two hinges, a load cell and a hydraulic jack. The capacity of the hydraulic jack was 500 kN while the load cell's capacity was 400 kN. Load was applied in cycles of loading and unloading. The same loading cycles were applied to all specimens. Loading was increased up to the yield of the flexural reinforcements or until the failure of the specimen. For each load increment, displacements, loads and strains were recorded simultaneously by means of an automatic data acquisition system. Four linear variable differential transformers (LVDTs) were used to monitor displacements. The LVTDs are located at the end of the beam for maximum displacement, under the rigid support to calculate the undesired displacement and finally on the rigid support to calculate the rotation.

Strain gauges help to determine the shear cracks before propagation with the help of increase in strains. Strain gauges were attached to CFRP strap along the fiber direction and were attached to straps that were situated at distances between 80 to 1600 mm apart from the rigid support. Eight strain gauges were used for each specimen.

4. TEST RESULTS

All strengthening types significantly increased the shear strength of the beams. The first crack always appeared as a flexural crack at the maximum bending moment region for all beams. Shear cracks followed the flexure cracks at the unstrengthened part of the specimens i.e. between CFRP strips. Later, shear cracks developed towards the strips due to the increasing load. Because of the different strengthening scheme, first flexure crack propagation was observed between 34 kN and 40 kN load level for all specimens.

5. EVALUATION OF TEST RESULTS

The test results pointed out that CFRP is an efficient material to increase the shear capacity of RC beams. Table 2 summarizes of the results of specimens during test.

Comparing to the reference beam, a minimum increase of 20% and a maximum of 75% in shear capacity was acquired from the strengthened specimens. Beam-2 showed 75% more strength than beam-1. Beam-3 showed 23% more strength than the reference beam. By decreasing the amount of the strengthened area (increasing the spacing of CFRP strips from 80 to 120), a decrease of 42% at contribution to the shear resistance was obtained according to beam-2. The beam-4 was strengthened in a manner similar to that of beam-3. The ends of the strips in beam-4 were anchoraged. Apart from beam-3, beam-4 had reached 100 kN loading cycle but failed in 83 kN load. The specimen showed 9% more strength than beam-3. Anchorages did not provide any substantial increase in shear strength but they obtained remarkable increment in the displacement capacity of beam-4.

Table 2. Experimental Results.

Specimen		W_f (mm)	S_f (mm)	Anchorage	Arrangements	Failure Load (kN)	Maximum load at the forward cycle	Maximum load at the backward cycle	Failure Mode at Ultimate
Beam-1	Control	------	------	------	------	61.7	62.4	-63.1	Shear
Beam-2	Strengthening	50	80	------	Side bonding	108.0	108.0	-99.2	Shear
Beam-3	Strengthening	50	120	------	Side bonding	-76.1	92.5	-89.0	Shear
Beam-4	Strengthening	50	120	Yes	Side bonding	-83.0	99.6	-89.7	Shear
Beam-5	Strengthening	50	160	Yes	Side bonding	-74.0	89.5	-79.8	Shear
Beam-6	Strengthening	50	160	------	Side bonding	84.0	84.0	-80.1	Shear
Beam-7	Strengthening	100	160	------	Side bonding	-85	89.5	-85	Shear
Beam-8	Strengthening	50	120	------	Wrapped	102.0	102.0	-100.6	Flexure

*Forward loading step was described with positive mark and backward loading step was described with negative mark in Table

Table 3. Comparison of Experimental and Analytical Results.

Specimen	Experimental Strengths	Calculated Strengths for ACI-440	Calculated Strengths for Concrete Society, TR55	Calculated Strengths for Fib Bulletin 14	Experimental/calculated		
	$V_{exp.}$ (kN)	$V_{cal.}$ (kN)	$V_{cal.}$ (kN)	$V_{cal.}$ (kN)	$V_{exp.}/V_{cal.}$ for ACI-440	$V_{exp.}/V_{cal.}$ for TR55	$V_{exp.}/V_{cal.}$ for Fib
Beam-2	108	66.15	114.15	77.29	1.63	0.95	1.40
Beam-3	76	60.32	93.89	73.81	1.26	0.81	1.03
Beam-4	83	61.04	95.10	74.53	1.36	0.87	1.11
Beam-5	74	53.21	76.83	67.19	1.39	0.96	1.10
Beam-6	84	52.16	75.21	66.08	1.61	1.12	1.27
Beam-7	85	68.60	80.87	79.53	1.24	1.05	1.07
Beam-8	102	62.84	128.41	84.59	1.62	0.79	1.21

Beam-5 had more strip space than aforementioned strengthened specimens. Hence, the least strength improvement was observed at that beam among the strengthened specimens. It was showed 20% more strength than the control beam. Beam-5 was showed 14% less strength than beam-6 that was strengthened with same CFRP arrangement without anchorages. This result might be arising from abrupt debonding failure of the specimen. Beam-6 showed 36% more strength than beam-1. It was minimum displacement capacity beam of the strengthened specimens. Beam-7 was nearly the same shear capacity as beam-6. The specimen had double strip width as to beam-6 but that application did not improve the shear strength.

As a result of the use of wrapping, a significant increase in the shear capacity was achieved in beam-8. Furthermore, the failure mode at ultimate changed from CFRP debonding to flexural failure mode.

5.1 Comparison of Test Results and Design Equations

Results from both analytical and experimental study are shown in Table 3. The analytical shear force capacities of specimens were calculated according to guidelines. For all these methods, shear strength of a strengthened RC section could be expressed as the sum of the three components.

$$Vn = Vc + Vs + Vf \quad (1)$$

Where, Vc is the contribution of concrete, Vs is the contribution of internal steel shear reinforcement and finally Vf is the contribution of CFRP in equation (1).

Concrete and/or steel stirrups can not utilize their ultimate strengths when CFRP is used as a strengthening material, as demonstrated in equation (1). Common design codes, however, still use the relative contribution of these materials to compute the total shear strength. It must be considered that these parameters interact with each other. For this reason, analytical and experimental results must be evaluated according to the aforementioned considerations.

Differences were determined between analytical shear load capacities that were suggested by ACI-440 committee report, The Concrete Society TR55, Fib Bulletin 14 and experimental results. The reasons for obtaining different results in analytical calculation and experiments from the expected were that the guidelines did not consider a/d ratio and the effects of anchorage.

The analytical shear force capacities of beams were calculated according to ACI-440 committee report, Concrete Society TR55 and Fib Bulletin 14 suggestions. Maximum divergences between analytical and experimental shear load capacities were calculated from ACI-440 results. Moreover, the Concrete Society TR55 calculated closer results for the specimens. The closest results of the experimental programme were obtained for beam-7. The shear capacity of beam-7 was 24% less than that of the experimental result for ACI-440 and 5% and 7% less than The Concrete Society TR55 and Fib Bulletin 14, respectively. The experimental and analytical results were not too close. Thus, the impact of the ratio a/d, the anchorage effect and the interaction of internal shear reinforcement to shear capacity due to CFRP were not included in the guideline. The proposed design results would be more realistic if the unconsidered parameters were measured while improving the preceding equations.

6. CONCLUSIONS

Strengthening of reinforced concrete beams in shear using CFRP straps appears to be a highly efficient technique. Developed anchorage detail worked efficiently under the cyclic load. Anchorages prevented the peeling of the CFRP from concrete. However, bottom anchorages were efficient as they prevent the premature peeling of the CFRP.

All CFRP applications improved the strength and behaviour of the specimens in a different way. The strength was increased between 1.20 and 1.75 times compared to reference beam. Besides the displacement capacity of the strengthened beams was improved meaningfully. In addition, Beam-8 that was strengthened with the wrapped CFRP strips behaved like a ductile flexural beam.

Significant differences were determined between the analytical shear force capacities that were suggested by ACI-440 committee report, The Concrete Society TR55, Fib Bulletin 14 and experimental results. The reasons for obtaining different results in analytical calculation and experiments from the expected were that the guidelines did not consider a/d ratio and the effects of anchorage.

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STRUCTURAL PRECAST CONCRETE IN TURKEY: DEVELOPMENT AND EARTHQUAKE PERFORMANCE IN LAST DECADES

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***Abstract*—In recent decades, the precast concrete structures become increasingly popular in Turkey. Besides its cost efficient and time-saving nature, sufficient earthquake resistance should be provided at the design stage for proper and safe utilization of those structures. For this purpose, prime importance should be given to joint locations such as beam-column connections where considerable amount of energy dissipation is required to ensure earthquake resistance. In this paper, the development of the structural precast concrete in Turkey over the last decades is discussed with special emphasis on the research and developments in the area of beam-to-column connections. In addition, the performance of the existing precast concrete structures in recent earthquakes is also reported in order to highlight the significance of know-how transfer to avoid the repetitive errors both at design and construction stages.**

Keywords: Precast concrete structure, Beam-column connection, Seismic performance.

1. INTRODUCTION

Structural precast concrete technology provides various advantages compared to the conventional cast in- place construction method. These well-known advantages may be listed as the minimized construction duration, higher compatibility with the architectural details, lower labor and material cost due to mass production process in a factory or a manufacturing site, reduced risks caused by the local climatic conditions by spending less time in construction site, easier and reliable quality-control process at prefabrication stage compared to in-situ construction [1]. In this technology, the structural elements of load-bearing system such as columns, beams and slabs were first produced and stocked up in a factory plant. Afterwards, the prefabricated structural elements were transported to construction site and assembled. The most critical requirement of this construction method is the specially automated system of industrial plant, since the design and production stages have to be well-organized together in an efficient manner.

Precast concrete structures were classified into two main groups according to their load bearing systems named as frame and panel systems respectively. Frame systems generally consist of beams, columns and floor elements whereas panel systems composed of wall and floor panels. The systems constructed using both frame and panel systems were called as composite systems [2].

The structural performance of precast concrete systems generally depends on the connection types between the precast concrete members [3]. Pin connected precast concrete systems were mostly preferred for industrial one-story structures with large girder span lengths. The well-known drawback of such systems is the non-moment resisting behavior that leads to unrepairable damages under lateral loading such as earthquake. On the other hand, moment resisting frame connection types may be divided into two groups namely dry and wet connections [4].

In the early years of application, precast concrete structures were generally used in low seismicity regions. By time, these type of structures spread through the seismically active regions depending on the growing interest. Accordingly, the

rigidity, strength, ductility and energy dissipation capacities of the structures become more critical in the design of such structures. For this purpose, the studies in the literature generally focused on obtaining a behavior similar to monolithic frame systems without sacrificing strength, ductility and energy dissipation capacities at the connection regions [5].

In this study a detailed review about the development and improvement of precast concrete systems in Turkey was presented and the performance of these systems under recent earthquakes is discussed.

2. PRECAST CONCRETE STRUCTURES IN TURKEY

Manufacture of State Fez building is known as the first structure that was built by precast technology in Turkey. The structure was built in 1868 and having 8.000 m^2 closed area. The load-bearing system of this building was constructed with shed roof sitting on thin walled cast iron cylindrical columns. Bulgarian Orthodox Church between Balat and Fener districts in Istanbul was reported as the first full prefabricated structure of the Ottoman period. Those buildings were built by steel precast structural systems due to poor soil conditions. The structural members were connected with bolts, nuts, rivets and welding.

In 1955, first prefabricated concrete structural elements, (centrifugal concrete poles used for street lightning purposes) were manufactured in order to balance the overproduction of the cement plants in Turkey. This may be accepted as the pioneering investment in concrete prefabrication industry in Turkey. Later on, first precast concrete structural constructions were built as one-story school buildings in the first half of 1960's. These buildings were constructed by using semi-precast systems. First, the surrounding walls were constructed. Afterwards, 7.20m wide span was covered by truss roof system that consists of iron rods and concrete top chord. The prefabricated roof panels were placed on the roof truss. A total of 234 school buildings were constructed in same manner in three months time and opened to public service (Fig. 1).
Meanwhile, the residential houses of Ereğli Iron Steel Factory were constructed by a well known precast concrete panel system called Larsen-Nielsen System that was developed in Denmark in 1948. The structural components of this system were wall and slab panels. The primary objective of Larsen-Nielsen System is to minimize in-situ construction process. The wall and floor panels were first assembled by bolt connections and then those connection regions were filled with mortar to ensure continuity which is a good example of wet type of connections. Total of 446 apartment were constructed by using mentioned method in one year time. It is possible to register the year 1961 as the birth of precast concrete were using in public buildings and residence (Fig. 1).

Fig. 1 School buildings and the residency of Ereğli Iron Steel Factory [6].

The utilization of precast concrete systems in construction of industrial buildings has been started in the second half of 1960s. The industrialization thrust in those years triggered the heavy and urgent demand for rapidly constructed industrial structures for storing and sheltering the machinery safely. The rapid construction of those buildings has prime importance due to loan cost of the machinery.
In 1968, The Ministry of Industry decided to establish small industrial sites at different districts to promote industrial development. In the late 1970s, Tekel tobacco warehouse was built by fully prefabricated system from foundation to roof. A total of 32 dormitories were built with prefabrication between the years of 1984 and 1989. These buildings were constructed with either panel

system consisting of hollow core slabs and wall panels or frame system consisting of prefabricated column, beam and double tee flooring unit. The popularity of precast concrete systems in construction of residential buildings increased significantly in 1980s. Bingöl earthquake houses and Kocaeli immigrant houses were the significant examples of residential buildings constructed by precast concrete systems in 1980s (Fig. 2). In 1990s, technological improvements in conventional methods such as widespread production and use of ready mixed concrete with increasing number of ready mix concrete plants and achievement of tunnel formwork concrete construction suddenly paused the growing interest in prefabrication especially for residential buildings [7].

Turkish Precast Concrete Association was founded in 1984 with the collaboration and supports of manufacturer companies. Beyond its representative role in order to promote the use of precast concrete,

Fig. 2 Bingöl earthquake houses [6].

Turkish Precast Concrete Association also put emphasis on the research and development activities, technical publications and regulations, etc. TS 9967 (1992) regulation including analysis principles and manufacturing and assembly rules of reinforced concrete prefabricated structural elements and structural systems was published by Turkish Standards Institute with the contributions of Turkish Precast Concrete Association [8].

The restrictive and router rules in design of precast structures were explained in Turkish Earthquake Code that activated in 1998 (TEC, 1998). The guidelines about precast concrete structures were then modified in Turkish Earthquake Code 2007 (TEC, 2007) according to performed research and development by different institutes.

3. RESEARCHES ON PRECAST CONCRETE SYSTEMS IN TURKEY

Precast concrete systems are most preferred construction method for industrial buildings, 85 percentages, in Turkey. It is vise versa in the case of residential structures, only 4 percentages [9]. Typical industrial structures are single-story and consist of simple portal frames of which connections fixed at the bottom side and hinged at the top side. Such precast framing systems which have hinged beam-to-column connections could be used in multi-story structures in case of that seismic load would be carried by cast in place reinforced concrete walls. Moreover, precast concrete structures with moment resisting connections are applicable for earthquake prone regions. It should be proven that such connections could provide equivalent strength and ductility as monolithic connections under reversed cyclic loading [10]. Basically, there are two methods to build moment resisting connections for precast concrete systems namely dry connection and wet connection. Outcomes of the experimental studies in Turkey on moment resisting connections for precast concrete beam-to-column joints are summarized in this section.

3.1 Dry Connections

In application of typical dry connections, steel rods and plates are embedded at both ends of precast beams during casting in order to connect those beams to columns by bolts or welding in construction site. Most of the time welding was preferred compared to bolted connections. Another type of dry connections was implementation of post tensioned steel for assembling purposes [4].

3.1.1 Welded connection

The first comprehensive research program that focused on the moment-resisting connection of precast structures in Turkey was performed in the Structural Mechanics Laboratory of METU. Precast beam-column connections, designed in multi-storey buildings located in seismic area, were modified to resist for earthquake loading [3]. Later on, the dry beam-column connections located away from column face were tested by Ersoy and Tankut. The connection between the column bracket and the precast beam consisted of steel plates, those were welded to plates embedded in the members on both top and bottom faces. According to test results, a modification was proposed to obtain a behavior similar to monolithic connections. In this modification, double steel plates were used on both top and bottom faces for moment transfer and additional double plates were used on both sides to provide shear transfer. The test results indicated remarkable increases in terms of strength, rigidity and energy dissipation capacity of the specimens [11].

Korkmaz and Tankut investigated the behavior of moment resisting precast concrete beam-to-beam connections under reversed cyclic loading. The connections consist of a middle

precast beam that is placed on a cantilever beam connected to the column. Within the connection region, the continuity of the bottom reinforcement was provided by welding them to the steel plate at the junction of middle and cantilever beams. On the other hand, continuity of top reinforcement was satisfied by lap-splicing and in-situ concrete casting. However, the performance of those specimens was questionable. Therefore, the insufficient anchorage due to lap-splicing on top was eliminated by welding the reinforcing bars. The test results indicated that the quality of workmanship in welding the bars has prime importance to avoid premature failure in the vicinity of welded region [12].

3.1.2 Bolted connection

Ertas et al. [13] developed a special bolted connection detail to minimize the duration of assembling in site. The connection detail consisted of rectangular steel boxes allowing dimensional advantages during construction. This detail was proposed especially for short span and low level of shear forces formed by vertical loads. In addition, steel plates were placed at the top and bottom of the beam section to delay crushing of the concrete adjacent to the column surface. Steel bars were welded around the steel boxes and rods passing through the box section to eliminate any possible sliding of the steel boxes with respect to concrete beam. Bolts were pre-tensioned during assembly of connection and specimens were tested under reversed cyclic loading. The test results revealed that the performance of bolted connection is better than monolithic connection in terms of strength, ductility and energy dissipation capacity.

3.1.3 Post-tensioned connection

Post-tensioned connection detail was first developed and tested by Pınarbaşı in Turkey. The effect of post tensioning steel ratio on behavior of precast connections was investigated. The test results indicated significant increase in strength, ductility, rigidity and energy dissipation capacity by application of post-tensioning[14]. Another post-tensioned connection detail was tested by Kaya and Arslan. The effect of diameter of post-tension strands was investigated in their experimental study [15].

Ozden and Ertas also proposed a special hybrid connection to improve the moment resistance of precast beam-column connections. The main objective of the study was to investigate the effect of mild steel reinforcement ratio on behavior of post-tensioned precast concrete connections. The performance of hybrid connections improved significantly by increase of mild steel reinforcement ratio such that the capacity of companion monolithic subassembly was almost reached in terms of strength, stiffness and energy dissipation [16].

3.2 Cast in Place Connections

The most common moment resisting connection detail using in precast concrete structures is cast in place connection in other words wet connection. The well-known type of wet connection is composite connection of which reinforcement continuity provided by welding or bolts. In general, tension due to positive moment is transferred through the welding or bolts while tension due to the negative moment is transferred through reinforcing steel bars longitudinally placed in cast in place concrete. It is revealed that adequate performance parameters such as strength, ductility and energy dissipation capacity can be provided with the composite connections. One of the well-known study was conducted by Ersoy and Tankut. The flexural reinforcement at the bottom of the connection was supplied by welding of steel plates embedded into joining portions of the beam and the corbel on the column. Besides the positive findings obtained from the tests, it was revealed that the observed damage accumulates in the connection region even though the connection is designed to be stronger than the joining members [17].

The composite connection is the most preferred cast in place connection type used in Turkey as basically described above. The composite connections was also examined in the study done by Ertas et al.. It was concluded that the viability of the connection in seismic areas were provided in terms of strength and energy dissipation capacity while ductility of connection was less than that of the monolithic one. It was thought that the low ductility level of the composite connection was caused by the adverse and overturning effect of the welding process on material characteristics [13].

In addition to composite connection, there are several types of cast in place connections. It is known that the connection region could be arranged in different locations in beam-to-column joint even outside of the joint zone. Two different cast in place connection techniques were summarized and tested in the research which was carried on by Ertas et al.. There is not any cast in place topic concrete along the upper side of beams in those wet connection types namely cast in place in column and cast in place in beam. Only concrete casting process is limited in either joint zone in column for "cast in place in column connections" or joining end of the beam for "cast in place in beam connections". The precast concrete beams protruding U shaped reinforcing bars which serves as both positive and negative flexural reinforcement at the connection region considering the anchorage issue are used in the either techniques. There is a gap of which height is precisely equal to beam depth at mid-height of precast concrete column so that precast

concrete beam can be easily seated and aligned in the assembling process in the cast in place in column connections. On the other hand in the cast in place in beam connections, there is no gap inside the column and U shaped reinforcing bars protruding from the column like joining beams are placed at the mid-height of the precast columns. Single leg ties are placed into joining zone and steel fiber reinforced concrete is used for filling the empty space in connection region. It is recommended that both precast concrete connections are suitable for high seismic areas in terms of strength properties and energy dissipation [13].

4. EARTHQUAKE DAMAGE IN PRECAST CONCRETE STRUCTURES

Severe and catastrophic earthquakes occurred frequently in Turkey as well as in the world in last decades. 92% of Turkish territory is located in a seismically active region which means that 95% of the total population and 98% of industrial facilities are placed in disaster prone and hazardous regions. Therefore excessive damage was observed in earthquakes occurred in Turkey and such events result in lots of fatalities and breakdown in industrial production [18]. The precast concrete structures exist in Turkey can be classified in three different categories which are panel systems, pin connected frame systems, moment resisting frame systems. The concrete precast structures exposed to Ceyhan Earthquake in 1998 for the first time in Turkey. 1999 Marmara Earthquakes and 2011 Van Earthquake caused to different level of damage on the precast structures as well.

Kocaeli immigrant houses and dormitory building have been built as concrete precast panel system which consists of slab panels and wall panels. There was not any significant damage in these buildings after Marmara Earthquakes in 1999. It was concluded that
the buildings were fully operational and occupied after the earthquakes (Fig. 3).

Fig. 3 Kocaeli immigrant houses.

Majority of the single-story precast industrial buildings are constructed with pin connections at beam-to-column joints. Severely damage was observed in these kinds of buildings in Marmara Earthquakes in 1999 [19, 20]. According to the field studies after the earthquakes, it was concluded that the main reason of the encountered excessive damage was inadequate and improper pin connection production in assembly phases of the prefabricated structural members. Encountered pin connection failures were caused by lack of transverse reinforcement around the pin holes, low strength and poor quality mortar usage for filling the holes and no washer, nut or welding detail consideration at the extruded ends of pins. It should be mentioned that overmuch lateral tip displacements of the columns in such structures due to improper cross-sectional design and lack of lateral rigidity also resulted in overloading and poor behavior at connections. Similar damage pattern was observed in only three pin-connected precast industrial structures which were under construction during the Van Earthquake in 2012. However, the level of damage was minimum in most of the precast industrial buildings. It is considered that the existence of external walls reduce the inter-storey drift, while the metallic roof cover results in a sort of diaphragm action (Fig. 4).

There were several examples of multi-story precast structures which have moment resisting connections subjected to recent earthquakes in Turkey. Dormitory building in Bolu could be given as an example of this kind of precast structures (Fig. 5).

Fig. 4 Undamaged Industrial building in the Van earthquake.

Fig. 5 Dormitory building in Bolu.

Moment resisting connections provided by welding steel plates which already placed at the contact points of the prefabricated beam and column at the bottom of the connection and additional longitudinal reinforcement passing through the connection region placed into cast in place topic concrete at the top of the connection in general. There was not any collapsed precast building which has moment resisting connections in Marmara Earthquakes in 1999. However, slight to moderate damages i.e. cracking in walls, plastification at end of structural members were observed such structures during the site surveys (Fig. 5).

Another way to build a moment resisting connection for precast systems is introducing the post-tensioning technique to the beam-to-column joints with special constructional details. During the site survey in Van Earthquake in 2012, such a multi-story concrete precast building with post-tensioned moment resisting connections examined closely. There were not any damage in precast structural members, the post tensioned connections and post tensioning ducts. In addition, flexural or shear cracks did not occur in column to foundation connections. On the other hand, shear cracks took place on some of the inner and outer walls due to incompatible displacement characteristics of the structural system and the walls (Fig. 6).

Fig. 6 Residential building in Van.

5. CONCLUSION

Prefabrication which is simply assembling of the precast concrete members in site and quick way to construct a concrete structure is a well known construction technology all over the world. The application area of the concrete precast systems got wider in Turkey last decades. Research issues and development are still in progress for the earthquake performance of precast systems. It is obvious that the performance of the connections dedicates the performance of the prefabricated structure since the structural members are produced individually in the factory under quality assurance. Several research projects were done in Turkey on performance and deficiencies of precast connection resulted in detailed and reliable connection design criteria.

Precast concrete systems are generally used for construction of industrial type of structures especially in Turkey. Most of these structures are built as single story with 6-9 meter height and composed non-moment resisting connections which constructed with pin connections which transfer only shear and axial loads among the structural elements. On the other hand, few examples of concrete precast multi-story structures which have moment resisting connections are in service for residential purposes. It is observed that these buildings were achieved quit good performance in the past earthquakes in Turkey. Even though, earthquake resistant and easily applicable moment resisting connection details should be investigate to build fast and reliable residential multi-story buildings in earthquake prone areas in Turkey. In fact these kinds of studies on concrete precast systems are in progress almost all over the world, the comprehensive multi-partner research project of which actors should be government, universities and construction industry must be conducted for development of more reliable moment resisting connection details for earthquake resistant precast structures. It should be a lead to advance of both national and international knowledge about the precast systems but also result in widespread usage of the faster and safer construction method in earthquake prone regions.

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THE HOMOTOPY PERTURBATION METHOD FOR COMPLEX LINEAR SCHRÖDINGER WAVE EQUATIONS

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***Abstract*—In this article, the analysis of the complex partial wave equations is presented by using the Homotopy Perturbation Method (HPM) and least square method. For this investigation, firstly complex linear Schrödinger equation and then complex nonlinear Hirota equation is studied.**

Keywords: Dynmacis of structures, Homotopy perturbation method (HPM).

1. INTRODUCTION

The analytical and/or numerical solutions of the linear and nonlinear partial differential equations (PDE) are required in the many branches of the science and engineering fields. Main purpose is to obtain first an analytical solution. If it is not possible, many researcher tries to find an accurate numerical solution. In the last two decades, researches progresses with this direction since analytical solutions to NPDEs are limited or sometimes unavailable. Complex varied linear and nonlinear PDEs are also interest of these fields. In the past, the traveling wave solutions of linear and nonlinear PDEs such as Schrödinger [1] and Hirota equation [2,3] is obtained by Fan [4] for Hirota equation. The mathematical model equation for the Schrödinger and Hirota equation is given in the following form, respectively.

$$iU_t + U_{xx} = 0\,, \tag{1a}$$

$$iU_t + U_{xx} + 2|U|^2 U + i\alpha U_{xxx} + 6i\alpha |U|^2 U_x = 0 \tag{1b}$$

The Homotopy Perturbation Method (HPM) is proposed by He [5] and further improved by He in [6-10] for better results. It provides a fast convergence for the studied problems. It shows a good accuracy and a fast convergence to the solutions of the linear and nonlinear partial differential equations.

In this article, the HPM is applied for an accurate numerical solution of Schrödinger and Hirota equations. The efficiency and convergence of the numerical results are compared with exact ones.

2. HOMOTOPY PERTURBATION METHOD

One considers the following nonlinear partial differential equation to represent the algorithm of the HPM,

$$Au - f(r) = 0\,, \quad r \in \Omega \tag{2}$$

with the boundary conditions of

$$B(u, \frac{\partial u}{\partial n}) = 0\,,\ r \in \Gamma \tag{3}$$

where *A* and *B* are general differential operator and boundary operator, respectively. Γ is the boundary of the domain Ω, and $f(r)$ is a given and known analytical function.

After dividing the general operator into linear part (*L*) and nonlinear part (*N*), one can write the equation (2) as

$$Lu + Nu - f(r) = 0 \tag{4}$$

By constructing the homotopy procedure to equation (4), one can rewrite a homotopy in the form

$$H(V, p) = (1-p)[L(V) - L(u_0)] + p[A(V) - f(r)] = 0$$
$$p \in [0,1]\,,\ r \in \Omega \tag{5}$$

where $p \in [0,1]$ is an embedding parameter, u_0 is an initial approximation of the equation (2) which satisfies the equation (3).

In the HPM, one use the embedding parameter as a small parameter. Thus, the solution of equation (5) can be given a power series of *p* in the form,

$$V = V_o + pV_1 + p^2V_2 + \ldots \tag{6}$$

By giving $p=1$, one can get an approximate solution of the equation (2) as,

$$u=\lim_{p\to 1} v = V_o + V_1 + V_2 + ... \qquad (7)$$

The combination of a small parameter with a homotopy is called homotopy perturbation method.

3. EXAMINATION OF COMPLEX LINEAR SCHRÖDINGER EQUATION WITH HPM

To test the accuracy of the proposed method, a comparison will be done by using two-dimensional finite elements having 4 nodes rectangular elements.

In this part, the numerical solution of the Schrödinger equation [1] is examined by using the HPM. By selecting $U=u+iv$, one can separate equation (1a) into real and imaginary parts. Thus, one gets (1+1) dimensional coupled system in the following

$$v_t - u_{xx} = 0$$

$$u_t + v_{xx} = 0 \qquad (8)$$

The exact solutions to equation (8) are given [1] by

$$u = c_1 Cos(c_2 x - c_2^2 t)$$

$$v = c_1 Sin(c_2 x - c_2^2 t) \qquad (9)$$

$u_x(0,t)=0$, $u_t(1,t)=0$,

$u_t(x,0)=0$, $u_t(x,1)=0$, $v(0,t)=0$,

$v_t(1,t)=0$, $v_t(x,0)=0$, $v_t(x,1)=0$

for $0\le x\le 1$, $0\le t\le 1$.

where c_1, c_2 are any arbitrary constants. For the simplicity, $c_1 = c_2 = 1$ are used in the analyses.

Let us assume an initial trial function as $u_o = u_0(x,t) = Cos(ax)$ and $v_o = v_0(x,t) = Sin(bx)$ for the equation (9) that satisfies the given boundary conditions and *a*, *b* are the unknown constants to be determined optimally. Substituting equation (6) into equation (9) and using equation (5), one gets a system of equation with *n+1* terms that needs to be solved simultaneously. Since computations are dependent on the value of u_o and v_o, a minor modification gives flexibility to choose the initial u_o [10,12,13]. For this purpose, the following homotopy is constructed

$$(w_t - u_{0t}) + p(r_{xx} + u_{0t}) = 0 \qquad (10)$$

$$(r_t - v_{0t}) + p(-w_{xx} + v_{0t}) = 0$$

where $w = w_0 + pw_1 + p^2 w_2 + ...$

and $r = r_0 + pr_1 + p^2 r_2 + ...$

The variables u and v can be obtained similar to equation (7) as

$$u=\lim_{p\to 1} w = w_o + w_1 + w_2 + ...$$

$$v=\lim_{p\to 1} r = r_o + r_1 + r_2 + ... \qquad (11)$$

After expanding with powers of *p* the equation (10) for *n=3*, i.e. the third power of *p*, one gets

$$\begin{aligned}
p^0: &\quad w_{0t} = u_{0t}, \\
&\quad r_{0t} = v_{0t} \\
p^1: &\quad w_{1t} + r_{0xx} + u_{0t} = 0 \\
&\quad r_{1t} - w_{0xx} + v_{0t} = 0 \\
p^2: &\quad w_{2t} + r_{1xx} = 0 \\
&\quad r_{2t} - w_{1xx} = 0 \\
p^3: &\quad w_{3t} + r_{2xx} = 0 \\
&\quad r_{3t} - w_{2xx} = 0
\end{aligned} \qquad (12)$$

The analytical solution of the equation (12) can be obtained as,

$$w_0 = Cos(ax)$$

$$r_0 = Sin(bx)$$

$$w_1 = b^2 t Sin(bx)$$

$$r_1 = -a^2 t Cos(ax)$$

$$w_2 = \frac{-1}{2} a^4 t^2 Cos(ax)$$

$$r_2 = \frac{-1}{2} b^4 t^2 Sin(bx)$$

$$w_3 = \frac{-1}{6} b^6 t^3 Sin(bx)$$

$$r_3 = \frac{-1}{6} a^6 t^3 Cos(ax) \qquad (13)$$

The optimal identification of the unknown parameters *a* and *b* in the trial function is determined using the least squares method over the given domain as in [10].

$$\int_0^1\int_0^1 R_1 \frac{\partial R_1}{\partial a} dxdt = 0, \quad \int_0^1\int_0^1 R_2 \frac{\partial R_2}{\partial b} dxdt = 0. \qquad (14)$$

where $R_1 = v_t - u_{xx}$, $R_2 = u_t + v_{xx}$

From the equation (21), one calculates $a=1$ and $b=1$. After substituting these values in the equation (11), one can write the optimal homotopy solution.

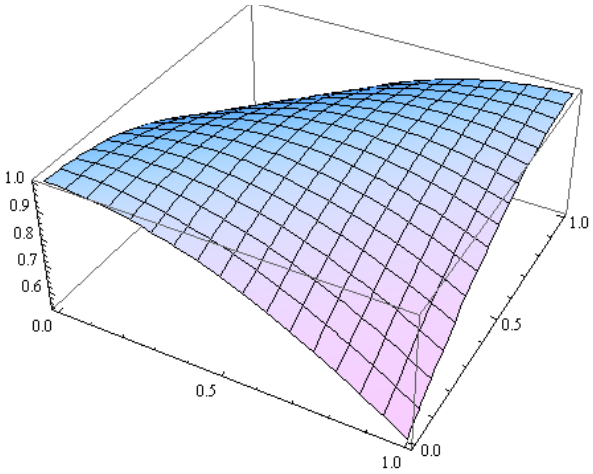

Fig. 1 Analytical solution of $u(x,t)$ for the intervals $0 \le x \le 1$ and $0 \le t \le 1$.

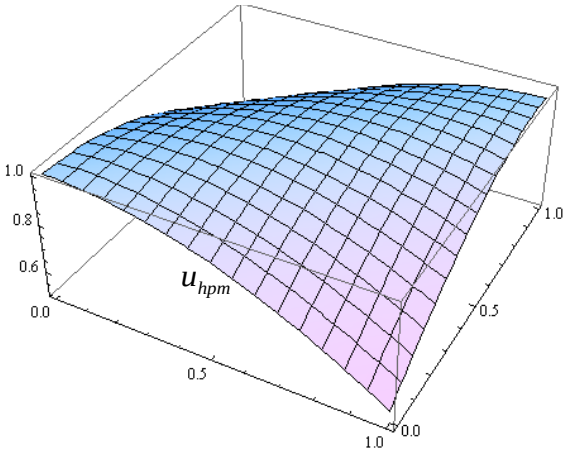

Fig. 2 HPM solution of $u(x,t) \cong w_0 + w_1 + w_2 + w_3$ for $0 \le x \le 1$ and $0 \le t \le 1$.

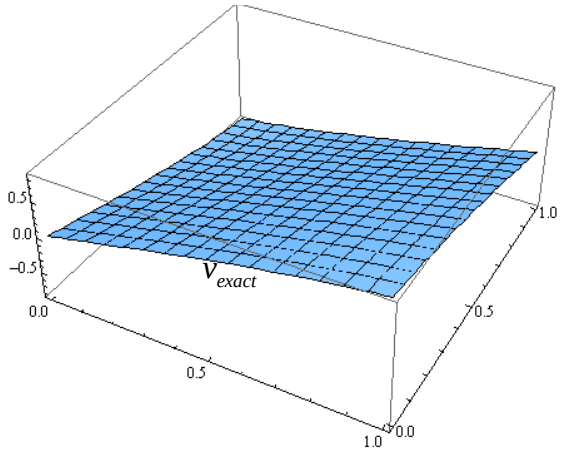

Fig. 3 Analytical solution of $v(x,t)$ for the intervals $0 \le x \le 1$ and $0 \le t \le 1$.

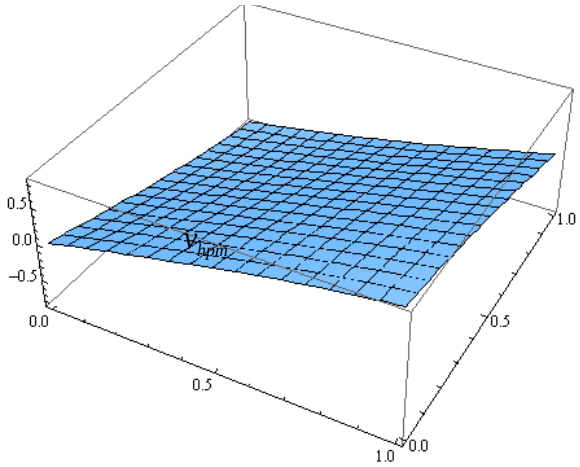

Fig. 4 HPM solution of $v(x,t) \cong r_0 + r_1 + r_2 + r_3$ for $0 \le x \le 1$ and $0 \le t \le 1$.

In Fig. 1, the analytical distribution of the $u(x,t)$ for the intervals $0 \le x \le 1$ and $0 \le t \le 1$ is given. The approximated solution of the $u(x,t)$ is given for the first four terms in the Fig. 2 for the same interval. In the Fig. 3, the analytical distribution of the $v(x,t)$ for the intervals $0 \le x \le 1$ and $0 \le t \le 1$ is given. The approximated solution of the $v(x,t)$ is shown for the first four terms in the Fig. 4 for the same interval.

4. CONCLUSION

The numerical analysis of complex PDEs were presented by using the Homotopy perturbation method (HPM) and the calculated numerical results were compared with the available exact solutions. The numerical HPM results were obtained by selecting a trial function and determining the unknown constants with the least squares method. This algorithm allowed an accurate and efficient use of the HPM with minimum numbers of iteration.

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COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF FIBER REINFORCED CONCRETE

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***Abstract*—Compressive strength and modulus of elasticity of concrete are two important properties involved in the design of reinforced concrete members and sections. The main objective of this investigation was to investigate the effect of fiber aspect ratio (l/d) and fiber volume on the compressive strength and modulus of elasticity of fiber reinforced concrete (FRC), and to study the relationship between compressive strength and modulus of elasticity of FRC. In this investigation, 19 fiber reinforced concrete mixes were produced by adding three different hooked-end steel fibers of aspect ratio of 60, 75, and 83 and also six different volumes of fibers namely 0.5, 1.0, 1.25, 1.5, 1.75, and 2.0 % by volume of concrete were added for each aspect ratio. As a result, good relationships between compressive strength of FRC and the fiber reinforcement index (FRI = V_f .l/d) between modulus of elasticity and FRI, and between compressive strength and modulus of elasticity of FRC were obtained.**

Keywords: steel fibers, compressive strength, modulus of elasticity, fiber reinforced concrete, fiber reinforcement index.

1. INTRODUCTION

The strength of a material is defined as the ability to resist stress without failure, and failure is identified with the appearance of cracks. Strength of concrete is related to the stress required to cause fracture and is signification with the degree of failure at which the applied stress reaches its maximum value [1]. The compressive strength is the most important overall measure of the quality of concrete, because strength is directly related to the structure of the hardened cement paste. Strength is not a direct measure of concrete dimensional stability and durability, but it has a strong relationship to the w/c ratio of the concrete. The w/c ratio affects durability, dimensional stability and other properties of the concrete by controlling porosity. Therefore, compressive strength is commonly used in specifying, controlling, and evaluating the quality of concrete [2]. The strength of concrete is affected by several factors including: degree of hydration, rate of loading, method of testing, testing conditions, specimen geometry, and the properties and proportions of the constituent materials such as the quality of fine and coarse aggregate, cement paste, and the paste-aggregate bond characteristics. As an example for testing conditions and specimen size effect: the strength of saturated specimens can be 15 to 20 % lower than that of dry specimens, and cube specimens can be 20 to 25 % higher strengths than that of cylindrical specimens. In general, larger specimens show lower strengths than that of smaller specimens [3].

Modulus of elasticity is the ratio of stress to the strain in the elastic region. Modulus of elasticity is important in structural member design. Modulus of elasticity depends on several factors, and these factors from different investigations are: (a) modulus of elasticity decreases as w/c ratio increases [4, 5], (b) modulus of elasticity (E_c) of concrete increases with its compressive strength, also increases with concrete age [6], (c) the elastic modulus increases with an increase in rate of loading [7], (d) concrete specimens tested in wet condition can exhibit 15 % higher modulus of

elasticity than tested in dry condition [3], (e) modulus of elasticity of high strength hardened cement paste has a very much higher modulus of elasticity than normal hardened cement paste, as a result, concrete has a more linear stress-strain curve and may exhibit an increased brittleness [8], (f) values of compressive strength and modulus of elasticity are influenced by the size of the cylindrical specimens. The relationship for calculating modulus of elasticity of 150 x 300 mm cylinder from 100 x 200 mm cylinders is in the from [9]: E_{c150} = 1.05 E_{c100}, (g) modulus of elasticity of HSC is influenced by the elastic properties of coarse aggregates [10], (h) modulus of elasticity of concrete is affected by the type of aggregates used in concrete [11], (i) elastic modulus is dependent on the aggregate characteristics including the mineralogy, aggregate content, surface texture, particle size, and age of the cement paste [12].

There are many investigations on the relationship between modulus of elasticity and compressive strength of plain concrete [13-24], on the other hand, there are few investigations on the relationship between compressive strength and modulus of elasticity of fiber reinforced concrete [25-29]. In these investigations, all the models proposed did not account for the fiber reinforcement index which involves the volume and the aspect ratio of fibers. On contrary, in this investigation the proposed model of modulus of elasticity for the fiber reinforced concrete involves the compressive strength, volume of fibers and aspect ratio of fibers.

2. EXPERIMENTAL STUDY

2.1 Materials

The w/c ratio used for the steel fiber reinforced concrete (FRC) mixes was 0.55. Blast-furnace slag cement complying with ASTM C 595 [30] was used for the FRC mixes. In all mixes, the coarse and fine aggregates used were limestone crushed rock. The maximum size of coarse aggregate used was 20 mm for all mixes. The specific gravity of aggregates used was 2.70. The fineness modulus of fine aggregate was 2.80. The amounts of cement, fine aggregate, coarse aggregate, water and high range water reducing admixture used in all concrete mixes were 410 kg/m^3, 916 kg/m^3, 829 kg/m^3, 225 kg/m^3 and 5 kg/m^3, respectively.

The type of the fibers used in all of the experiments throughout this investigation was bundled low carbon steel hooked-end fibers. The aspect ratios (l/d ratio) of these fibers were 60, 75, and 83. The lengths and diameters of the hooked-end steel fibers were 30, 60, and 50 mm and 0.5, 0.8, and 0.6 mm, respectively. Six different steel fiber percentages were added to the FRC as 0.5, 1.0, 1.25, 1.5, 1.75, and 2.0 % by volume of concrete (39.25 kg/m^3, 78.50 kg/m^3, 98.125 kg/m^3, 117.75 kg/m^3, 137.375 kg/m^3, and 157.0 kg/m^3, respectively) as shown in Table 1. The specimens used in all the experiments throughout this investigation were kept in their moulds for 24 hours as covered with wet burlap and polyethylene sheets. After about 24 hours of casting, the specimens were stripped and transferred to a standard curing tank where kept through the curing period at a constant temperature of 20 ± 2 ºC for 28-day.

2.2 Compressive Strength Test

The size of standard cylinder mould used for compressive strength of FRC was 150 x 300 mm. For each mix (total of 19 mixes), six test cylinders were casted in accordance with ASTM C 31 [30]. These cylindrical test specimens were cured and compacted according to the above mentioned procedures. The cylindrical test specimens were capped at both ends with a capping compound in accordance with ASTM C 617 [30].

2.3 Modulus of Elasticity Test

For each mix, six test cylinders (150 x 300 mm) were casted, compacted, cured, and capped at both ends as the same above mentioned procedures for compressive strength test. The chord modulus of elasticity was determined in this investigation for each mix of FRC (total of 19 mixes) at 28-day in accordance with ASTM C 469 [30]. Tests were conducted in a digital stiff closed-loop servo controlled hydraulic compression and flexural testing machine with a capacity of 3000 KN in compression and 200 KN in flexure. A compressometer consisted of two yokes was used, the lower one is rigidly attached to the specimen and the upper one attached at two diametrically opposite points so that it is free to rotate. At one point on the circumference of the upper rotating yoke, midway between the two support points, a pivot rod exist in order to maintain a constant distance between the two yokes. The opposite point on the circumference of the rotating upper yoke, the change in distance between the yokes, is equal to the sum of the displacement due to the specimen deformation and the displacement due to the rotation of the yoke about the pivot rod. For more accurate and sensitive results, a linear variable differential transformer (LVDT) was used in order to measure the deformations of the test specimens instead of dial gage. The load was applied at a constant rate of 0.3 MPa/s and up to 40 % of the ultimate load of the specimen.

The chord modulus of elasticity is calculated from the following Eq. 1 according to ASTM C 469 [30]:

$$E_c = \frac{\sigma_2 - \sigma_1}{\varepsilon_2 - 0.000050} \quad (1)$$

where, E_c is chord modulus of elasticity (MPa), σ_2 is stress (MPa) corresponding to 40 % of ultimate load, σ_1 is stress (MPa) corresponding to a longitudinal strain (ε_1) of 0.00005, and ε_2 is longitudinal strain produced by stress σ_2.

3. RESULTS AND DISCUSSION

3.1 Compressive Strength

Compressive strength test results of FRC are given in Table 1. Fig. 1 shows the compressive strength of FRC versus fiber volume fraction. As it can be seen from Fig. 1, as the fiber volume fraction increases, compressive strength increases for all FRC mixes. This increase in compressive strength is due to the fact that, as fiber volume fraction increases the spacing between fibers is reduced, therefore, this allows faster load transfer and support by the adjacent fibers. The maximum increase in compressive strength is 33.7 % obtained by the FRC mix with l/d ratio of 83 and at fiber volume fraction of 2.0 % compared to plain normal strength concrete. For FRC mixes with l/d ratio of 60 and 75, the maximum increase in compressive strength is 24.3 and 28.2 %, respectively, obtained at fiber volume fraction of 2.0 % compared to plain concrete.

Fig. 2 shows the relationship between compressive strength and FRI for FRC. From Fig. 2, it can be seen that as the FRI increases, compressive strength increases. From regression analysis, the following linear Eq. 2 was obtained for FRC (MPa). The R^2 for Eq. 2 is 0.93905. Therefore, compressive strength of FRC can be estimated from Eq. 2 in terms of plain compressive strength (f_c) and fiber reinforcement index.

$$f'_{nc} = f'_c + 0.06929\ V_f\ (l/d) - 1.26687 \quad (2)$$

where f_{nc} is compressive strength (MPa) of FRC.

3.2 Chord Modulus of Elasticity

Chord modulus of elasticity test results of FRC are given in Table 1. Fig. 3 shows of FRC versus fiber volume fraction. As it can be seen from Fig. 3, as the fiber volume fraction increases, the chord modulus of elasticity increases for all FRC mixes. The maximum increase in chord modulus of elasticity is 15.18 % obtained by the FRC mix with l/d ratio of 83 and at fiber volume fraction of 2.0 % compared to that of plain normal strength concrete. For FRC mixes with l/d ratio of 60 and 75, the maximum increase in chord modulus of elasticity is 9.46 and 14.0 %, respectively, obtained at fiber volume fraction of 2.0 % compared to that of plain normal strength concrete.

Fig. 4 shows the relationship between modulus of elasticity and FRI for FRC. From Fig. 4, it can be seen that as the FRI increases, chord modulus of elasticity increases for both FRC. From regression analysis, the following non-linear Eq. 3 is obtained FRC (GPa). R^2 for Eq. 3 is 0.90889. Therefore, modulus of elasticity of FRC can be estimated from Eq. 3 in terms of chord modulus of elasticity of plain concrete (E_c) and fiber reinforcement index (FRI = V_f (l/d)).

$$E_{ncf} = E_c + 0.01677\ V_f\ (l/d) + 0.000095\ [V_f\ (l/d)]^2 \quad (3)$$

where E_{ncf}, and E_c are chord modulus of elasticity (GPa) of FRC and plain concrete, respectively.

3.3 Relationship Between Compressive Strength and Chord Modulus of Elasticity

In Fig. 5, chord modulus of elasticity of FRC is plotted against square root of compressive strength multiplied by fiber volume fraction for l/d ratio of 60, 75 and 83. From Fig. 5, it can be seen that as the compressive strength increases, chord modulus of elasticity increases for each l/d ratio. From regression analysis, linear relationships between chord modulus of elasticity and compressive strength are obtained as Eqs. 4, 5 and 6 for l/d ratio of 60, 75, and 83, respectively. R^2 for Eqs. 4, 5 and 6 is 0.91990, 0.97424, 0.98180, respectively.

$$E_{nc60} = 0.24759\ \sqrt{f'_{nc}}\ V_f + 33.36591 \quad (4)$$

$$E_{nc75} = 0.35728\ \sqrt{f'_{nc}}\ V_f + 33.73567 \quad (5)$$

$$E_{nc83} = 0.37907\ \sqrt{f'_{nc}}\ V_f + 34.10814 \quad (6)$$

where E_{nc60}, E_{nc75}, and E_{nc83} are chord modulus of elasticities (GPa) of FRC for l/d ratio of 60, 75, and 83, respectively, and f'_{nc} is compressive strength (MPa) of FRC.

As a result of regression analysis, the non-linear Eq. 7 for FRC is obtained which quantifies the relationship between chord modulus of elasticity and, compressive strength and FRI. R^2 for Eq. 7 is 0.89496.

$$E_{ncf} = 33.47174 + 0.00497\ V_f\ (l/d) \times \sqrt{f'_{nc}} \quad (7)$$

where E_{ncf}, f'_{nc} are chord modulus of elasticity (GPa) and compressive strength (MPa) of FRC, respectively.

4. CONCLUSIONS

1. The percentage increase in compressive strength for FRC was 24.3, 28.2, and 33.7 % for l/d ratio of 60, 75, and 83, respectively.
2. Eq. 2 can be used to estimate compressive strength of fiber reinforced concrete in terms of plain concrete compressive strength and FRI. Eq. 2 is valid for compressive strength between 32.06 and 42.87 MPa.

Table 1. Fiber l/d ratio, volume of fibers, fiber reinforcement index (FRI), amount of fibers, compressive strength and chord modulus of elasticity test results of FRC.

Aspect Ratio (l/d)	**Volume of Fibers, V_f** (%)	**FRI** V_f (l/d)	**Amount of Fibers** (kg/m^3)	**Compressive Strength, f_c** (MPa)	***Increase in Compressive Strength, f_c** (%)	**Chord Modulus of Elasticity, E_{cf}** (GPa)	***Increase in Chord Modulus of Elasticity** (%)
-	0.0	0.0	0.0	32.06	-	33.781	-
60	0.5	30	39.25	32.66	1.87	33.951	0.50
60	1.0	60	78.50	34.11	6.39	34.242	1.36
60	1.25	75	98.125	36.28	13.16	35.125	3.98
60	1.5	90	117.75	37.46	16.84	35.413	4.83
60	1.75	105	137.375	39.27	22.49	36.021	6.63
60	2.0	120	157.0	39.85	24.30	36.978	9.46
75	0.5	37.5	39.25	33.73	5.21	35.058	3.78
75	1.0	75	78.50	34.63	8.02	35.549	5.23
75	1.25	93.75	98.125	36.61	14.19	36.042	6.69
75	1.5	112.5	117.75	38.31	19.49	37.277	10.35
75	1.75	131.25	137.375	39.63	23.61	37.611	11.34
75	2.0	150	157.0	41.17	28.42	38.512	14.00
83	0.5	41.5	39.25	33.99	6.02	35.613	5.42
83	1.0	83	78.50	35.26	9.98	36.265	7.35
83	1.25	103.75	98.125	37.09	15.69	37.173	10.04
83	1.5	124.5	117.75	39.73	23.92	37.634	11.41
83	1.75	145.25	137.375	41.27	28.73	38.432	13.77
83	2.0	166	157.0	42.87	33.72	38.910	15.18

* Increase with respect to plain concrete

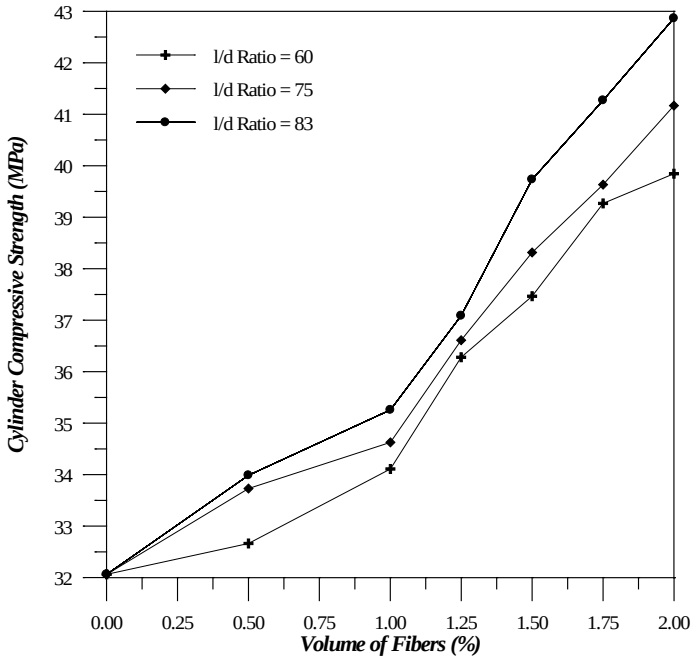

Fig. 1 Compressive strength of FRC versus fiber volume fraction for l/d ratio of 60, 75, and 83.

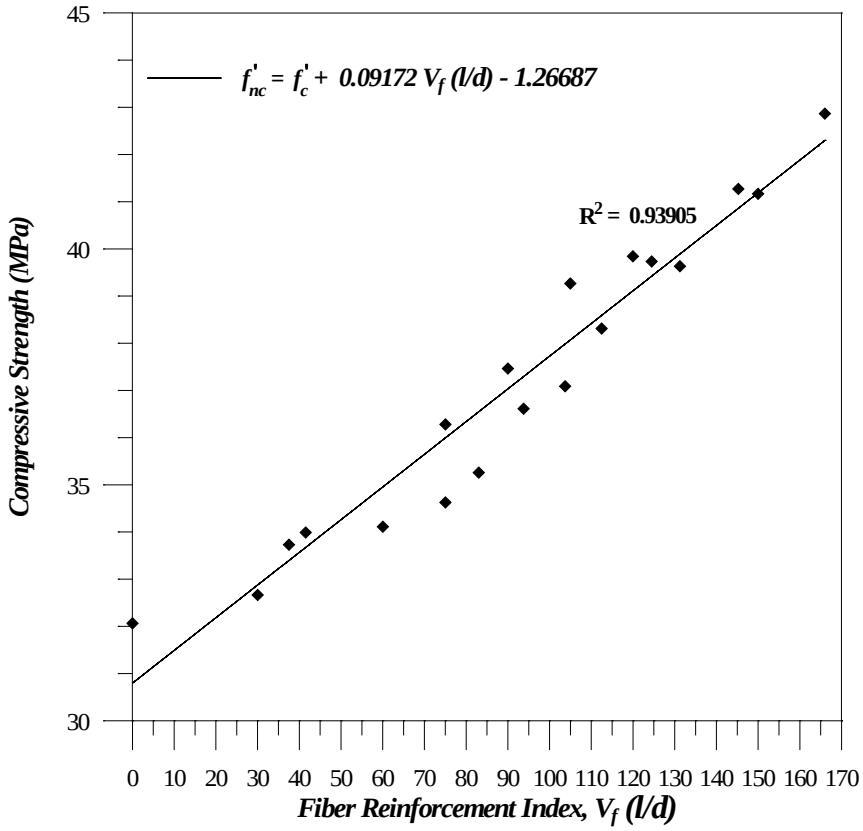

Fig. 2 Relationship between compressive strength and FRI for FRC.

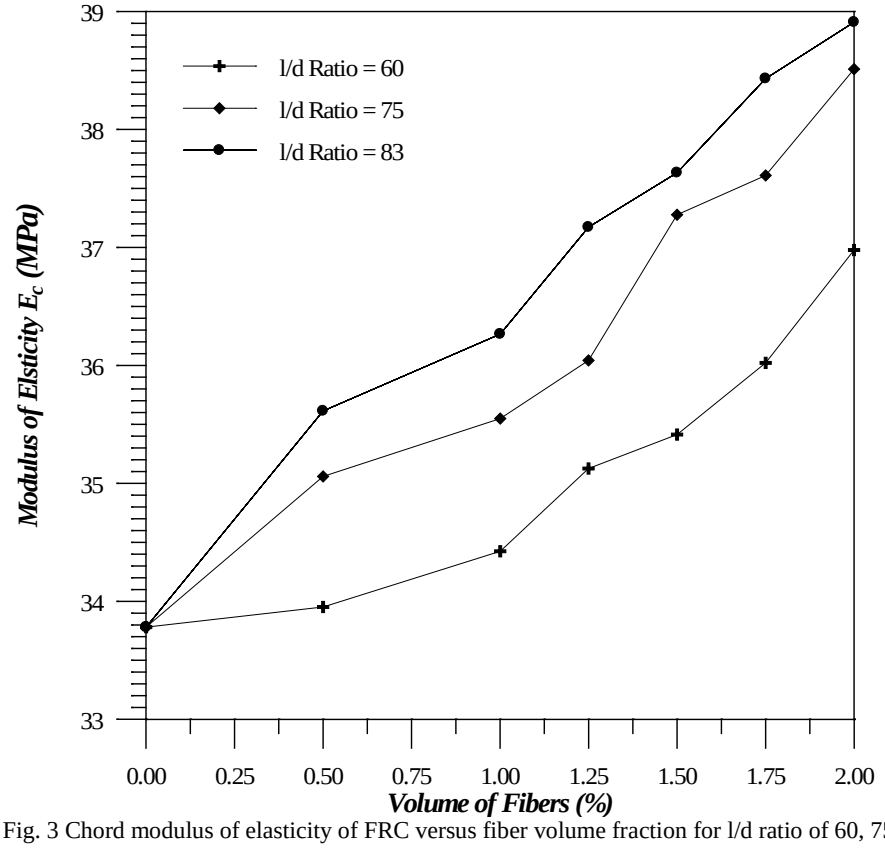

Fig. 3 Chord modulus of elasticity of FRC versus fiber volume fraction for l/d ratio of 60, 75, and 83.

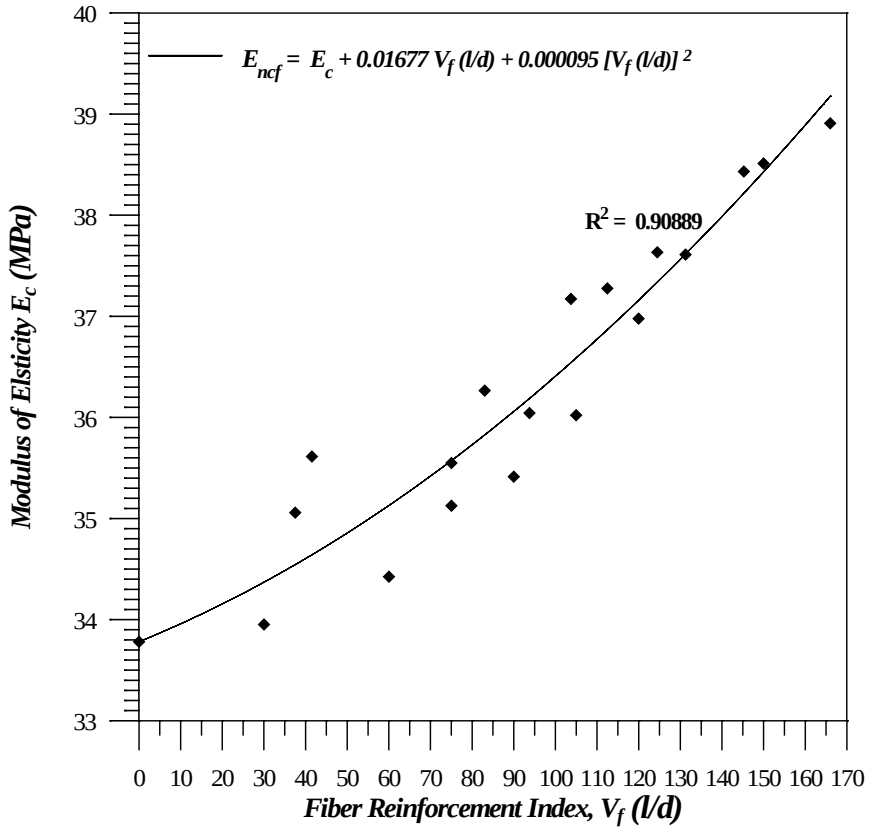

Fig. 4 Relationship between chord modulus of elasticity and fiber reinforcement index (FRI) for FRC.

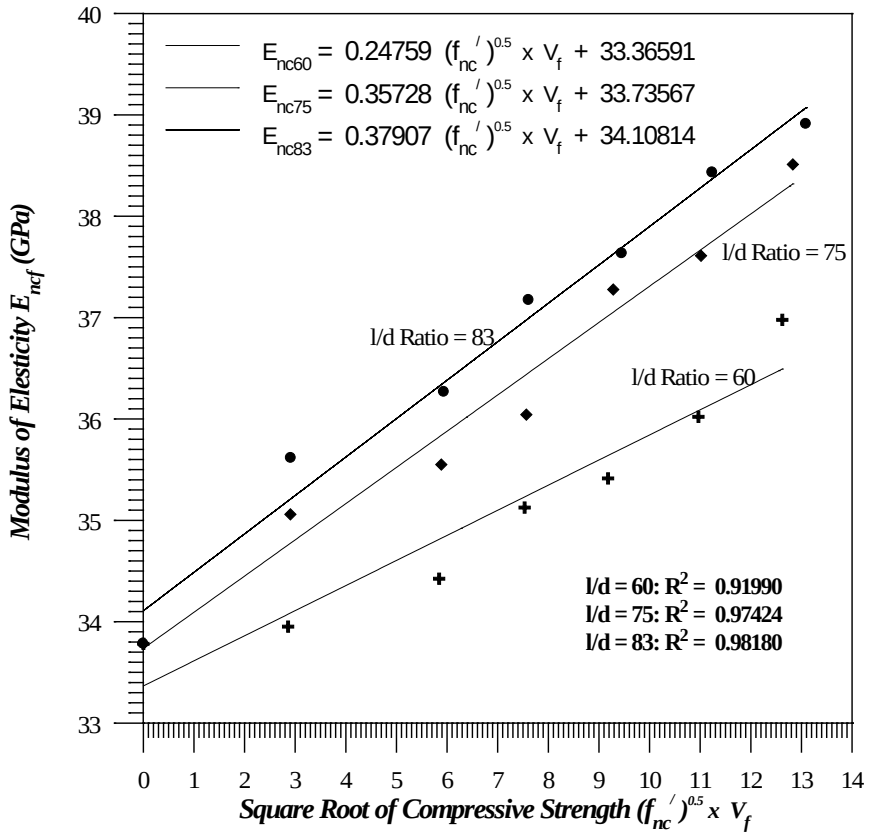

Fig. 5 Relationship between chord modulus of elasticity and square root of compressive strength of FRC for l/d ratio of 60, 75, and 83.

3. As FRI increases, chord modulus of elasticity increased for all FRC mixes. The highest increase in chord modulus of elasticity of FRC was for l/d ratio of 60, 75, and 83 was 9, 14, and 15 % obtained at fiber volume fraction of 2.0 % compared to that of plain normal strength concrete.
4. Eq. 3 was obtained in terms of FRI and E_c that can be used to estimate the chord modulus of elasticity of FRC (GPa). Eq. 3 is valid for plain concretes chord modulus of elasticity (E_c) between 33.781 and 45.547 GPa.
5. In General, chord modulus of elasticity increases with increase in compressive strength, fiber volume fraction, and l/d ratio (see Figs. 4 and 5). This is indicated by the linear Eq. 7 that is valid for compressive strengths between 32.06 and 42.87 MPa.

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COMPUTER BASED PHASE ANALYSIS OF CEMENT MORTARS WITH CHEMICAL ADMIXTURES BY IMAGE PROCESSING

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***Abstract*—Computer based microstructural phases of cementitious materials were investigated. Cement mortars incorporating with admixtures were prepared to define the microstructural development. Micrographs of polished sections of cement mortars were taken by using scanning electron microscope (SEM). Image analysis results indicate that cement mortars without chemical admixtures have a more regular microstructure compared with mortars with admixtures with reference to pore structure and phase development. Formation developments of calcium hydroxide, undifferentiated hydration products and pores were shown. Microstructural developments of undifferentiated hydration product and calcium hydroxide as a result of hydration of cementitious materials are slowly at early ages of hydration.**

Keywords: Porosity; Image Processing; Microstructure; Cement Mortar.

1. INTRODUCTION

Current experimental studies in engineering and material science have focused on the characterization of construction materials, extensively. Characterization of inner structure is necessary in order to understand the behavior of a material under different condition. Current technological advantages and computational capabilities lead material scientist to have more information on the microstructural properties of materials, and to improve the properties of investigated materials. Cement is also one of these materials affected by environmental conditions and needed recruitments according to innovations in construction technology.

In construction technology, cements are the most widely used materials among other construction materials. Products of the hydration reaction between cement and water serve as a binder for aggregates and other materials such as fibers. Hydration of ordinary Portland cement (OPC) affects the mechanical and physical properties of concrete in fresh or hardened state. Hydration is an exothermic reaction affected by the temperature and the fineness of cement grains. The rate of hydration is temperature dependent and increases with increasing temperature. The structure of the hydrated material may also be altered, at the same time [1]. Properties of these products affect the fresh and hardened state of concrete. Indeed, cement characteristics such as composition and fineness influence fresh and hardened concrete properties.

Hydration process is affected by many parameters such as the properties of cement, the environmental conditions such as heat and humidity. Since the rate of hydration is a function of temperature, the strength development of a given concrete depends on its time-temperature history, assuming that sufficient moisture is available for hydration [2].

During hydration process, cement paste microstructure changes by time especially at early ages. Early age characteristics can be classified into rheological behavior, heat evolution, setting, and

strength development. Hydration products also affect hardened concrete structures, as well as the durability of reinforced concrete.

Therefore, as a result of some requirements in concrete production; the production of high performance concretes which are more durable against physical, biological and chemical attacks has increased considerably. Chemical and mineral admixtures are the main ingredients for this type of concrete besides cement. Chemical admixtures improve the specified properties of concrete with side effects.

The most important criteria are knowledge of their effects on cementitious materials, determining the optimum usage limits, limiting the side effects and improve the efficiency mechanism. Generally chemical admixtures affect the early age mechanism of hydration process.

Knowledge of the microstructural evolution of cementitious materials at early age is helpful for forecasting their performance. Microstructural studies and numerical simulations become increasingly important to understand the formation of the microstructure of cementitious materials [3].

Some studies [4-7] show that image analysis of micrographs of cementitious materials are performed to quantify the microstructure of cement pastes for determination of porosity, pore structure and phases such as undifferentiated hydration products and anhydrous cement content.

To investigate polished sections of cementitious materials by microscopy techniques has become more important in order to investigate properties of the inner structure. There are several advantages of this technique. Although cement paste has a reasonably heterogeneous structure, this technique has a great capacity on determining the properties of microstructural formation in cementitious materials by quantitative measures. Indeed, microscopy applications improve the ability to characterize the microstructure of cement mortar and concrete, and helps in investigating effects of admixtures, determining durability problems and service life [8].

Since the debut in the early of 1980 [9,10], SEM micrographs under backscattered electron (BE) mode have shown a great potential to investigate cementitious materials. Flat sections of cementitious materials were prepared by obeying some specimen preparation techniques. Specimens covered by using epoxy are polished by diamond paste or micro sprays in order to get polished sections.

In this study, polished sections of cement mortars were prepared at early ages to investigate the development of microstructural phases of cement mortars. The effects of admixtures used in order to get different microstructural formations, on hydration process were obtained.

2. EXPERIMENTAL

2.1 Materials

Trisodium citrate (C6H5Na3O7–Ret.1) and citric acid (C6H8O7–Ret.2) were used in order to form different microstructures of ordinary Portland cement (OPC). The properties of organic admixtures are given in Table 1. Ordinary Portland cement (OPC) was obtained from Jura cement factory in Switzerland. The chemical composition, mineralogical composition, physical and mechanical properties of cement are given in Table 2, Table 3 and Table 4, respectively.

The mixes were prepared by using the retarders with different ratios (0.2 and 0.3 % of cement weight). The chemical admixtures and cement were mixed before adding mixing water. Mixes were dropped into little glasses. Therefore, these mixes were omitted and they weren't examined under SEM. To avoid the effects of cutting process required for image processing, during casting some amount of mixtures were cast into plastic caps. After 1, 2, and 7 days, hydrations of mixes were stopped by putting specimens into alcohol ispropylique during 5 days.The specimens were covered by polyester, and, then each specimen was examined by polishing process.

Each specimen was sanded by 600 and 1200 sandpapers. Each specimen was polished by 0.25, 1, 3 and 9 µm diamond paste for 120 seconds, after sanding [11]. The polished specimens were covered by carbon for SEM analysis.

Table 1. Properties of retarders.

Retarder	Molecular weight	Melting Point (C°)	Density	Water Solubility
Trisodium citrate	258.07	150	1.7	72 g / 100 ml
Citric acid	132.125	153	1.542	10 g / 100 ml

Table 2. Chemical composition of OPC.

Oxide	Comp. (wt.%)
SiO_2	19.22
Al_2O_3	5.24
Fe_2O_3	2.91
CaO	62.01
MgO	2.09
SO_3	3.35
K_2O	1.10
Na_2O	0.15
P_2O_5	0.22
ZnO	0.04
Cl	0.031

2.2 Method

2.2.1 SEM Studies

To avoid the effects of cutting specimens, during casting some amount of mixtures were cast into plastic caps. After seven days, hydrations of mixes were stopped by putting specimens into alcohol ispropylique during 5 days. The specimens were covered by epoxy, and, then each specimen was examined by polishing process. Therefore, the specimens were covered by carbon for SEM analysis.

The effects of retarders on the early microstructure of cement were investigated. The microstructural analysis was performed using SEM. Micrographs were taken with BE mode to determine the area ratio values indicating the ratio of a phase to the whole area of the polished section.

Table 3. Mineralogical composition of OPC.

Phase	**Comp. (wt.%)**
C_3S	51.17
C_2S	17.06
C_4AF	5,58
C_3A	13.00
CaO	0.26
MgO	3.3
Gypsum	1.6

2.2.2 Image Analysis

The second part of experimental studies implies microstructural investigations. Quantification of phases by performing image analysis requires, firstly, good specimen preparation and imaging technique to produce representative images, and secondly, a phase-segmentation that is objective, precise and reproducible. Among this microstructural investigations; preparation of materials, taking of micrographs of prepared sections, image processing and analysis take part.

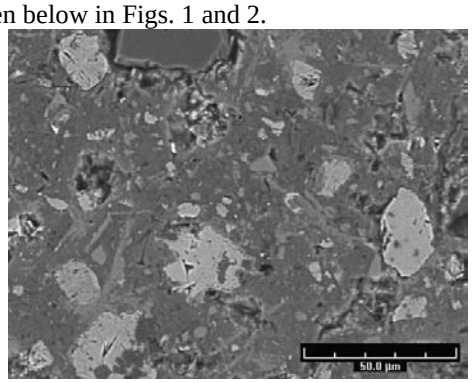
A high-quality original image is the prerequisite for accurate segmentation of phases and subsequent quantification steps. For optimum performance, the electron microscope operating configuration (accelerating voltage, beam current and working distance) must be set depending on the particular contrast produced by the specimen/detector system [12]. Furthermore micrographs of polished sections were taken with the same magnification, contrast and brightness level. Segmentation analysis was conducted to determine the area ratio values of phases.

To get better results depends on analyzing polished sections carefully. Phases in polished sections of cementitious materials should be segmented with accuracy and consistency. These accurate analyses lead meaningful quantitative data that can be used for comparative studies and to characterize development of microstructural phases of cementitious materials.

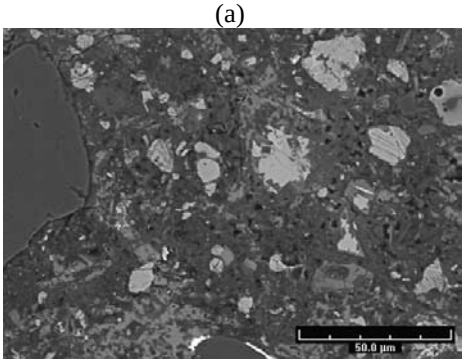
The microstructural analysis of cement mortars are performed by using image analysis program. Pore formation of cement mortars were investigated by using overflow method [12]. The development of microstructural phases of cement mortars by time was given. At the same dosage, the effects of chemical admixtures were obtained, indeed.

3. RESULTS

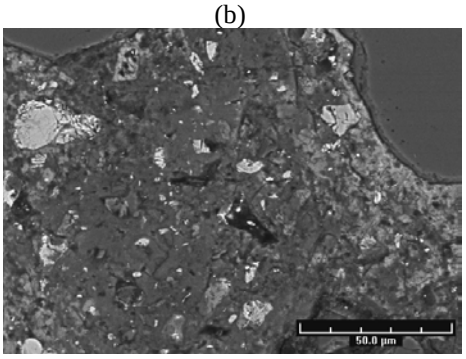
Image processing and image analysis were performed on the micrographs of polished sections of cement mortars. The micrographs with two different magnifications (2000X and 4000X) are given below in Figs. 1 and 2.



(a)

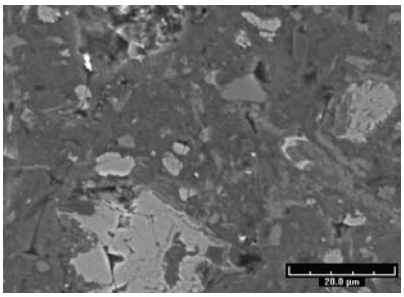


(b)

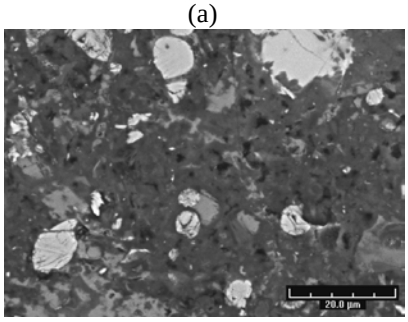


(c)

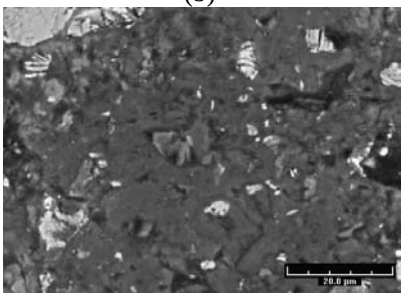
Fig. 1 Micrograph of 7-day a) Cement Mortar without a retarder, b) with 0.2 % of Ret.1 c) with 0.2 % of Ret.2, by 2000X magnification.



(a)



(b)



(c)

Fig. 2 Micrograph of 7-day a) Cement Mortar without a retarder, b) with 0.2 % of Ret.1 c) with 0.2 % of Ret.2, by 4000X magnification.

Pore structure development of cement mortars obtained by the image analysis results are given below. The control specimens prepared without a chemical admixture shows the minimum pore area ratio values at all ages (Fig. 3).

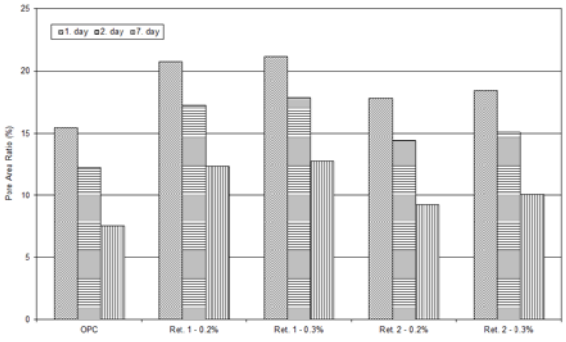


Fig. 3 Pore area ratio development of cement mortars.

Cement mortars with trisodium citrate indicating the most excessive retardation effect have the maximum pore area ratio values. Therefore, the phases such as undifferentiated calcium silicate hydrate, calcium hydroxide, unhydrated cement and pore were investigated (Fig. 4-6).

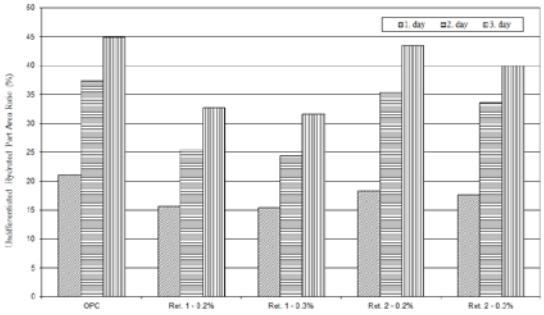


Fig. 4 Undifferentiated hydrated part area ratio development of cement mortars.

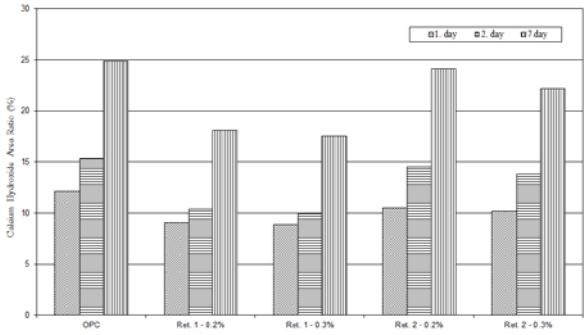


Fig. 5 Calcium hydroxide area ratio development of cement mortars.

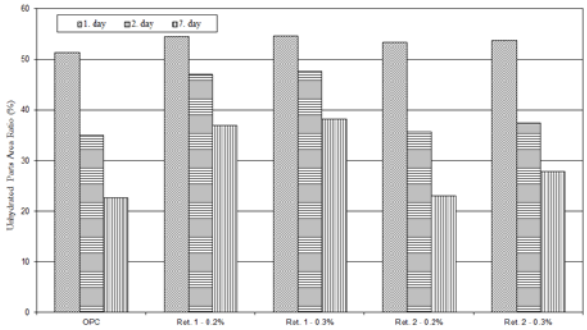


Fig. 6 Unhydrated parts area ratio development of cement mortars.

4. CONCLUSIONS

Computational capabilities and developments in analysis techniques for characterization of inner structure lead us to have a capacity to quantify the phases of cementitious materials. Electron microscopy and image analysis techniques on micrographs taken by electron microscopes bring good results to understand and determine the microstructural properties. Image analyses of micrographs taken by new technologies have a great capacity to realize the inner structure. Understanding the inner structure of cementitious materials leads engineers and material scientists to

improve the macro properties of cementitious materials such as strength and durability characteristics.

Investigation of microstructural phase developments of cement mortars incorporating with chemical admixtures at early ages were performed in this study. Two different chemical admixtures were used in order to form different microstructural inner body. All chemical admixtures have retardation effect on the hydration process of OPC.

There can be seen more anhydrous cement phase in cement mortars in case of using retarders at early ages. This effect causes more porosity in the microstructure of cement mortars. Indeed, the porosity increases as the amount of retarder in cement mortar increases.

The maximum area ratio values of pore structure decreasing by time are obtained by adding trisodium citrate (C6H5Na3O7–Ret.1) with 0.2%. Furthermore, "Undifferentiated hydrates part/Unhydrated parts area ratio" values increase according to hydration process by time (Fig. 7) for all mixes. Obviously, due to the development of hydration process by time, undifferentiated hydrated parts increase while unhydrated cement parts decrease.

Indeed, "Pore/Undifferentiated hydrates part area ratio" values decrease by time (Fig. 8). This trend indicates that undifferentiated hydrates such as calcium silicate hydrate form and take place in the pore structure of cement mortars.

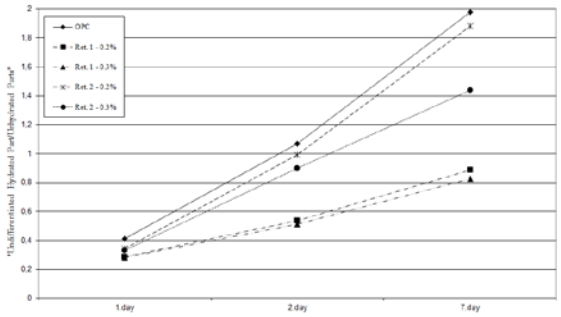


Fig. 7 Development of "Undifferentiated hydrates part/Unhydrated parts area ratio".

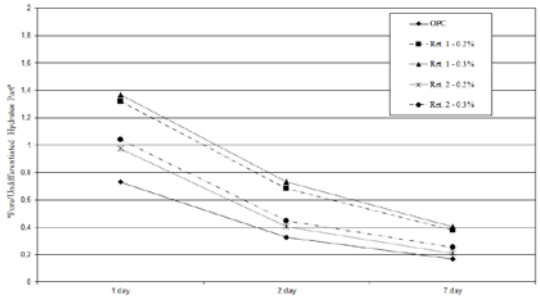


Fig. 8 Development of "Pore/Undifferentiated hydrates part area ratio".

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PROPERTIES OF MORTAR CONTAINING WASTE GLASS AS FINE AGGREGATES

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***Abstract*—The effect of replacement of sand with different color waste glasses namely brown, transparent and green on properties such as flowability, compressive strength and flexural strength of mortars were studied. Percentage replacement level of 100% and 50% of sand with fine waste glass were considered. The purpose of using ground waste glass was to investigate whether glass could be used as a replacement for sand in order to protect the environment by saving more landfills. It was found that: (a) decrease in glass replacement level decreases workability, (b) Mortars contained transparent glass had the highest compressive strength, (c) compressive strength of green glass mortar was higher than that of the brown glass, and (d) transparent glass had the highest flexural strength, green was in second place and brown was in third place.**

Keywords: waste glass, compressive strength, flexural strength, flow table test

1. INTRODUCTION

The amount of waste glass has increased in recent years because of the increasing demand on the use of glass products and also because of the growing industry. The majority of waste glass is not being recycled and dumped into landfill sites and this leads to a serious environmental pollution problems [1, 2]. Landfill sites are limited, and landfilling waste glass is undesirable because they are not biodegradable and is becoming more difficult. However, when waste glasses are reused in making concrete, the production cost of concrete will decrease [2].

Concrete is an ideal material for the reuse of crushed glass or other waste materials. Crushed glass has been used as aggregate replacement resulting in a poor bond with concrete exhibiting alkali–silica reaction (ASR). On the other hand, when glass is reduced to a finer particles and used as cement replacement can produce good quality concrete that does not exhibit ASR [3]. In an investigation [4] on the use of milled glass as a partial replacement of cement, it was concluded that benefitial pozzolanic reaction can take place when milling the waste glass to sub-micron particle size. The high surface area of milled waste glass changes the kinetics of chemical reaction towards beneficial pozzolanic reaction hence utilizing the available alkalis before the production of a potential ASR gel.

The use of pozzolanic glass fines is newly introduced method to mitigate ASR expansion. Because of the amorphous structure of the glass, it undergos pozzolanic reaction with the presence of alkaline activator such as lime, cement and the alkalis in cement and contribute to the calcium silicate hydrate structure and the hydration products [5]. From a review [6] on the use of waste glass in the production of cement and concrete, it was concluded that (a) test results indicated that the ASR expansion decreases as glass replacement increases, and is under the deleterious limit if the glass replacement is 50% or more, (b) the combined use of other supplementary cementing materials

such as fly ash, blast furnace slag and metakaolin decreases the expansion from ASR, and (c) Lithium salt can be an effective additive to prevent the ASR expansion of concrete containing glass powders. In another investigation on the pozzolanic reactivity of glass powders [7], it was concluded that, finely ground glass powders exhibited very high pozzolanic activity. The finer the glass powder the higher its pozzolanic activity. Results from ASTM C 1260 testing indicated that the replacement of Portland cement with ground glass powder reduced the ASR expansion, although it was not as effective as fly ash. A number of studies [8 - 12] have proven the suppressing ability of fly ash, metakaolin, silica fume when used along with crushed glass on ASR.

Approximately 40000 green and transparent bottles are getting wasted every day. If % 40 of them could be collected, this means that we will have about 16000 bottles per day collected in TRNC. Green bottles are nearly 7000 and transparent bottles are nearly 9000. General average of brown bottles is about 12500 bottles/day but most of these bottles are collected and reused by the manufacturer, therefore, brown bottles are not considered as a wasted glass and can't be used for manufacturing concrete or mortar. The mass of the wasted glass per day can be calculated as follows:
Average mass of a green bottle = 350 gr
Average mass of a transparent bottle = 550 gr
Mass of green bottles=7000×0.350=2450 kg/day
Mass of transparent bottles=9000×0.550=4950 kg/day

Content of glass sand is 687 gram in 2117 gram mortar mix. In TRNC 5390-7550 kg mortar can be manufactured from green bottles and 10170-15250 kg mortar can be manufactured from transparent bottles.

In Turkish Republic of Northern Cyprus (TRNC) there is no plant for recycling of waste glass so using waste glasses in material industry can bring many profits for TRNC. The objectives of this investigation was to study the properties of mortar made with sand replacement of fine waste glass. Replacement percentages were 100% and 50%.

2. EXPERIMENTAL WORKS

2.1 Materials

Ordinary Portland cement (42.5 MPa), ASTM standard sand, green glass, brown glass and transparent glass were used. All glass sizes used were finer than 5 mm.

2.2 Preparation of Glass and Feasibility Works

Approximately 500 bottles were collected from bars and restaurants and divided into three groups. These groups contained green bottles, brown bottles and transparent bottles. The glass bottles were crushed by hammer and then the crushed glass was refined further by using Los-Angeles abrasion testing machine. After the crushing process of bottles by the machine, glass was sieved on 5 mm sieve and the glass finer than 5 mm was used as a sand replacement of 0%, 50% and 100% throughout this investigation.

2.3 Mixture Proportioning

The mixture proportioning of materials for the standard mortar was one part of cement to 2.75 parts of graded standard sand or waste glass sand by weight, and the water/cement ratio was 0.485.

(a) Control mix (without glass)

Cement, gr	500
Sand, gr	1375
Water, gr	242

(b) 1st mix (with waste glass; brown, transparent, green - 100% replacement by weight of sand)

Cement, gr	500
Waste glass, gr	1375
Water, gr	242

(c) 2nd mix (with waste glass; brown, transparent, green - 50% replacement by weight of sand)

Cement, gr	500
Sand, gr	688
Waste glass, gr	687
Water, gr	242

(d) 3rd mix (with waste glass; brown - 50% replacement by weight of sand with less water content)

Cement, gr	500
Sand, gr	688
Waste glass, gr	687
Water, gr	210

In the 3rd mix design less water content was used due to the fact that the water absorption of glass is zero, especially for mortar mixes with sand replacement of 100% by glass sand where this causes an increase in the water content of the mortar mix and results in a decrease in strength and leads to difficulties when comparing among the test results.

2.4 Testing Equipment

A closed-loop compressive and flexural strength testing machine of capacity of 2000 KN in compression and 300 KN in compression used only to test specimens in compression (see Fig. 1), flexural strength testing machine (see Fig. 2), scale of 2000 gr capacity, specimen mould of 50x50x50 mm for compressive strength, specimen molds of

40x40x160 mm for flexural strength, mortar mixer, flow table and flow mould (see Fig. 3), tamper, and trowel were used in this investigation.

Fig. 1 Compressive strength test.

Fig. 2 Flexural strength testing machine.

Fig. 3 Flow table and flow table mold.

2.5 Testing Procedure

Mortar mixing procedure was done according to ASTM C 305-99 [13]. Water was placed first in the bowl, then cement was added and mixed within 30 s and continued mixing for another 30 s at same speed, then speed was changed to medium for additional 30 s, then the mixer was stopped and mortar stood for 1.5 minutes and finally the mixture mixed for one more minute.

The flow table test was done in accordance with ASTM C 1437-07 [14]. The mortar was placed as a layer of about 2.5 cm in the flow mold and tapped 20 times, then the second layer of mortar was placed and tamped for 20 times, then the excessive mortar was cut with a trowel, after that the mold was removed and the flow table was dropped 25 times in 15 s, then finally the diameters along the four scribed lines were measured and the sum of four readings was recorded as the percent increase in the original diameter.

Compressive strength determination was done according to ASTM C 109 [15]. 7 and 28 day strengths were determined to ascertain the strength gain as a function of time. Mortar cube test specimens (50x50x50 mm) for the determination of compressive strength are shown in Fig. 4.

Fig. 4 Cube test samples.

The flexural test of mortar prism specimens (40x40x160 mm) was done according to ASTM C 348 [16]. Test specimens were tested at 7 and 28 day (see Fig. 5).

Fig. 5 Samples of flexural strength test prisms.

3. RESULTS AND DISCUSSION

3.1 Flow Test Results of 1st Mix Design

Fig. 6 shows the flow table test results for control, brown, transparent, and green glass of a sand replacement of 100% replacement level. As it can be seen from Fig. 6, the flow test results of control, brown, transparent, and green glass of sand replacement level of 100% were 1.84, 2.13, 1.94 and 1.97, respectively. the use of glass as a replacement of sand increased workability. The workability increased 15.76% for brown glass, 5.43% for transparent glass and 7.06% for green glass when compared to the control mix (sand only).

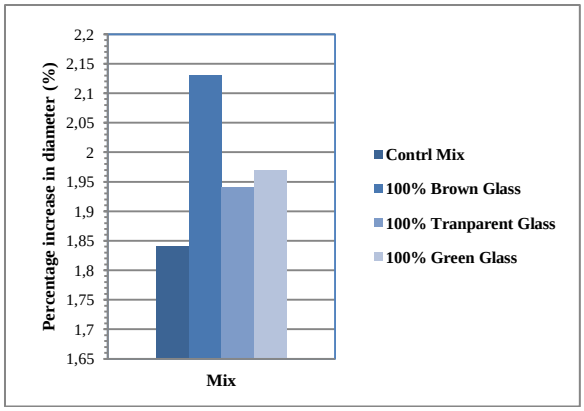

Fig. 6 Flow table test results for control, 100% brown, 100% transparent and 100% green glass replacement level.

3.2 Compressive Strength Test Results For 1st Mix Design

The compressive strength test results of control, 100% brown glass, 100% transparent glass and 100% green glass replacement level were 17.83, 15.37, 16.32 and 15.48 MPa, respectively (see Fig. 7). The test results of the compressive strength of the first mix design showed a decrease in compressive strength when replacement level of glass was 100%. However, samples containing glass had a better workability. Although the workability of samples containing 100% glass increased, the compressive strength decreased. The decrease in compressive strength of brown glass, transparent glass, and green glass samples were 13.80%, 9.25% and 13.19%, respectively.

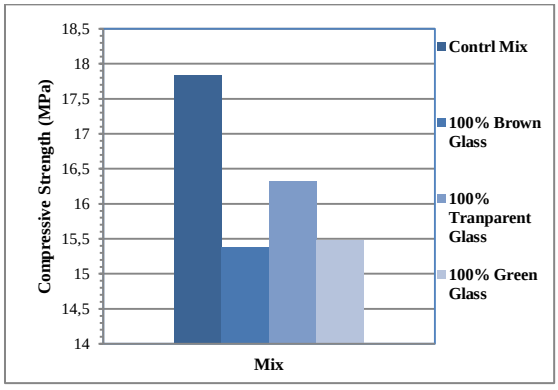

Fig. 7 Compressive strength test results for control mix and 1st mix (100% replacement level of glass).

3.3 Flow Test Results for 2nd and 3rd Mix Design

The flow table test results for control (1st Mix), brown glass (2nd Mix), transparent glass (2nd Mix), green glass (2nd Mix) and brown glass (3rd Mix) at glass replacement level of 50% were 1.85, 2.03, 1.90, 1.92, and 1.75 (with less water), respectively (see Fig. 8).

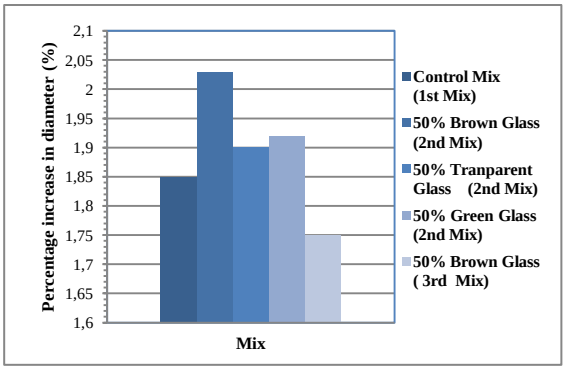

Fig. 8 Flow table test results of control (1st Mix), brown glass (2nd Mix), transparent glass (2nd Mix), green glass (2nd Mix) and brown glass (3rd Mix).

3.4 Compressive Strength Test Results of 2nd and 3rd Mix Design

Fig. 9 shows compressive strength test results of sand, 50% brown glass, 50% transparent glass, 50% green glass, and 50% brown glass with reduced water. As it can be seen from Fig. 9, the compressive strength test results of sand, 50% brown glass, 50% transparent glass, 50% green glass, and 50% brown glass with reduced water were 15.77, 14.36, 14.89, 14.63 and 16.48 MPa respectively. Compressive strength test result shows the effect of glass replacement level clearly. Compressive strength of mortar with replacement level of 50% of waste glass decreased. The compressive strength of mortar made with transparent glass was the highest, followed by the green glass followed by the mortar made with brown glass. Decrease in compressive strength of brown glass, transparent glass, and green glass samples were 9.82%, 5.91% and 7.79%, respectively.

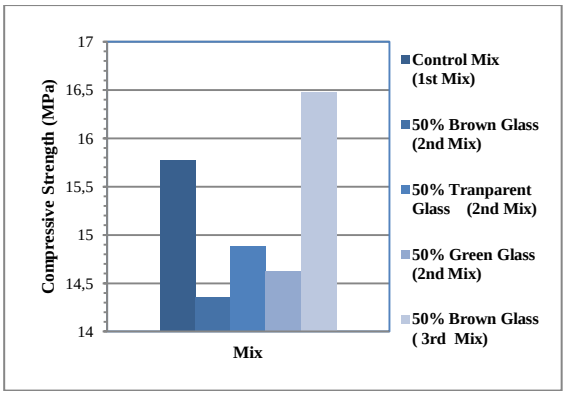

Fig. 9 Compressive strength test results of control mix, 50% brown glass, 50% transparent glass, 50% green glass, and 50% brown glass with reduced water.

3.5 Flexural Strength Test Results of 2nd and 3rd Mix Design

Fig. 10 shows flexural strength test results for three mortars of sand replacement of 50% glass and control mix (without glass replacement). As it can be seen from Fig. 10, the flexural strength test results for sand, 50% brown, 50% transparent, 50% green and 50% brown glass with reduced water were found to be 8.03, 6.93, 7.31, 6.89 and 8.81 MPa, respectively. Flexural strength of mortar with replacement level of 50% glass was decreased. Flexural strength of mortar containing brown glass, transparent glass and green glass were 15.87%, 9.84% and 16.54%, respectively.

Increase in flexural strength of transparent glass samples was 9.71%. When water content of mortar made with 50% brown glass was reduced, it had a higher flexural strength than that of the control mix.

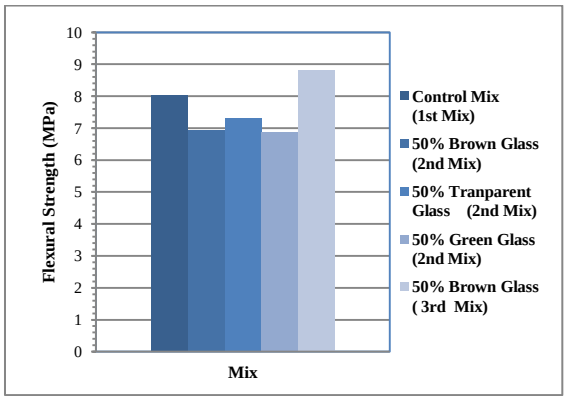

Fig. 10 Flexural strength test results for three mortars of sand replacement of 50% glass and control mix (without glass replacement).

4. CONCLUSION

This research studied the feasibility of the use of glass as sand replacement in mortar mixes and studies behaviors of mortars made with different color glasses. The following conclusions could be drawn from this study:

1. Decrease in glass replacement level decreases workability. However, both mortars containing 100% and 50% glass replacement levels showed good results in flow test. Brown glass mortar was the most workable, followed by the green glass mortar, followed by transparent glass.
2. Mortars contained transparent glass had the highest compressive strength. Although compressive strength of mortar made with 50% transparent glass as sand, it was close to compressive strength of the control sample. Compressive strength of mortars contained brown and green glass were close to each other. Compressive strength of green glass mortar was higher than compressive strength of the brown glass. Decreasing glass percentage provided better compressive strength.
3. Percentage decrease in flexural strength was greater than the decrease in compressive strength. Transparent had the highest flexural strength, green was in second place and brown was in third place.
4. Reducing water content resulted in a higher compressive strength and flexural strength but the workability of mortar was reduced. After water content was reduced, glass sand mortars had higher compressive strength and flexural strength but had less workability than control samples.
5. If satisfactory results of compressive strength, flexural strength and workability are desired, the followings can be used:
   - Mortar containing 50% transparent glass as sand performed well in flow test, compressive strength and flexural strength tests. Workability of transparent glass was not as good as brown glass sand mortar but a little decrease in water content may be applied to obtain a better results.
   - Decreasing water content of mortar containing 50% brown glass as sand resulted in a higher compressive and flexural strengths but low workabilities. If that water content could be balanced with brown glass quantities, satisfactory results could be obtained in workability, compressive strength and flexural strength.

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BOND PERFORMANCE OF SELF COMPACTING CONCRETE AND PRESTRESSING STRAND USING DIRECT TENSION PULL-OUT TEST TECHNIQUE

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Abstract —**The successful use of prestressing strands in concrete specimens is dependent on the effective transfer of force between the strands and the surrounding concrete. Variation in strand supply and changes in concrete properties are common in the precast concrete industry. Alterations in the concrete or strand properties can produce considerable reductions in the bond strength between prestressed strand and concrete. To ensure that bond quality is not compromised a pull-out bond test has been developed and utilized to monitor the bond quality in prestressed concrete bridge beam production. The methodology, development of the procedure, the repeatability and reproducibility of the test methods are presented. Also the bonding performance of the self compacting concrete (SCC) and high early strength concrete (HES) were compared.**

Keywords: Prestressing strand, Bonding tests, Direct tension pull-out test.

1. INTRODUCTION

In the study, a new strand-concrete bond test methods was developed. Various bond test methods have been developed with the goal of pre-qualifying the strand; however, these methods do not necessarily validate the use of the strand in its intended application. Quick and simple methods that validate the strand-concrete bond in prestressed specimens are not in widespread use. Strand can also be used in a variety of concrete types such as normal strength, high early strength (HES), and self consolidating (SCC). The variation of the strength of the concrete plays a vital role in the bond transfer process. There has also been a growth in the use of SCC in the pre-stressed/precast industry despite a lack of research investigating the characteristics of the bonding of the product to prestressing strands. Thus, there is a need for a testing program that to explore both the bond mechanics of prestressing strand and the bond characteristics of SCC.

A new test method described in this paper covers the determination of bond quality of a seven-wire strand when used in prestressed concrete applications by means of a destructive pull-out test of a strand from a concrete prism. This study details the method developed for the examination of both strand quality and concrete bond properties.

2. EXPERIMENTAL PROGRAM

The experimental program uses a new test method to fulfill the research objectives of addressing a lack of information over SCC bonding properties, developing a new test method, and quantifying individual bond mechanisms .

2.1 Direct Tension Pull-out Test (DTPT)

The direct tension pull-out test (DTPT) was conducted to evaluate the ability of a new test for bond strength to accurately depict the bond performance of various prestressing materials under the full distribution of bond mechanisms associated with prestressing. The DTPT, shown in Fig. 1, combines the realistic bond mechanisms of a flexural beam test with the ease of operation of a direct pull-out test. DTPT specimens are short beams of rectangular 6.5” x 12” cross section that are prestressed using a single seven-wire strand. The prestressing steel runs through the centroid of the cross-section so that concrete cover thickness around the strand can be maximized. The

specimens were fabricated so that 40" of strand protrudes from the end of the beam.

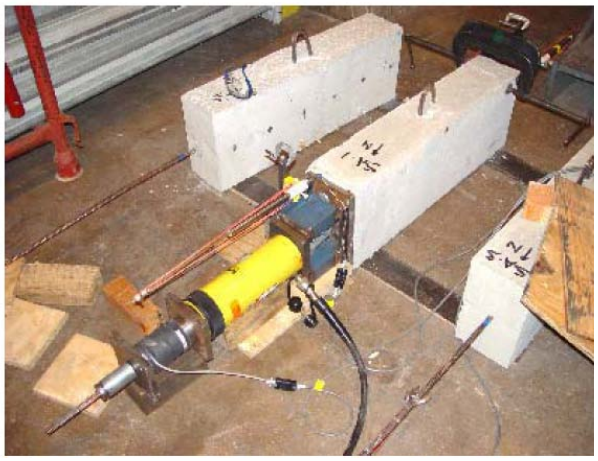

Fig. 1 Direct tension pull-out test [1].

3. DIRECT TENSION PULL-OUT TEST DEVELOPMENT

The test method covers the determination of the bond quality of seven-wire strand when used in prestressed concrete applications using a destructive pull-out test of a strand from a concrete prism. The method is developed for the examination of both strand quality and concrete bond properties [1].

3.1 Test Specimens

The test specimens are cast in a horizontal orientation on the precast plant prestressing bed around the stressed strand. An initial jacking stress of 75% of the strand ultimate strength is recommended.

The specimens are situated at the end of the strand run, in line with the precast element that is being fabricated. Adequate free strand length is provided on the unbonded end to allow for jacking (a length of 40 in is recommended). A minimum of three specimens are computed for each combination of strand type and concrete mix design. A minimum of six companion concrete cylinders 6 in by 12 in are fabricated and tested according to ASTM C39 [2] to determine the compressive strength of the concrete at transfer and at the time of testing. The cylinders are match cast with the specimens to ensure consistency between the specimen and cylinder strengths. Concrete mix design and concrete hardened compressive strength were given Table 1 and Table 2 respectively.

The initial stress is transferred in accordance with plant procedures. The minimum release strength of the concrete is 6800 psi. The release strength is verified before release by performing compressive strengths tests on match cast cylinders. After the release of stress the strand was cut flush with the surface of the specimen at the dead end and a 40 in length of strand is left on the live end (debonded side). The cut side is ground flat to provide a smooth bearing surface for the displacement measurements. The pull-out tests of the three specimens are conducted sequentially over duration of less than 1 hour to ensure consistency in concrete strengths.

The low relaxation 270 ksi strand was used in all facets of the project. The mechanical properties of prestressing strand was presented in Table 3.

3.2 Measuring Apparatus

The movement of the strand with respect to the concrete is measured on the dead end. A dial gauge or displacement transducer with a graduation of 0.001 in and a minimum stroke of 0.5 in is used. The load measured with a calibrated load cell in line with the strand being tested.

Table 1. Concrete mix design.

Material Type	**HESC**	**SCC**
Total Cement (lb/yd^3)	750	849
Slag Cement (%)	34	25
Fine Aggregate SSD (lb/yd^3)	1172	1283
Coarse Aggregate (lb/yd^3)	1383	0
Coarse Aggregate #8 (lb/yd^3)	552	1651
Water / Cement Ratio	0.34	0.32
Water Reducer (oz/yd^3)	60.0	136.2
Retarding Admixture (oz/yd^3)	4.0	0
Air Entrainment (oz/yd^3)	2.4	2.0
Viscosity Modifying Admixture (oz/yd^3)	0	16.0
Coarse AggregateVolume %	39	34
Target Air Content (%)	NA	NA
Target Slump / Spread (in.)	NA	NA

Table 2. Hardened concrete properties.

	Compressive Strength [f'$_c$]	**Elastic Modules [E$_c$]**
	[psi]	[ksi]
HES	7300	5724
SCC_1G	8815	5163
SCC_2G	6806	4537

Table 3. The mechanical properties of prestressing strand.

	Elasticty	**Strand**	**Ultimate**	**Fracture**
Diameter	modulus	area	stress	strain
	[ksi]	[in^2]	[ksi]	[%]
0.5"	29080	0.1639	280.3	0.0854

The testing apparatus consists of a through-hole ram or universal testing machine, a reusable strand chuck and bearing plate. The bearing plate has an adequate bearing area to prevent crushing of

the concrete surface. The through-hole ram has a controller that allows for a smooth pressure increase of 20 kips/min.

3.3 Loading Procedure

The specimens are tested in the horizontal position. The displacement gauges are attached to the strand at the live end and the dead end of the specimen. At the dead end the displacement gauge is attached to one of the outer strands. The inner wire should be avoided as relative slip may occur between the inner and outer wires resulting in an inaccurate measurement.

The jack and bearing plates are supported with a blocking plate prior to loading to ensure that the strand is centered in the test setup. Load is applied at a maximum rate of 20 kips/min or if displacement control is available at a rate less than 0.02 in/min.

The applied load and displacements are regularly recorded such that a minimum of 10 points are recorded prior to a dead end displacement of 0.01 in, bond stress reaches critical bond strength level. The load is recorded to the nearest pound and the displacement recorded to the nearest 0.001 in. It is important to note the load level when the dead end reaches a displacement of 0.01-in F_i (Critical bond strength), and 0.1 in. The recording is continued until;

- a decrease in load by 25% is measured,
- the strand fractures,
- a displacement of 0.5 in is measured at the live end.

4. EXPERIMENTAL RESULTS

In this study [2], a DTPT was performed on three groups of specimens. 270 K oversize low relax prestressing strand of 0.5 in. diameter was used for all the samples. The first group used HES. The 28 day compression strength of this concrete is 7300 psi. The characteristic feature of this concrete is that it reaches high strength in a short period. SCC of two types SCC-1G and SCC-2G was used in the second and third groups. The characteristic feature of these types of concrete are that they are self-placing. The compressive strength of these concrete types are 8815 and 6806 psi, respectively [1].

The bonding strength of the specimens was determined as having displacement values of 0.01in, 0.1 in. occurring at the dead ends of the strands. The experiments were terminated when there was 0.5 in. of displacement, the LVDT end contacts the concrete surface, or for safety reasons.

4.1 High Early Strength Concrete

Considering the DTPT results, the force applied during 0.01 and 0.1 in. of displacement determined at the dead end of the strands of specimens using HES are 21.65 and 25.97 kips, respectively. The maximum force applied to the specimens in this group during the pull-out test was determined as 28.08 kips. The transfer length and development length determined to the specimens in this group during pull-out test was measured as (L_{ti}) = 51.09 in. and (L_{ult})=56.28 in. respectively. The experiments in this group were terminated when 0.5 in. displacement measured at live ends of prestressing strands or end contacted the concrete surface [1].

4.2 Self-Compacting Concrete (SCC_1G)

Considering the experiment results for SCC_1G, the force applied during 0.01 and 0.1 in. of displacement determined at the dead end of the strands of these specimens was 26.50 and 32.79 kips, respectively. The maximum force applied to the specimens in this group during the pull-out test was determined as 36.33 kips. In the pull-out test the measured transfer length and development length for the strands of specimens were 41.15 in. and 44.03 in., respectively. The experiments on the specimens in this group were terminated when the 0.5 in. displacement determined at the live ends of prestressing strands [1].

4.3 Self-Compacting Concrete (SCC_2G)

Considering, the DTPT experiment results of the SCC_2G group, the forces applied when 0.01 and 0.1 in. of displacement determined at the dead ends of the strands are 29.50 and 32.20 kips, respectively. The maximum force during the pull-out test applied to the strands of specimens in this group was determined as 32.20 kips. In this group the transfer length and development length in the strands to which the pull-out test were measured as 37.09 and 44.91 in., respectively. Experiments applied to the specimens of the same group were terminated when the LVDT end contacted the concrete surface [1].

5. EVALUATION OF THE TEST RESULTS

The determined transfer length (51.09 in.) during DTPT applied to the HES specimens was 70 and 104% longer than the transfer lengths specified in AASHTO and ACI, respectively. The measured transfer length (41.15 in) applied to the specimens using SCC_1G group was 47 and, 77% longer than the transfer lengths specified in AASHTO and ACI respectively. The measured transfer length (37.09 in.) during DTPT applied to the specimens in the SCC_2G group was 24 and 48% longer than the transfer lengths specified in AASHTO and ACI, respectively.

The transfer lengths measured applied to the SCC_1G and SCC_2G groups were 20% and 29% shorter than the transfer lengths of strands of specimens in the HES.

The measured development length was 56.28 in. in the group using HES. This value was 46 and 22% shorter than values specified in AASHTO-LRFD (104 in.) and ACI318-08 (71 in.), respectively. The determined development length was 44.03 in. in the group using SCC_1G for the DTPT. This value was 58 and 39% shorter than values specified in AASHTO-LRFD (104 in.) [3] and ACI318-08 (71 in.) [4], respectively. The measured development length was 44.91 in. in the group using SCC_2G for the DTPT. This value was 57 and 37% shorter than values specified in AASHTO-LRFD (104 in.) and ACI318-08 (71 in.), respectively.

The development lengths measured in the SCC_1G and SCC_2G groups were 22% and 20% shorter than the development length of specimens using HES, respectively.

The force applied to the specimen using SCC_1G when 0.01 in. of displacement measured was 22% more than applied force in the group using HES. The maximum force applied to these specimens was 29% more than the maximum force applied to the HES. The force applied to the specimen using SCC_2G when 0.01 in. of displacement measured was 36% more than applied load in the group using HES. The maximum force applied to these specimens was 15% more than the maximum force applied to the HES.

6. CONCLUSIONS

It was seen that transfer and development lengths in strands of group members using SCC were shorter than the transfer and development lengths determined in the strands of group members using HES.

The maximum forces and applied forces to constitute 0.01 in. displacement at the dead ends of the strands of experiment members using SCC were bigger than the force applied to constitute same amount of displacement at the dead ends of members using HES.

It was seen that transfer lengths determined in the strands of members in the SCC groups were longer than the transfer lengths specified in ACI 318 and AASHTO LRFD. As a result, a new test method recommended for explaining both the bond mechanics of prestressing strand and the bond characteristics of SCC. This test method combines the cost effectiveness and simplicity of pull-out block test and the flexural beam test.

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APPLICATION OF GEO-SYNTHETICS IN CONTROLLING LANDFILL LEACHATE

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***Abstract*—Preventing penetration of leachate in subterranean layers using impermeable layer and drainage system is the key discussion of engineering-hygienic landfill design. Regarding environmental problems caused by pollution of landfill leachate, in this paper special types of geo-synthetic products were studied in order to control the leachate and prevent secondary leachate. Proper criteria for accurate selection of these coatings in engineering-hygienic landfills are represented by comparing these products with common methods of producing different types of impermeable coats.**

Keywords: engineering-hygienic landfill, geo-synthetics, primary and secondary leachate, leachate drainage system, impermeable coating.

1. INTRODUCTION

From biological viewpoint, an engineering-hygienic landfill is a biological-chemical reactor. Due to existence of different climates in Iran, scattering of population in northern part of the country, and high permeable soil of these areas and high level of underground waters of big cities, it's necessary to lay wastes in an accurate and well-designed hygienic-engineering landfill. This issue has significant role on preventing pollution of vital water and soil resources. It also prevents dangerous and stinky gases to enter atmosphere and guarantees health and hygiene of society of humans and other living beings. From engineering viewpoint, a hygienic-engineering landfill is a geotechnical structure which considers other engineering issues such as static and dynamic resistance, settlement, deformity, etc. along with bio-environmental issues. On the other hand, geo-synthetic covers (polymeric prepared covers manufactured in factories), offer new options to have engineering control on landfills. Different textiles have considerable advantages such as light weight, easy transportation, uniformity of product, easy and quick installation and performance, tensile strength and resistance against UV ray corrosion and chemical attacks. Today, different types of textiles are used in engineering projects of Iran. In this paper we discuss advantages of using different types of geo-synthetic material in engineering-hygienic landfills to improve their bio-environmental and geo-technical condition. Then, structure of engineering-hygienic landfills will be introduced. By introducing the covers, we will discuss application of these layers in preventing distribution of pollutions caused by entering of landfill leachates to ground and underground waters.

2. URBAN SOLID WASTES

The first step in designing control system of engineering-hygienic landfill leachates is to be aware of quality and quantity of solid wastes; it plays an important role on evaluating design and selection of suitable methods in different levels of managing solid wastes. From physical, chemical and biological aspects, determining quality and composition of wastes are important and has important role on predicting the amount of leachate. From the first day of filling to decades later, features of leachate are under change, this issue makes selection of investigation methods for controlling landfill leachate more significant.
The most considerable physical parameters of solid wastes include:

- Weight percent of components and composites
- Garbage percent moisture
- Garbage density

Urban solid wastes of Iran are mainly consisting of organic material, paper, carton, plastics, glasses, steels, bones, textiles, wood and building wastes. Table 1 compares wastes of Tehran and some regions of foreign countries. It's considered that the amount of organic material in wastes of Tehran is higher than foreign countries, which is one of the main causes of producing high amount of leachate in wastes of Iran. Increasing organic material of wastes not only will increase volume of leachate, but destroys chemical materials by changing different reactions.

3. LEACHATE

Leachate is the water of garbage passes through liquids of wastes and garbage to areas around the place that garbage is collected. It is caused by the pressure upon garbage or the weight of garbage itself. Leachate is produced due to polluted urban garbage and contains various solutes and suspended materials, and many biological and chemical materials. Leachate doesn't have stable feature, and is changeable in different masses of garbage. Passing of time will change some features of leachate. Its features depends on factors such as type and sort of wastes, temperature of environment, the way of garbage disposal, amount of rain in the region, etc. In general, leachate is divided as young and old. Although most of the time this division is not recognizable, Table 2 compares amount of different components in young and old leachate.

3.1 Effective factors on features of leachate

Effective factors on composites of leachate are divided to four groups:

a) Composition of wastes: compared to other garbage, existence of different types of leachate in urban garbage is very high. Due to these differences shown in Table 1, quality of leachate is very different. The effect of these differences is more severe in corruptible garbage than incorruptible garbage.

b) Age of garbage: passing of time will change density of garbage. This change is in a way that leachate produced in the first year has lower density than future, so passing of long time will change composites of leachate in a way to decrease its density. This is illustrated in Fig. 1.

c) Temperature of environment: temperature of environment of landfill has direct effect on features of leachate by effecting growth of bacteria and chemical reactivity. Below zero temperature will make some parts of garbage solid and prevents some of chemical reactions.

d) The amount of oxygen: the effect of oxygen on reactivity of corruptive garbage is very important. Since aerobic chemical analysis is different from non-aerobic chemical analysis, creating non-aerobic situation especially in corruptive urban garbage is helpful in controlling features of landfill leachate. Non-aerobic situation can be created by performing alternative covering (daily or weekly) on landfill garbage. When landfill is filled, oxygen rate will decrease. In contrast, thick landfill of garbage without appropriate covers can create suitable situation for aerobic reactions. Later in this article, suitable textiles of these covers will be introduced.

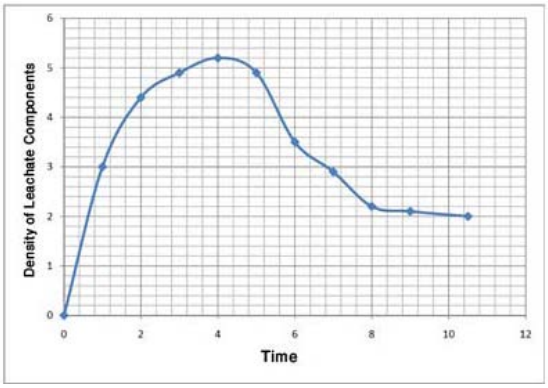

Fig. 1 Leachate density variations by passage of the time (year).

3.2 Factors Affecting the Amount of Leachate

The amount of rains, permeability of underground waters and the amount of primary moisture of garbage are the most important factors affecting the amount of produced landfill leachate. The amount of snow and rain and geographical area of landfill has also direct effect on the amount of leachate. Sometimes substrate surface of landfill is lower than level of underground waters, in this way permeability of underground waters in landfill will increase the amount of leachate. Various soil covers are used in traditional landfills to control these factors. Therefore, final design and cover of landfill in engineering-hygienic landfills considered these issues. This cover prevents production of secondary leachate that is produced after filling of landfill caused by permeability of surface water to landfill.

3.3 Bioenvironmental Dangers of Permeation of Leachate in Soil

The most important detrimental effect of leachate depletion in environment is pollution of underground waters. These waters not only are important for continuance of life, but plays important part in economical and social development of societies, the pollution of which will cause different diseases that are the main concern of developing countries. By the pressure of landfill layers, leachates of garbage can easily penetrate on soil and water resources, unless it produces impermeability cover in the bed of landfill. One of the main applications of textiles in engineering landfill is creation of this impermeable layer. This cover should be with other layer having drainage system to prevent amassingleachate in high landfills. Hence, using compound geo-synthetic covers is very important. Later in this paper, these covers will be described in detail. Fig. 2 illustrates different types of common covers in an engineering-hygienic landfill.

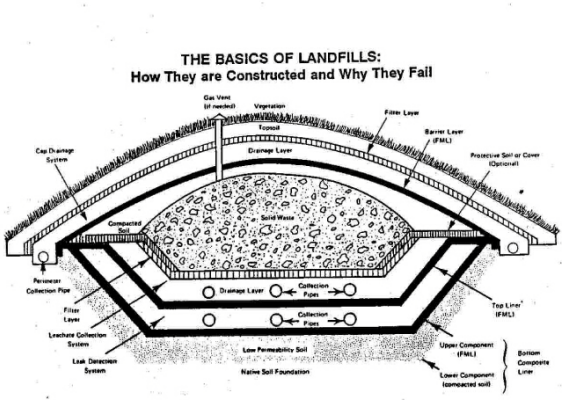

Fig. 2 Engineering-hygienic covers of an urban garbage landfill.

4. GEO-SYNTHETIC COVERS AND ADVANTAGES

4.1 Geo-membranes

Geo-membranes are thin polymer materials with no absolute impermeability, but are impermeable comparing other soils such as clays. Tests on steam passing show that permeability coefficient of these layers is in the range of 0.5×10^{-12} – 0.5×10^{-13} m/s, which is very lower than impermeable covers of clays. Other unique advantage of these layers is that when used as the final cover of landfill in dry regions, they will not crack due to severe sun shine. In final covers, disposal of cracking dry layers of clay is of high importance. Asymmetric settlement of garbage will cause tensile cracks on clay layers, as it's shown in Fig. 3.

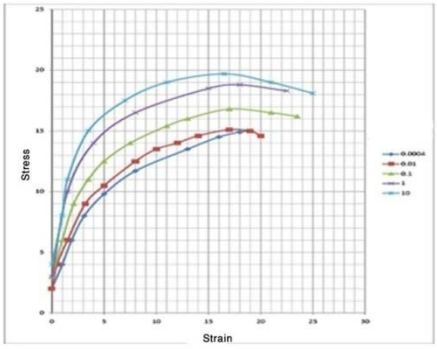

Fig. 3 The variations of HDPE Geo-membranes with thickness of 0.1mm under the experiments of Strain in temperature of 21°c.

Furthermore, these layers are made of polymer and never erosion in wind or runoffs. Due to high concentration of Carbon Black in polymer formula, these covers are able to catch solar energy and limit and decrease UV. Antioxidants are also added to these materials to prevent harmful chemical reactions and produce a minimum amount of oxidation.

4.1.1 Selection of geo-membranes

Various commercial geo-membranes have wide range of useful applications such as high density polyethylene, very low density polyethylene, lining low density polyethylene, PVC. Appropriate alternatives can be used for these covers in workshop. When geo-textile is added to pitch, a very low permeable cover will be produced. In fact, these covers don't have fixed technical features and can concern developing factorial geo-membranes. Table 3 illustrates the most important factors on selecting geo-membranes of every layer of landfills.

A geo-membrane should have ability to resist increasing of forces during installation process and sustainable operation. In some of the applications, it should resist great restrains without rupture. Therefore, preparing a comprehensive evaluation of geo-membrane reactions in long time is needed to guarantee protective features of geo-membrane covers.

In this section, physical reactions of HDPE geo-membranes are explained. Installation process can create interior tensile stress in geo-membranes. As an example, increasing tensile stress of geo-membranes can occur due to the following reasons:

- Installing geo-membranes in a temperature higher than its operation temperature
- Increasing pressure due to wind
- Relocating covering material of geo-membranes on slope. By putting covering layer materials, this tension can be limited toward upward.

Table 3. The Most Important Factors in Selecting Geo-Membranes.

Selecting geo-membranes	The relative importance for layer		
	First cover	Mid cover	Final cover
Insulation against leachate	5	4	1
Resistance against UV	3	2	4
Resistance against Oxidation	5	5	4
Flexibility	4	4	5
Twoaxialstress and strain	4	3	5
Punch resistance	1	1	1
Strain of punch	5	5	5
Friction between thesurface	4	3	5
Resistance against crack stresses	5	5	5
Not loosing additives	5	5	4
Joint making capability	5	5	3
Number of joints in site	3	2	2
State of joint making	4	3	3

Tension can also be intensified due to tensions and deformities caused by materials placed on geo-membranes. Absolutely, this includes tensile stress on lateral slopes and inhibited points upon the slopes. Not only tensile stress of geo-membranes, but strength ofmaterials on geo-membrane should be considered. Asymmetric settlement of geo-membranes can create tension. Settlements also can be caused due to locale deformities caused by wastes of sands or wrinkle of geo-membranes. Tension may occur when geo-membranes cover a hole on sub-base layer.

4.2 Synthetic clay layers (GCLs)

GCLs are thin layers (5-10mm thickness) including a layer of bentonite made of a layer of geo-textile or plastic plate attached by punch, chemical adhesives or sewing. GCLs are usually placed on a prepared or crushed sub-base. Manufactured in factories, they can achieve high level of quality and assurance in control tests. In fact, the only way of sedimentation of these thin layers is overlapping. During production process, it is important to prevent their cracking or rupture.

4.2.1 Different types of GCLs

In a special type, bentonite is placed between two layers of geo-textiles connected to each other by sewing or needle punching or using glue. In needle punch process, many needle punches are infixed in an integrated geo-textile placed on a fiber and extended to a betonite and reach the lowest geo-textile and sew all of them together. Fiber layer punched on upper geo-textile increases its rigidness and interior friction angle. Some of the time, after heating lowest geo-textile, fiber layer of upper geo-textile is melted and create a strong adherence between two textiles and betonite layer. Cover geo-textile is usually punched not sew, while bearing geo-textile can also be sewed (depending on type of GCLs). These covers are more applicable in primary and final covers. They are rarely used in daily layers of a landfill.

4.2.2 Technical characteristics of GCLs

Two important parameters of permeability (K) and diffusivity (D) in GCL are directly related to porosity of GCL (e_B) which depends on production method and limiting tension in which hydration (compounding leachate with moisture) will occur. Petrov et al (1997) prove that final porosity of GCL cover is different in needle punch and other types of non-needle punch GCLs, this is indicated in 2nd and 3rd rows of Table 4. As it's shown in Table 4, needle punch has significant effect on inflation of a special GCL. Due to needle punch fibers, heating during production process has considerable effect on final porosity. For instance, row 3 and 4 of Table 4 compares final porosity of GCL on two different types of GCL, both using needle punch, one of which is heated and the other one is not heated during production process.

Table 4. The Amount of Final Porosity of GCL Cover with Production Conditions.

Confining stress (kPa) (1)	Final bulk void ratio (CB)		
		Needle - punched	
	No needle – punching[1] (2)	No thermal treatment[1] (3)	Thermal treatment[2] (4)
6	7.58	5.12	3.98
25	4.04	3.25	2.97
100	2.58	2.26	2.25
200	1.96	1.68	1.69
400	1.50	1.24	1.19

1Petrov et al. (1997b)
2 Lake and Rowe (2000a)

It's proved that relation between limiting tension and maximum porosity of different types of GCLs depend on production method and types of GCLs. Relation between maximum porosity and enclosing tension of GCLs are different and there is different hydraulic condition for every GCL. Table 5 shows characteristics of three different GCLs.

Following formulas are found as a result of studying these data to describe inflation condition and hydration due to leakage.

$$\sigma'_v \cong 3.25 - 0.57e_B \quad \text{for the NWNWT GCL}$$

$$\log_{10}\sigma'_v \cong 3.5 - 0.69e_B \quad \text{for the WNWT GCL}$$

$$\log_{10}\sigma'_v \cong 2.9 - 0.32e_B \quad \text{for the WNWBT GCL}$$

Sustainability of landfill covers system should be check before, during and after placing garbage. There are different ruptures in geo-membrane covers of landfills. Since Sodium Montmorillonite has lower shear strength in GCLs, potential of having weak layers in lining or covering system should be checked. Not all of GCLs have the same shear strength. Production process and techniques used in its production such as sewing, using needle punch or heat and melted fiber, can increase interior shear strength of GCLs. Following parameters should be considered when studying lining and covering system of GCLs:

- Interior rupture of GCLs in bentonite or geo-synthetics of GCL
- Rupture in one of the common contact surfaces of GCL

Analyzing these rupture potentials need advanced geotechnical laboratories, sufficient data and analyzing stability. Researchers have done studies on interior shear tension of reinforced and non-reinforced GCLs. Table 6 show the result of some of these studies. It indicates that non-reinforced GCLs have very low shear strength during hydration process. Using sewing or needle punch method, reinforced GCLs have higher shear strength than non-reinforced GCLs. In vitro rupture changes depend on the type of GCL and test method.

Table 5. Properties of Three Types of GCLs with Thermal Production Conditions.

Symbol	Sodium bentonite layer	Polypropylene carrier geotextile	Polypropylene cover geotextile	Typical geotextile mass/area MGEO (g/m^2)	Minimum GCL mass/area MGCL (g/m^2)	Mean, std dev of MGCL (g/m^2)
NWNWT	Granular	Nonwoven, woven reinforced	Nonwoven	620	5,270	5,898,312
WNWT	Granular	Woven	Nonwoven	390	5,665	5,795,113
WNWBT	Powder	Silt – film, woven	Nonwoven	500	5,481	5,578,85

*Include 500(g/m^2) of powder bentonite in cover geo-textile

Table 6. Maximum Interior Shear Strength Parameters (Adherenceand Angle ofInternal Friction).

GCL product	Lower geotextile	Reinforcement	C (kPa)	Ø (deg)
GCL - 1	PP slit – film woven	None	2.4	10.2
GCL – 2	PP slit – film woven with a 0.1mm PE geomembrane laminate	Stitch – bonded, 102mm on centre	71.6	4.3
GCL - 3	PP needle – punched nonwoven	Needle - punched throughout	98.2	32.6

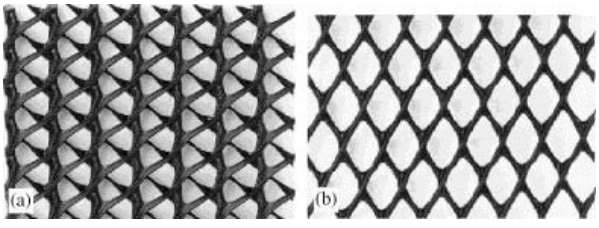

Fig. 4 (a) Three Dimension and (b) Two Dimension.

4.3 Different Types of Geo-textiles

There are two different drainage geo-nets including 2 and 3 dimensional geo-textiles. 2 dimensional geo-textiles have two nets, filaments of which are cross connected to each other, while 3 dimensional geo-nets have vertical dimension offering a wide path for passing of leachate and prevents entering of other material to this path.

5. CONCLUSION

Regarding bio environmental problems caused by distribution of garbage leachate pollution, in this paper special types of geo-synthetic products are studied to control and lead leachate and preventing production of secondary leachates. Comparing these materials with common methods used in production of various impermeable covers, appropriate criteria are offered to accurately select these covers in engineering-hygienic landfills. All in all, using these materials instead of clay can increase assurance coefficient of designing and performing engineering-hygienic landfills, and in long time guarantee efficiency of landfill components.

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EFFECT OF ELEVATED TEMPERATURE ON BEHAVIOR OF COHESIVE SOILS

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***Abstract*—This study investigated the effect of heat treatment on compacted cylindrical soil samples, prepared from industrial kaolin. Dramatic increases in strength with increasing treatment temperature were seen. The effect of dry density on strength above 600 ^{o}C was double the effect of dry density below 600 ^{o}C for the soil tested. Variations in other properties of clay properties such as specific gravity, liquid limit, plastic limit, porosity, void ratio and volumetric and weight changes were also studied. As the treatment temperature increased, specific gravity of the samples initially dropped faster and kept decreasing slowly until the treatment temperatures of 600 ^{o}C, beyond which it showed gradual increase with increasing treatment temperature.**

Keywords: Heat treatment; characterization methods; soil improvement; clay strength; thermal treatment.

1. INTRODUCTION

Elevating the temperature of a soil influences its properties, varying from temporary to permanent, depending on the treatment temperature [1-2]. Thermal treatment has been applied to clayey soils for the purpose of stabilization and for making bricks since the ancient times [3]. Since the development of geotechnical engineering, thermal modification of soils in-situ has been implemented on a few occasions [4-8].

Historically, the use of heat treatment has been limited due to high operating costs [3]. Recent developments have resulted in fast innovations in all aspects of engineering. As a result, thermal modification of soils as a soil improvement technique is being reconsidered [3, 9, 10].

Studies by Tan et al. [9] and Abu-Zreig et al. [10] investigated the changes in some of the geotechnical properties of clayey soils upon heating at temperatures up to 400 ^{o}C and 1000 ^{o}C, respectively. In both studies, heat was applied to powdered clay particles, which does not accurately represent in-situ effect of the heat treatment because neighboring clay particles in the field are closely compacted in the soil matrix.

Joshi et al. [2] investigated the effect of heating on compressive strength of highly plastic clays for temperatures between 300 ^{o}C and 700 ^{o}C. However, they neither studied the effect of heating on other properties of treated soils for the temperature range of their consideration nor did

they study the effect thermal treatment beyond 700 ^{o}C. No report of a study considering the effect of heat on silty soils was reported in the literature.

The aim of this study is to contribute to a better understanding of the thermal modification of cohesie soils. The effect of heat treatment on initially compacted cylindrical soil samples for the temperatures, ranging from room temperature to 1.050 ^{o}C, was investigated. Comparisons of unconfined compressive strength values on the pre- and post treated samples were primarily investigated.

The effects of heating of the samples on the strength, volume-weight relationships, dry density, specific gravity and Atterberg limits are presented.

2. EXPERIMENTAL PROCEDURE

The strength of a soil is characterized by the Mohr-Coulomb criteria, consisting of apparent cohesion and internal friction angle, and it depends on many factors, including the dry unit weight and heterogeneity.

In order to study the effects of the treatment temperature without a bias, it is important to keep all governing factors the same, but the temperature. For this reason, this study has been conducted with homogeneous soil "processed kaolin" obtained from Eczacıbası Esan of Istanbul, Turkey. Although the soil is called kaolin, it turned out as low plasticity silt (ML), according to the Unified Soil Classification System.

Prior to sample preparation, routine properties of the soil such as grain size distribution, liquid and plastic limits, specific gravity and compaction characteristics were determined according to relevant ASTM standards as summarized in Table 1.

Porosity and void ratio of all pre- and post-treated samples were calculated through measuring the sample dimensions and obtaining the solid volume from using the experimentally determined specific gravity.

Table 1. Summary of Initial Properties of Samples.

Specific Gravity (G)	2.59
Liquid Limit (LL)	41
Plastic Limit (PL)	31
Plasticity Index (PI)	10
Maximum dry unit weight (γdmax): kN/m3	14.3
Maximum dry density (□dmax) : g/cm3	1.46
Optimum water content of compaction (%)	18.5
Silt fraction, SF (75□m– 2 □m) ,(%)	55
Clay fraction, CF (75□m– 2 □m), (%)	45
Unified Soil Classification Symbol	ML
Activity , A=PI/CF	0.22

Soil samples were prepared at dry density and optimum water content levels found from the standard compaction test using a Harvard miniature compaction device. The initial cylindrical samples had a diameter of 33.25 mm and a height of 71.50 mm, corresponding to an initial volume of 62.08 cm^3. After weighing the samples, they were set aside overnight to become air-dry. Then, the samples were transferred into an oven where they were kept at 105 ^{0}C for 24 hours in order to completely dry them. Finally, oven-dried cylindrical samples were transferred into an electrical furnace where they were kept another 24 hours at elevated temperatures of 200, 400, 600, 800 and 1050 ^{0}C. Samples were cooled keeping inside the furnace with the door open until they are cool enough to handle, which took about an hour or less. In order to realistically observe the effects of heat treatment, five samples for each treatment temperature and five for oven-dried samples were used. Oven-dried samples adopted to serve as the reference values for comparison of the heat treated samples.

3. RESULTS AND COMPARISONS

Properties of soil samples showed significant changes upon heating. The underlying mechanism for the changes is the increasing loss of adsorbed water on clay particles up to a specific temperature value, i.e., 600 ^{o}C for the samples tested, and commencement of partial sintering upon heating. Variations on soil properties upon heating are presented below

.

3.1 Variations in Volume, Weight and Dry Density

Values obtained for each treatment temperatures are divided by the values of oven dried samples in order to show the relative changes. As seen in Fig. 1, while volume of samples has gradually decreased with increasing temperature, weight of samples has dropped more rapidly at the treatment temperature of 600 ^{o}C.

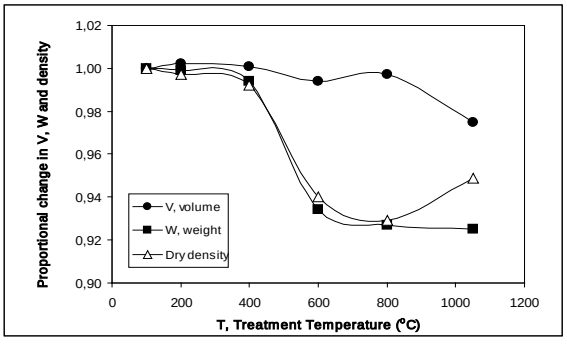

Fig. 1 Variation of volume, weight and dry density with temperature.

Comparison to Joshi et al.'s [2] study, shows the dehydroxylation temperature is about the same

(e.i., less than 600 ^{o}C), but the amount of dehydroxylation as evident from percent weight loss is different (7% of this study versus 14%). It should be noted that Joshi et al. [2] determined the dehydroxylation through thermograms while this study used the initial and treated sample weights. Furthermore treatment time (24 hours here versus 2 hours), origin for kaolin used and sample cooling (fast here versus slow) were different, and may have influenced the percent weight loss of samples. The issue remains to be further studied.

3.2 Atterberg Limits and Specific Gravity

Atterberg limits consist of liquid limit, plastic limit and plasticity index. All values gradually decreased and the soil has finally reached to the non-plastic state at 600 ^{o}C, which is in agreement with findings of Tan et al. [9] and Abu-Zreig et al. [10].

This agreement means that heating permanently removes soil plasticity at a specific temperature whether the clay is in compacted state or in powdered particle state. This specific temperature agrees with the dehydroxylation temperature.

The influence of the heat treatment on the specific gravity has been found to be a function of the treatment temperature (Fig. 2).

Values sharply decrease until the treatment temperature of 600 ^{o}C, after which a gradual increase was observed. Behavior beyond 600 ^{o}C has been reported to be constant by Abu-Zreig et al. [10], in which heat was also applied on powdered clay particles. The difference reported here is due to commencement of permanent particle bonding among neighboring clay particles at 600 ^{o}C.

3.3 Porosity and Void Ratio

Effect of heat treatment on the porosity and void ratio of the samples are presented in Fig. 3. Both factors show the same behavior; almost no change up to the treatment temperature of 400 ^{o}C is seen. From 400 ^{o}C until 600 ^{o}C, sharp increases of 10% for porosity and 17% for void ratio with respect to the corresponding values of oven-dried state are seen. The increase in the porosity and the void ratio continue to gradually increase until 800 ^{o}C and afterwards a sharp decline is observed.

3.4 Compaction Behaviour

Effect of heat treatment on the compaction curve of the samples with treatment temperatures under dehydroxylation temperature (e.i., natural, at 200 ^{o}C and 400 ^{o}C) is presented in Fig. 4. Samples with treatment temperatures at and higher than dehydroxylation temperature were excluded as particle bonding forms. As seen in the figure, optimum water content and the maximum dry unit weight tend to increase with the treatment temperature. The effect is much pronounced for the maximum dry unit weight at 400 ^{o}C treatment temperature.

Compaction behavior at and above the dehydroxylation temperature would depend on the particle size and shape of the crushed pieces of the treated material as particle bonding generates. This issue remains to be investigated. The maximum dry unit weight and the optimum water content values are tabulated in Table 2.

3.5 Unconfined Compressive Strength

Unconfined compressive strength of clay samples showed dramatic increase with increasing treatment temperature. Samples tested had slight changes in their dry density values despite the efforts made during sample preparation. Because the dry density is an important factor in influencing the soil strength, test results are shown as a function of treatment temperature and dry density. As seen in Fig. 5, the unconfined strength of soil increases both with increasing density and treatment temperature. While the increase in the strength is gradual for temperatures up to 400 ^{o}C, it suddenly rises at the treatment temperature of 600 ^{o}C. The strength continued to increase beyond the treatment temperature of 600 ^{o}C until the maximum treatment temperature of this study. It is also quite notable that the effect of dry density on strength above 600 ^{o}C is double the effect below 600 ^{o}C. This is most likely because of establishment of permanent bonds between neighboring clay particles (increasing partial sintering), starting around this treatment temperature. Unconfined strength of air-dried and oven-dried samples increased an average of 88% and 210% respectively in comparison to strength of wet samples. This means that the strength of the compacted wet sample triple when all of the gravity controlled water is removed from the soil matrix.

Strength gain continues to rise as the treatment temperature increases in the order of about 35 kPa, 7% of wet strength, 3.8% of air dried strength, or 2.3% of completely dry strength for each 10 ^{o}C increase in treatment temperature beyond 100 ^{o}C.

The findings of this study on strength of heat treated clays are significantly different from those of Abu-Zreig et al. [10], in which the unconfined compression strength was reported to drop to zero after the treatment temperature of 400 ^{o}C. This difference is because of the fact that they applied the heat on powdered clay particles and the strength was attempted to be determined on cylindrical samples afterwards. Because around 400 ^{o}C, water retention mechanism on the surface of clay particles has permanently changed, the soil has become non-plastic much like very finely grinded sand particles.

As a result, unconfined strength becomes zero since all apparent cohesion comes from plasticity was lost. However, plasticity of soil is not the only source for apparent cohesion. As shown in this

study, keeping the clay particles close to each other in a compacted state during the heat treatment allows generation of new bonds between clay particles, which in turn generates a new source for apparent cohesion. Unconfined strengths reported for kaolin by Joshi et al. [2] were around 10-15 MPa, were significantly higher than 3-5 MPa of this study for temperatures around 600-800 ^{o}C. Differences in treatment time (24 hours here versus 2 hours), origin for kaolin used and sample cooling (fast here versus slow) may have affected the results. However, the issue of maximum possible strength for a given temperature and its conditions remains to be further studied.

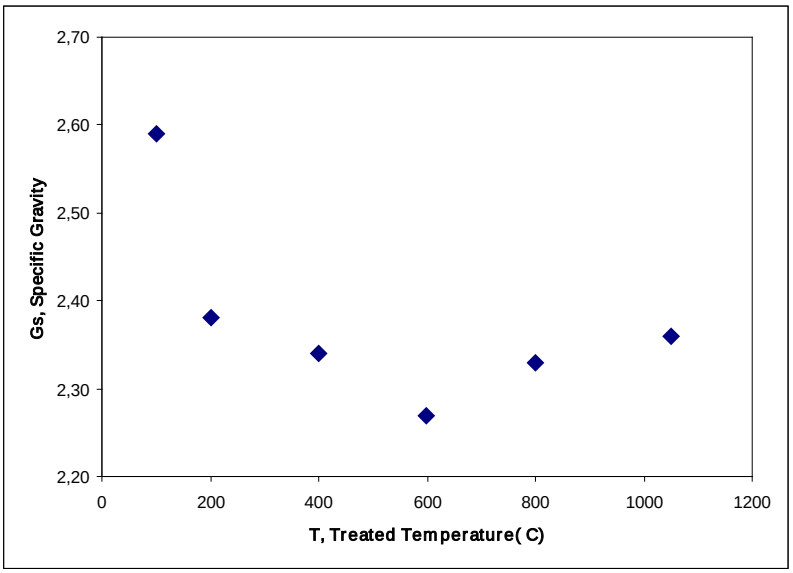

Fig. 2 The effect of heat treatment on specific gravity.

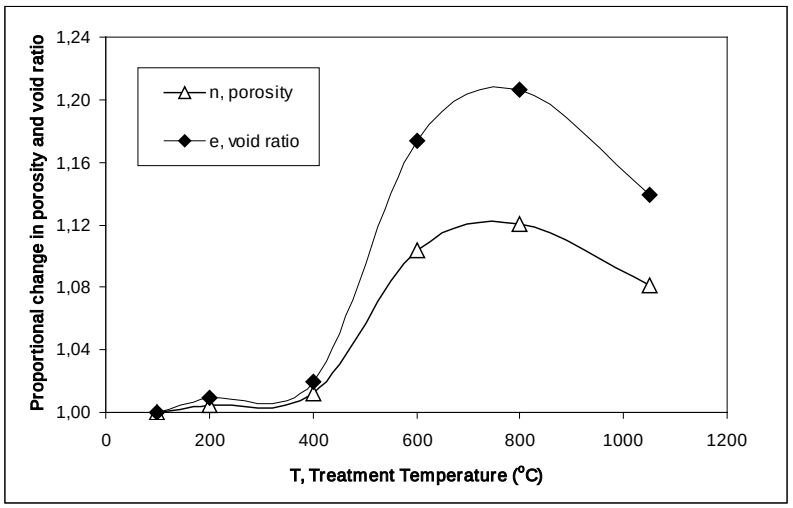

Fig. 3 The effect of heat treatment on porosity and void ratio.

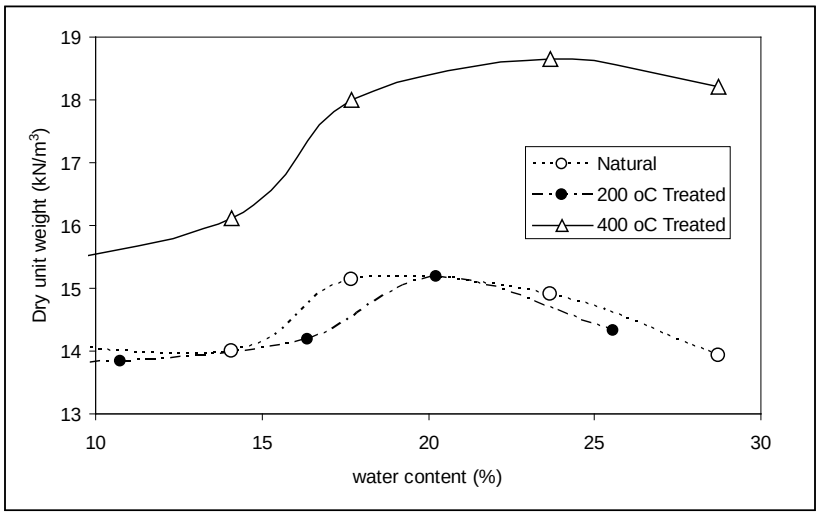

Fig. 4 The effect of heat treatment on compaction behaviour.

Table 2. Maximum compaction and optimum moisture content of treated soil.

Treatment Temperature	natural	200 °C	400 °C
Optimum moisture content (%)	17.7	20.2	23.7
Maximum dry unit weight (kN/m^3)	15.15	15.19	18.66

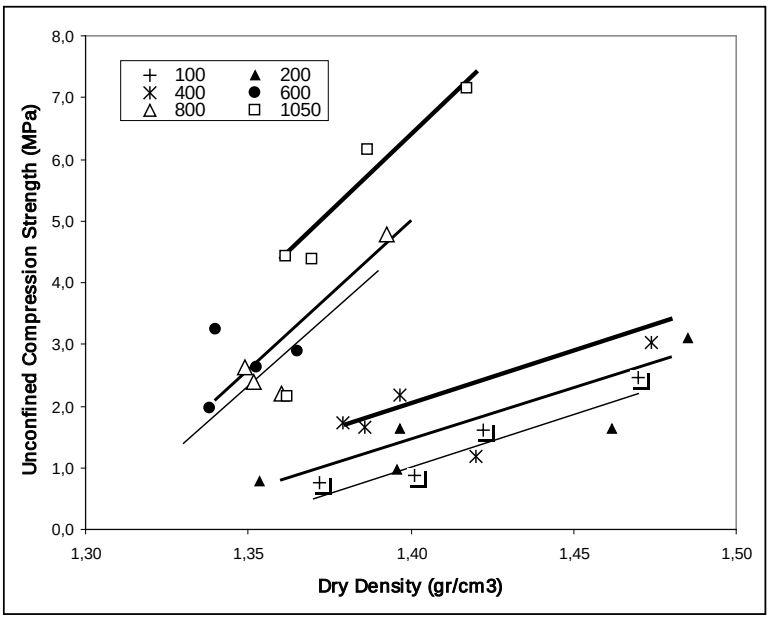

Fig. 5 Effect of dry density and temperature on soil strength.

4. CONCLUSIONS

Volume and weight of clay samples tested are found to decrease with increasing treatment temperature. Sharp increases in porosity and void ratio are seen from 400 ^{o}C to 600 ^{o}C. The increases in both properties continue to gradually rise until 800 ^{o}C, and afterwards a sharp decline is observed. The influence of the heat treatment on the specific gravity is found to be a function of the treatment temperature as well. Values sharply decrease until the treatment temperature of 600 ^{o}C, after which a gradual increase is observed.

Based on the this study and the previous work in the literature it can be concluded that heating removes soil plasticity permanently at a specific temperature whether the soil is in compacted state or in powdered particle state.

Optimum water content and the maximum dry unit weight tend to increase with the treatment temperature. The effect is much pronounced for the maximum dry unit weight as the treatment temperature approaches to the dehydroxylation temperature. Effect of heating on compaction characteristics at and beyond the dehydroxylation temperature remains to be investigated.

Unconfined compressive strengths of samples show dramatic increase with increasing treatment temperature. The increase in strength is gradual for temperatures up to 400 ^{o}C, but it suddenly rises between treatment temperatures of 400 ^{o}C and 600 ^{o}C. The average strength gain is about 35 kPa, 7% of wet strength, 3.8% of air dried strength, or 2.3% of completely dry strength for each 10 ^{o}C increase in treatment temperature beyond 100 ^{o}C. The effect of dry density on strength above 600 ^{o}C is double the effect below 600 ^{o}C.

The underlying mechanism observed on the heat treated soil samples is believed to be the increased loss of adsorbed water on clay particles (dehyroxylation) up to a specific temperature value, and establishment of permanent bonds (partial sintering) between neighbouring clay particles. However, further studies are needed to determine the exact cause.

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OPTIMIZATION OF GRAVITY RETAINING WALLS IN COHESIONLESS SOILS

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***Abstract*—Optimization of a gravity retaining wall is examined in this study. An optimization problem that contains continuous functions is developed. The functions, object function and constraint function, are derived from external stability and internal stability checks. The checks are overturning, forward sliding, bearing capacity, shear in stem and bending in stem. Gravity retaining walls of 2.0, 3.0, and 4.0 m heights are taken into account in the optimization problem to investigate general geometry of optimum cross section and the effect of wall height on the geometry. In the optimization procedure, applicability was leaved aside and problem was tried to be solved mathematically. As a result of the procedure, common feature of optimum cross sections of different height walls is to have a wide lower part. Bending checks in stem and no separation between wall base and foundation soil are effective in determining optimum cross section geometry of the wall of 3.0 and 4.0 m heights.**

Keywords: Gravity retaining wall, nonlinear optimization, continuous variables, constraints.

1. INTRODUCTION

The structures which are built to ensure the security of other structures and holding back a soil mass are called retaining structures. In other words retaining walls are constructed with the aim of holding soil mass in different levels at same place. The various types of earth retaining structures fall into three broad groups: gravity walls (masonry, gabion, crib, reinforced concrete, counterfort, buttressed, etc.), embedded walls (driven sheet pile, braced or propped, contiguous bored-pile, secant bored-pile, diaphragm, etc), and reinforced and anchored earth (reinforced earth, soil nailing, ground anchors, etc) (Whitlow, 2001). Parameters like existing construction material, wall height, ground water level, soil type, etc. should be evaluated to determine feasible wall type to construct.

Gravity retaining walls resist lateral pressures with their own weights. The walls of this type are uneconomic because the wall material (masonry or mass concrete) is used only for its dead weight (Craig, 1994). Masonry and plain concrete are widely used in construction of gravity retaining walls. In some cases the provision of sand, gravel, and cement are easier and more economic than other wall materials so it is preferable to built concrete retaining wall compared masonry retaining wall.

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While considering horizontal deformation of soil medium, two situations occur. These are state of elastic equilibrium and state of plastic equilibrium. A body is said to be in a state of elastic equilibrium when a small change in stress acting upon it produces a corresponding and reversible change in strain. In a state of plastic equilibrium (active state, passive state) irreversible strain takes place at constant stress; the Mohr-Coulomb failure criterion is one way of representing this state (Whitlow, 2001). In other words, in the state of elastic equilibrium (at-rest state) any lateral deformation does not occur. In this case, lateral pressure of soil is called as earth pressure at rest. The earth pressure is

$$\sigma'_o = K_o \sigma'_d \quad (1)$$

σ'_o : horizontal effective stress
σ'_d : vertical effective stress
K_o : coefficient of soil pressure at rest.

Some K_0 values depending on the soil type are given in Table 1.

Table 1. K_0 values depending on soil type (Whitlow, 2001).

Soil Type	K_0
Loose sand	0.45-0.6
Dense sand	0.3-0.5
NC clay	0.5-0.7
OC clay	1.0-4.0
Compacted clay	0.7-2.0

If a wall moves away from soil, lateral earth pressure acting on the wall will be decreased. If the wall moves far enough away, shear failure of soil will occur. The earth pressure exerted on the wall at this state of failure is known as active earth pressure, and it is at minimum value. In a similar way, if the wall moves far enough toward the soil, shear failure of the soil will occur. The earth pressure exerted on the wall at this state of failure is known as passive earth pressure, and it is at maximum value (Cheng and Jack, 1998).

Rankine's theory offers a solution for lateral earth pressure of a homogeneous and isotropic mass of cohesionless soil in states of plastic equilibrium. The theory is based on several assumptions. The primary one is that there is no adhesion or friction between wall and soil. Sliding surface and ground surface are assumed to be planar. Length of the wall is infinite and the wall moves forward and backward enough to produce active and passive lateral earth pressures. Resultant force acting on the retaining structure acts parallel to the slope of the fill. The equations for computing lateral earth pressures according to Rankine's theory are as follows:

$$\sigma_{a,p} = K_{a,p}\sigma_d \qquad K_{a,p} = \tan^2\left(45 \mp \frac{\phi}{2}\right) \qquad (2)$$

1.1 Design of Gravity Retaining Walls

The design of gravity retaining walls must satisfy external and internal stability. Sliding (on ground), overturning, bearing capacity, overall failures checks are made to show the wall remains fixed in the desired location. Additionally, internal stability checks (shear and bending) are made to demonstrate the wall is able to carry internal stresses without rupturing. Settlement check can also be important and done in some situations.

1.1.1 Optimization

Optimization can be defined by the maximizing or minimizing of a given function possibly subject to some type of constraints. The general formulation of optimum design problems can be defined mathematically as follows:

Minimize $f(\boldsymbol{x})$
subject to $g_i(\boldsymbol{x}) \le 0 \qquad i=1,.....,n \quad (\boldsymbol{x}^l \le \boldsymbol{x} \le \boldsymbol{x}^r)$
(3)

$\boldsymbol{x}$: Vector of design variables
$f(\boldsymbol{x})$: Objective function
$g_i(\boldsymbol{x})$: Set of constraints
$\boldsymbol{x}^l$ *and* $\boldsymbol{x}^r$**:** Vector of lower and upper bounds of design variables respectively.

Typical design variables, objective functions, and constraints used in structural optimization problems can be listed as follows;

i. Design variables : Coordinates of key points, Thickness and/or width at key points, Cross-sectional area of member, Etc.
ii. Objective functions : Weight, Volume, Area, Etc.
iii. Constraint functions **:** Specifications, Checks, Etc (Ozakca et al., 2010).

Different optimization techniques are improved to satisfy the requirement of different type of optimization problems. Major categories of optimization methods are classical optimization techniques, linear programming, nonlinear programming, geometric programming, dynamic programming, integer programming, stochastic programming, evolutionary algorithms, etc. If any of the functions among the objectives and constraint functions is nonlinear, the problem is called a nonlinear programming (NLP) problem this is the most general form of a programming problem. Nonlinear problems can be solved by using Search Algorithm Methods (Direct or Indirect) or transforming the nonlinear function (both objective and constraints) into a set of linear functions and then applying methods for linear programming.

Optimum design of retaining walls has been investigated by many researchers. Alshawi et al. (1988) designed a new type of retaining wall that is tied to the base by two parallel inclined ties and optimum position of the ties were explored. Dembicki and Chi (1989) developed a non-linear multiobjective optimization problem and investigated optimum shape of a retaining wall modelled by the co-ordinates of a set of points. Saribas and Erbatur (1996) also developed an multiobjective optimum design problem so that the objective functions are cost and weight of the wall. Basudkar and Lakshman (2006) developed an optimization problem by taking into account settlement. Akin and Saka (2010) used harmony search algorithm to find optimum design of concrete cantilever retaining wall. Salami et al. (2010) used a simplex algorithm dealing with reinforced concrete cantilever retaining wall. The paper is concerned with the solution and

determination of the optimum shape of a concrete gravity retaining wall modeled by the co-ordinates of a set of points.

2. OPTIMIZATION PROCEDURE

The design of gravity retaining wall requires the selection of cross section properties so that the wall satisfies all checks while using minimum amount of contruction material (e.g. concrete, masonry). This may be accomplished by the utilization of structural shape optimization procedures in which the shape of the wall is varied to achieve a specific objective function satisfying certain constraints. Problems of structural optimization are characterized by various objectives and constraints that are nonlinear or linear functions of the design variables. The design variables of the optimization problem are widths of the gravity retaining wall at some distance from the base (x_i i=1,7) as shown in Fig. 1.

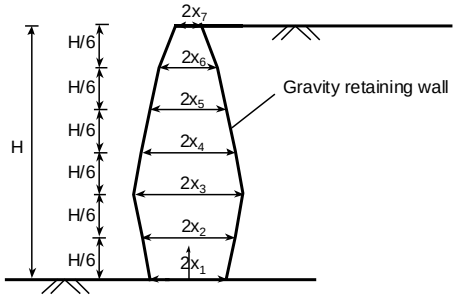

Fig. 1 Design variables of gravity retainin wall.

2.1 Objective Function

In this study, the objective function is the cross section area of the wall so that the cost of the wall is directly related to the area. Therefore, minimization of the area induces minimization of the cost.

$$\text{Min } f(x) = \frac{H}{6}(x_1 + 2x_2 + 2x_3 + 2x_4 + 2x_5 + 2x_6 + x_7) \quad (4)$$

2.2 Constraints

The design of the gravity retaining wall involves some checks about the construction material or materials and in situ conditions to provide enough safety and strength. Therefore, constraints can be classified as geotechnical (external) constraints and structural (internal) constraints. Lateral earth pressures are calculated by Rankine's theory and the effect of soil mass that moved with the wall is neglected. Besides, Lateral earth pressure is acted horizontal to vertical back face of the wall according to Rankine's theory in case of horizontal surface of backfill soil. In other words, back face of the wall is not vertical in this study but the pressure is assumed to be acting horizontally.

2.2.1 Geotechnical (External) Constraints

a) Overturning

Lateral earth pressure is calculated by using Rankine's theory in all constraints and factor of safety against overturning is taken as:

$$g_1(x) = FS_o \cdot \frac{\gamma_n}{6} \cdot H^3 \cdot K_a - \gamma_c \frac{H}{6}\left[(x_1 + x_7) + (2x_2 + 2x_3 + 2x_4 + 2x_5 + 2x_6)\right] \cdot x_1 \le 0 \quad (5)$$

b) Sliding:

Factor of safety against sliding is taken as 1.5 and the friction angle between soil and wall is accepted as ϕ.

$$g_2(\mathbf{x}) = FS_s\left(\frac{1}{2}\gamma_n H^2 K_a\right) - \gamma_c \frac{H}{6}\left[(x_1 + x_7) + (2x_2 + 2x_3 + 2x_4 + 2x_5 + 2x_6)\right] \cdot \tan\phi \le 0 \quad (6)$$

c) No separation between wall base and foundation soil:

$$g_3(x) = \frac{1}{6}\gamma_n H^3 K_a - \frac{H}{6}\gamma_c\left[(x_1 + x_7) + (2x_2 + 2x_3 + 2x_4 + 2x_5 + 2x_6)\right]\frac{2x_1}{6} \le 0 \quad (7)$$

d) Bearing Capacity:

Terzaghi's bearing capacity theory is used to calculate ultimate bearing capacity of foundation soil. Factor of safety for bearing capacity is accepted as 3.

$$g_4(x) = \frac{\gamma_n H^3 K_a}{4x_1^2} + \frac{H\gamma_c\left[(x_1 + x_7) + (2x_2 + 2x_3 + 2x_4 + 2x_5 + 2x_6)\right]}{12x_1} - \frac{\gamma_n x_1 N_\gamma}{FS_b} \le 0 \quad (8)$$

2.2.2 Structural Constraints

Structural constraints are shear and bending in stem at regular intervals from the base of the wall. Shear capacity of the gravity wall is calculated according to TS 500. Constraints from bending are derived from that no tensile stresses occur in the stem of the wall

a) Constraints from shear:

$$g_5(x) = \frac{1}{2}\cdot\gamma_n\cdot\left(\frac{H}{6}\right)^2\cdot K_a - 0.65\cdot f_{ctd}\, 2x_6 \le 0 \quad (9)$$

$$g_6(x) = \frac{1}{2}\cdot\gamma_n\cdot\left(\frac{H}{3}\right)^2\cdot K_a - 0.65\cdot f_{ctd}\cdot 2x_5 \le 0 \quad (10)$$

$$g_7(\mathrm{x}) = \frac{1}{2}\cdot\gamma_n\cdot\left(\frac{H}{2}\right)^2\cdot K_a - 0.65\cdot f_{ctd}\cdot 2x_4 \le 0 \quad (11)$$

$$g_8(\mathrm{x}) = \frac{1}{2}\cdot\gamma_n\cdot\left(\frac{2H}{3}\right)^2\cdot K_a - 0.65\cdot f_{ctd}\cdot 2x_3 \le 0 \quad (12)$$

$$g_9(\mathrm{x}) = \frac{1}{2}\cdot\gamma_n\cdot\left(\frac{5H}{6}\right)^2\cdot K_a - 0.65\cdot f_{ctd}\cdot 2x_2 \le 0 \quad (13)$$

b) Constraints from bending:

$$g_{10}(\mathrm{x}) = \frac{\gamma_n\left(\frac{H}{6}\right)^3 K_a}{4x_6^{\,2}} - \frac{\gamma_c\frac{H}{6}(x_6+x_7)}{2x_6} \le 0 \quad (14)$$

$$g_{11}(\mathrm{x}) = \frac{\gamma_n\left(\frac{H}{3}\right)^3 K_a}{4x_5^{\,2}} - \frac{\gamma_c\frac{H}{6}(x_5+2x_6+x_7)}{2x_5} \le 0 \quad (15)$$

$$g_{12}(\mathrm{x}) = \frac{\gamma_n\left(\frac{H}{2}\right)^3 K_a}{4x_4^{\,2}} - \frac{\gamma_c\frac{H}{6}(x_4+2x_5+2x_6+x_7)}{2x_4} \le 0 \quad (16)$$

$$g_{13}(\mathrm{x}) = \frac{\gamma_n\left(\frac{2H}{3}\right)^3 K_a}{4x_3^{\,2}} - \frac{\gamma_c\frac{H}{6}(x_3+2x_4+2x_5+2x_6+x_7)}{2x_3} \le 0 \quad (17)$$

$$g_{14}(\mathrm{x}) = \frac{\gamma_n\left(\frac{5H}{6}\right)^3 K_a}{4x_2^{\,2}} - \frac{\gamma_c\frac{H}{6}(x_2+2x_3+2x_4+2x_5+2x_6+x_7)}{2x_2} \le 0 \quad (18)$$

c) Constraints from design:

Minimum width at the top of gravity retaining wall for presizing is recommended as least 0.3 m to prevent problems which occur due to the workmanship quality and workability of concrete (Smith and Smith, 1998). Thus, minimum width is limited as 0.30 m at all parts of the wall and as a result following constraints are obtained.

$$\begin{bmatrix} -1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & -1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & -1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & -1 \end{bmatrix} \times \begin{bmatrix} x_1 \\ x_2 \\ x_3 \\ x_4 \\ x_5 \\ x_6 \\ x_7 \end{bmatrix} + \begin{bmatrix} +0 \\ +0 \\ +0 \\ +0 \\ +0 \\ +0 \\ +0 \end{bmatrix} \le \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix} \quad (19)$$

2.3 Optimization with MATLAB

The MATLAB R2008b Programming Environment was used to solve the optimization problem. A source code was developed and then the problem was solved with the help of Optimization Toolbox of The MATLAB R2008b. The nonlinear optimization problem defined in this study was solved by fmincon that is a gradient-based framework with three different algorithms. The algorithms are Trust-Region Reflective, Active Set, and Interior Point. The solutions were obtained by the Active Set algorithm.

3. OPTIMIZATION EXAMPLES

Some variables were prescribed to obtain optimum cross section of gravity retaining walls of different heights. The variables related to the optimization problem are summarized in Table 2. The heights of gravity retaining walls supporting a backfill soil are 2.0 m, 3.0 m and 4.0 m. Gravity retaining walls attain its stability by resisting lateral pressures with its weight. In this study, plain concrete is used as construction material of the wall. This material is composite construction material and composed of aggregate, cement and water and used for its mass without using any strengthening material.

The retaining wall with 2.0 m height is considered as the first example. The factors of safety for overturning, against sliding, and bearing capacity are 3.0000, 2.3994, and 8.6823 respectively. The optimum dimensions obtained from the algorithms for the gravity retaining wall are x_1= 0.5838 m, x_2=0.4819 m, x_3=0.3756 m, x_4=0.2581 m, x_5=0.1500 m, x_6=0.1500 m, and x_7=0.1500 m with cross section area of 1.1883 m^2 which is the objective function value of the optimum design problem. The optimum results reported here corresponds to cross section dimensions for which the wall has minimum area. The first striking feature of the cross section is that wider part occurred at the bottom of the wall (Fig. 8). It is seen from the results of constraints of 2 m height wall given in Table 3 that bending constraints of sections orresponding to x_2, x_3 and, x_4 and no separation between wall base and soil are critical in determining solution of this problem.

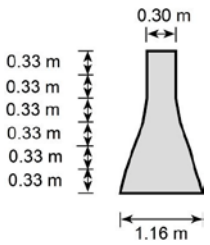

Fig. 2 Optimized cross section of the gravity retaining wall of 2 m height.

The retaining wall with 3.0 m height is considered as the second example. The factors of safety for overturning, against sliding, and bearing capacity are 3.000, 2.3835, and 8.8582,

respectively. The optimum dimensions are x_1=0.8816 m, x_2=0.7330 m, x_3=0.5825 m, x_4=0.4263 m, x_5=0.2484 m, x_6=0.1500 m, and x_7=0.1500 and the objective function takes the value of 2.6559 m^2. Critical constraints of this wall are bendings of sections related to x_2, x_3, x_4, and x_5 and no separation between wall base and soil. The values of corresponding functions are approximately zero (Table 3). It is seen from Fig. 3 that bottom of the wall is very wide.

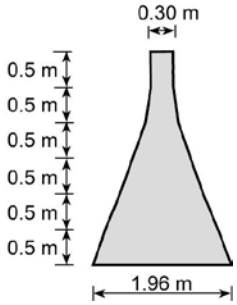

Fig. 3 Optimized cross section of the gravity retaining wall of 3 m height.

Table 2. Input parameters for design examples.

Input Parameter	Unit	Symbol	Value		
			Example 1	Example 2	Example 3
Height of wall	m	H	2.0	3.0	4.0
Internal friction angle of retained soil	degree	ϕ	35	35	35
Internal friction angle of base soil	degree	ϕ_b	35	35	35
Coefficient of friction between the wall base and soil	degree	δ	tan(ϕ)	tan(ϕ)	tan(ϕ)
Unit weight of retained soil	kN/m^3	γ_n	16.0	16.0	16.0
Unit weight of base soil	kN/m^3	γ_{nb}	16.0	16.0	16.0
Concrete design strength in tension	Mpa	f_{ctd}	0.9	0.9	0.9
Concrete design strength in compression	Mpa	f_{cd}	16.0	16.0	16.0
Unit weight of concrete	kN/m^3	γ_c	25.0	25.0	25.0
Factor of safety for overturning stability	-	FS_o	2.0	2.0	2.0
Factor of safety for against sliding	-	FS_s	1.5	1.5	1.5
Factor of safety for bearing capacity Bearing Capacity Factor	- -	FS_b N_γ	3.0 47.3	3.0 47.3	3.0 47.3

The retaining wall with 4.0 m height is considered as the third example. The factors of safety for overturning, against sliding and bearing capacity are 3.000, 2.3796, and 8.9013, respectively. Optimum values of the variables are x_1=1.1773 m, x_2=0.9806 m, x_3=0.7832 m, x_4=0.5839 m, x_5=0.3741 m x_6 =0.1500 m and x_7 =0.1500 m and area of cross section is equal to 4.7141 m^2. Critical constraints for this wall as in foregoing examples are bending constraints corresponding to sections of x_2, x_3, x_4, and x_5 and separation check between wall base and foundation soil. General shape of the cross section is similar to the other walls.

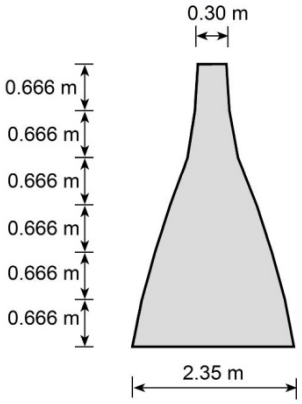

Fig. 4 Optimized cross section of the gravity retaining wall of 4 m height.

Table 3. Results of constraint functions at the optimum solutions (x_i) for the retaining walls.

Constraints	H=2 m	H=3 m	H=4 m
$g_1(x)$	-5.7813	-19.5120	-46.2507
$g_2(x)$	-7.7980	-17.2383	-30.5123
$g_3(x)$	0.0000	0.0000	0.0000
$g_4(x)$	-96.3873	-147.0758	-196.9053
$g_5(x)$	-175.2591	-174.9580	-174.5364
$g_6(x)$	-174.5364	-288.4074	-433.8838
$g_7(x)$	-299.7807	-493.9228	-674.5063
$g_8(x)$	-435.6173	-672.8869	-900.9787
$g_9(x)$	-557.8161	-844.0050	-1123.2000
$g_{10}(x)$	-6.5490	-6.4778	-2.3918
$g_{11}(x)$	-2.3918	0.0000	0.0000
$g_{12}(x)$	0.0000	0.0000	0.0000
$g_{13}(x)$	0.0000	0.0000	0.0000
$g_{14}(x)$	0.0000	0.0000	0.0000

The values of constraint functions at optimum solution points of the nonlinear optimization problem are given in Table 3. It is seen from Table 3 that $g_3(x)$ constraint function which is corresponding to no separation between wall base and soil check takes the value of zero for all gravity walls of different heights. Therefore, it is understood that no separation

between wall base and foundation soil constraint is one of effective constraints that determine optimum cross section. Apart from that, constraints derived from bending in stem are the other effective constraints on optimum cross section. The $g_{12}(x)$, $g_{13}(x)$, $g_{14}(x)$ constraints which are derived from bending corresponding to x_2, x_3, x_4 sections are zero for all walls and the $g_{11}(x)$ constraint is equal to zero for the walls of 3 and 4 m height. Thus, it is realized that bending in stem becomes an efficient constraint with increasing wall height.

Common feature of optimized cross sections of different height walls is to have a wide bottom part. Because, the need of performing separation check between wall base and soil causes the base part to widen. Bending checks in stem is also other effective checks determining cross section geometry. General appearances of optimum cross section are similar to each other (Figs. 8, 9, and 10).

4. CONCLUSIONS

In this study, it was aimed to determine optimum cross section geometry of gravity retaining wall supporting cohesionless soil. The area of cross section of the wall is accepted to be directly related to cost so object function is chosen as cross section area while determining optimum cross section. Additionally, constraints were derived from the checks that a gravity retaining wall should satisfy. Thus, the nonlinear optimization problem defined by object function and constraints was derived. The problem was solved by developing a source code in MATLAB R2008b.

Check of no separation between wall base and foundation soil are effective constraint determining cross section geometry in external stability checks and bending in stem checks are effective constraints in internal stability checks. Common feature of optimized cross sections of walls is to have a very wide lower part and narrow upper part. General appearances of optimum cross sections of the gravity walls of different heights are similar to each other.

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TDR APPLICATIONS IN CIVIL ENGINEERING

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Time Domain Reflectometry (TDR) is a well-established remote sensing technology with a relatively long history beginning with cable testing in 1930s [1]. TDR is based on sending a fast rising electromagnetic (EM) step pulse or impulse into an electrical cable or a transmission line and measuring the reflections due to the change of system geometry or material dielectric permittivity. Transmission lines typically consist of a co-axial cable or two parallel conductors. The transmission line is embedded within a structure or surface-mounted on the structure as a sensor to monitor local changes in the geometry or material properties.

The use of TDR has been expanded to include civil and structural engineering with a wide variety of applications such as measuring soil water content [2], landslide monitoring [3], bridge scour monitoring [4] and detecting-locating cracks in structural elements [5].

1. TDR FOR SOIL WATER CONTENT MEASUREMENTS

Quantification of the soil water content (SWC) plays a key role in unsaturated soil mechanics, and it is essential to know the value of SWC for many geotechnical engineering applications such as early detection of landslide risk and compaction quality control in earthwork and highway projects. Additionally, determination of SWC is also one of the fundamental needs in the soil physics and hydrology as defining the optimal time for irrigation, estimating the infiltration rate and evaluating the potential leakage from a waste site are often needed [6, 7].

Several traditional methods are currently being used for determination of the SWC. Although the oven-drying method (gravimetric sampling) is the most accurate and widely known in geotechnical practice, it is time consuming and destructive [8]. Radioactive methods such as neutron scattering and gamma ray attenuation are widely accepted non-destructive methods but they require special caution and licensing to operate in order to avoid possible health hazards [9]. As an alternative non-destructive method, Time Domain Reflectometry (TDR) became popular for measuring the volumetric soil water content (θ). Accurate, remote, rapid and automated measurement, using simple probes and measuring the dielectric permittivity directly are some of the features associated with TDR [10, 11].

The early developments of TDR for SWC measurement started in 1970s [12] and furthered by Topp et al. [2] with the retrieval of a relationship between the θ and the apparent dielectric permittivity (ε_a) of the soil. TDR measured ε_a represents the real part of the complex dielectric permittivity when the dielectric losses assumed to be negligible in the TDR bandwidth [13].

The TDR method for SWC determination is based on generating a fast rise-time pulsed EM signal (usually in step-wave formation) and coupling it into a probe that acts as a waveguide which is embedded in the soil under investigation. The incident and the reflected signals are recorded with a TDR system in order to perform a travel time analysis (Fig. 1).

The round-trip travel time (Δt) of the EM signal along the probe of length (L) depends on the ε_a of the soil surrounding the metallic conductors of the waveguide, as

$$\varepsilon_a = \left(\frac{c\,\Delta t}{2L} \right)^2 \quad (1)$$

where c is the velocity of light ($2.998.10^8$ ms^{-1}).

Topp et al. [2] developed probably the most cited relationship between the ε_a and the θ [14].

$$\theta = -0.053 + 0.0292\,\varepsilon_a - 5.5\times10^{-4}\,\varepsilon_a^2 + 4.3\times10^{-6}\,\varepsilon_a^3 \quad (2)$$

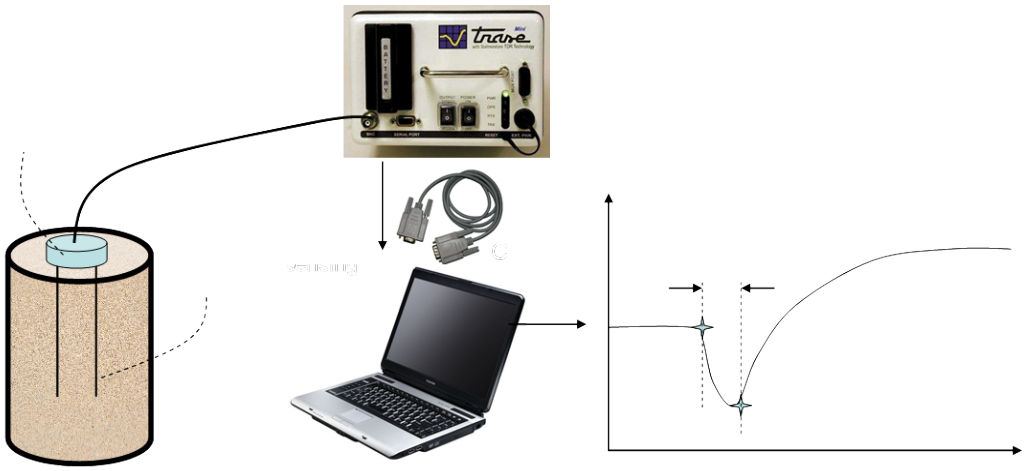

Fig. 1 Sketch of TDR apparatus for laboratory measurements of SWC.

The above relationship was initially considered to be universal and has been concluded to be accurate, within a few per cent, for medium-textured soils, and it needs adjustment for peat and heavy clay soils [15].

An even simpler empirical relationship presented by Ledieu et al. [16] was calibrated on loam soils of varying bulk dry densities (1.38-1.78 g/cm^3).

$$\theta = 0.1138\,\varepsilon_a^{0.5} - 0.1758 \qquad (3)$$

Ledieu et al. (1986) also demonstrated that using the bulk dry density (ρ_d) in the ε_a - θ relationship could improve the accuracy of the method.

$$\sqrt{\varepsilon_a} = a\,\rho_d + b\,\theta + c \qquad (4)$$

where a, b, and c are calibration constants (a = 0.297, b = 8.79 and c = 1.344).

The equation proposed by Malicki et al. [17] which contains not only the ε_a but also the ρ_d derived a higher level goodness of fit. The calibration was performed with wide ranges of soil textures and bulk dry densities.

$$\theta = \frac{\varepsilon_a^{0.5} - 0.819 - 0.168\rho_d - 0.159\rho_d^2}{7.17 + 1.18\rho_d} \qquad (5)$$

The accuracy of the round-trip travel time measurement and the calibration models are the essentials in order to make a successful prediction of the θ from the ε_a measured by TDR. Tangent line fitting procedure [2] and using the apex of the first derivative of the TDR recorded waveform [18] are demand to be reliable methods to perform the travel time analysis. On the other hand, the discrepancy between the existing calibration models and the experimental results, obtained with oven - drying method ($\theta_{measured} - \theta_{predicted}$), became notable especially for soils with high clay content because of the increase in the resolution of the TDR measuring systems. The cause for this deviation has not been completely identified but the bound water effect is one of the possible reasons due to its polarization of which is mainly impeded by the charged surfaces of the clay particles. The reduced polarization results in a lower apparent dielectric permittivity for the bound water (e.g., 6 to 30 depending on the bound water layer thickness) than that of the free water (around 80 at 20 ^{o}C), all of which will lead to measure a lower apparent dielectric permittivity. Therefore, TDR inherently underestimates the water content of soils with high clay content.

In contrast to earlier research [19], the bulk dry density of the soil is another possible source of error for the TDR based SWC due to its effect on the volume fraction of the bound water. Additionally the bulk apparent dielectric permittivity of the dry soils get larger as the dry density increases because the apparent dielectric

permittivity of the solid soil particles is in the range of 2 to 5 versus unity for the air [20]. For soils with considerable volume change potential, the effect of the bulk dry density becomes more important.

2. SLOPE MONITORING WITH TDR

Monitoring of slope stability involves observing the displacement of a slope in time. Accurate monitoring of slopes has been conducted with varying methods like inclinometers and extensometers. However, these methods are expensive and time consuming methods. They also require trained personnel for periodical monitoring of the slopes. Naturally, such a periodical measurement can sometimes result in missing of a movement in the slope. In addition, the obtained data by these methods needs to be carefully interpreted to determine a risk for slope stability. A schematic diagram of a TDR system used to detect movements in the slopes can be seen in Fig. 2.

As can be seen in Fig. 2, the TDR device sends the EM pulse into the coaxial cable embedded in the borehole. The borehole which traverses the anticipated shear surface of a slope failure is generally grouted with a weak sand-cement grout. When the pulse encounters an anomaly or defect along the length of the cable, such as those produced by shearing, a portion of the pulse energy reflects to the TDR device. The reflection causes a spike in the reflection coefficient-time graph. The relative amplitude of the spike and the reflection time are used for determination of the magnitude of the movement and the location of the shear surface, respectively. The relative amplitude of the spike increases as the displacement due to the movement of the slope increases.

The principle advantage of the TDR system is the ability of monitoring the cable sensor remotely by dataloggers or telemetry. A warning system based on TDR monitoring system provides an immediate warning when a movement occurs in the slope and critical facilities adjacent to unstable slopes can be real time monitored. TDR is also an advantageous slope monitoring method with its digital data that can be easily acquired and analyzed with suitable software.

3. BRIDGE SCOUR MONITORING WITH TDR

Bridge scour, refers to the lowering of the streambed around bridge piers or abutments, is one of the major causes leading instability of bridges constructed over waterways. According to the National Cooperative Highway Research Program (NCHRP) Report 396, score accounts for 60% of bridge failures in the United States and approximately 84% of all the 575,000 bridges require scour mitigation. Local scour caused by the interference of piers and abutments with stream flow is the most dangerous type of scour that results in holes around piers and abutments and reduces the lateral support of the surrounding soil [4].

The methods in practice for determination of bridge scour mainly rely on “after-the-fact” surveys such as visual inspection by trained divers. Therefore the obtained data does not reflect the real situation of the highest flow conditions due to the possible sediment inflow to the scour holes. Sonar monitoring and bathymetric radar are significantly useful methods but they have major limitations.

Using TDR technology for scour monitoring relies on the big differences between the dielectric permittivity of water and that of the air (1) or sediment soils (dielectric permittivity of dry soil varies between 2 and 7, while it increases as the degree of saturation increases), Fig. 3. The EM pulse reflects from the interfaces between these material layers due to the significant difference between dielectric properties. The probe consisting of two metallic conductors is connected to the TDR device and mounted on the surface of the pier, Fig. 4. When scour occurs the sediment layer thickness around the conductor of the probe differs and the cable signature changes. The difference in the layer thickness can be easily calculated with a trace analysis.

4. DETERMINATION OF CRACKS ON STRUCTURAL MEMBERS

Crack detection is one of the primary concerns in health monitoring of civil infrastructures, because cracks may lead to structural degradation [21]. Especially, knowing the structural integrity of deep foundations, tunnels, water tanks and similar underground structures is an expensive and time consuming process due to the lack of visual inspection.

Conventional point sensors can easily miss a crack because the inhomogeneous structure of the concrete does not lead to know where the crack will exactly occur. Two point sensors which measure the displacement between two points cannot differ a wide open crack and too many fine cracks. A crack sensing method using TDR technology has been developed for structural members without prior knowledge of crack location [5].

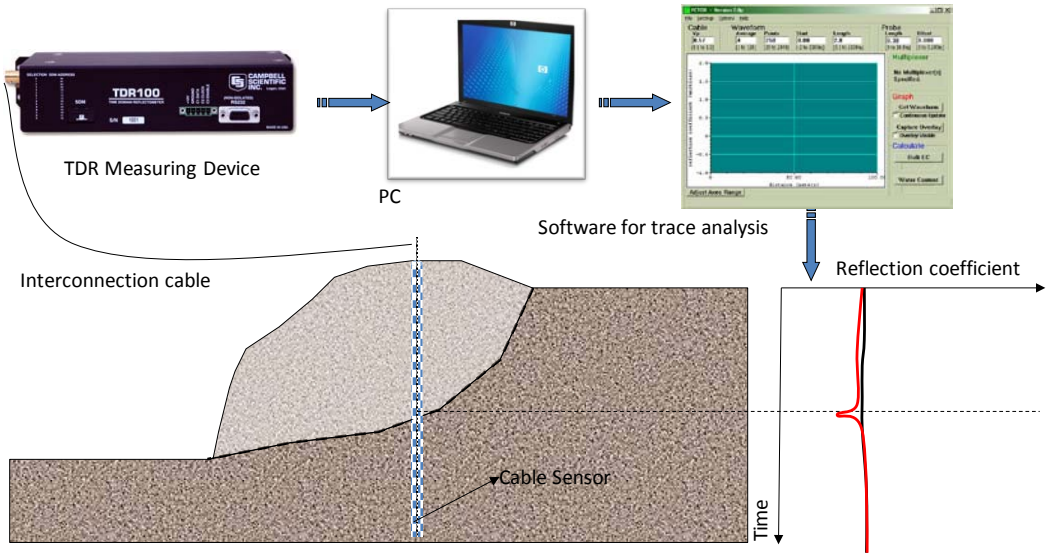

Fig. 2 A typical slope monitoring system with TDR technology.

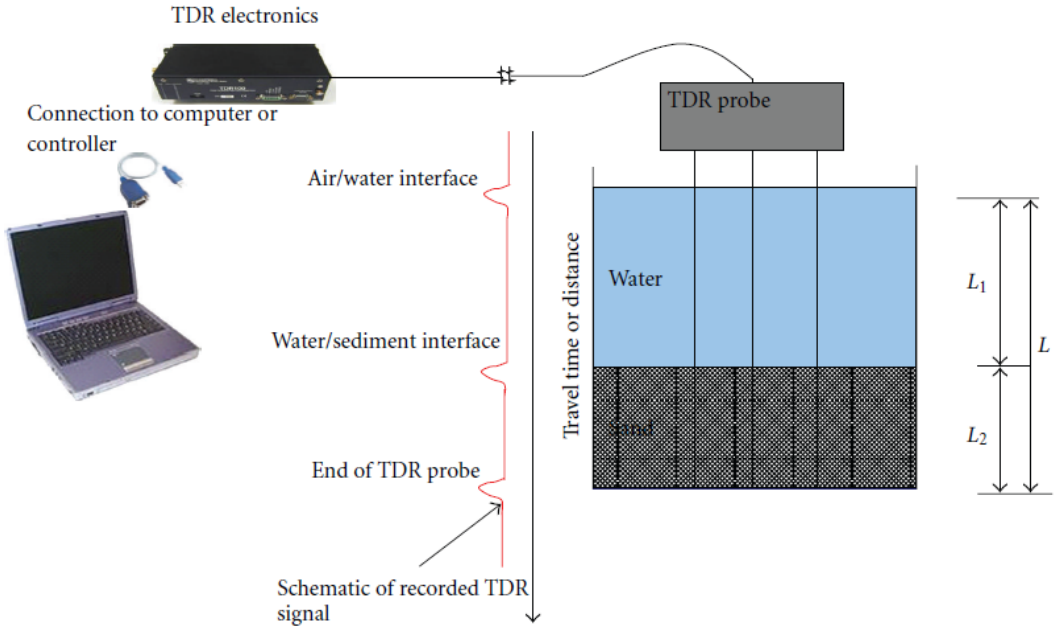

Fig. 3 Determination of sediment layer thickness with TDR [4].

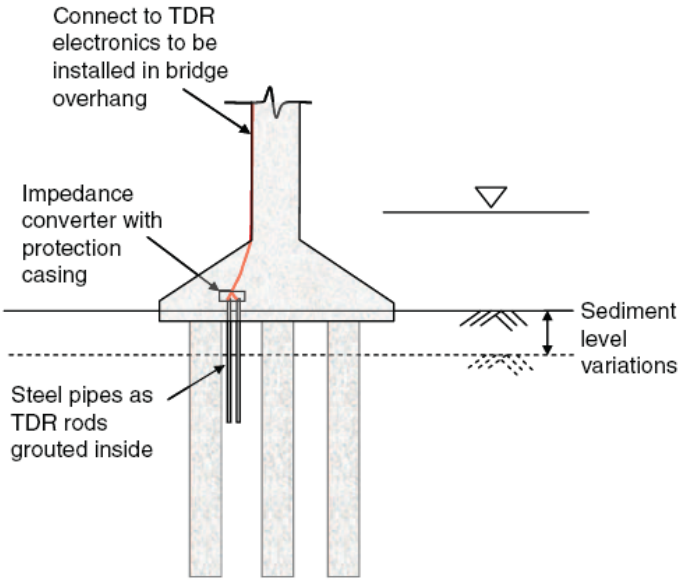

Fig. 4 TDR scour monitoring system with newly constructed pier [22].

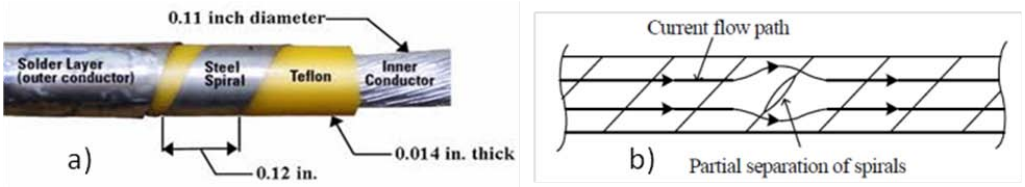

Fig. 5 a) Prototype cable sensor b) Change in current flow path [5].

The ability of placing a cable sensor to several locations in a structural member and inexpensive measuring system allow TDR to be a fruitful structural health monitoring method. The commercial coaxial cables are classified as non-sensitive for crack determination applications based on the results of the previous works. These cables consist of inner conductor, outer conductor, and a dielectric layer between these two conductors.

For crack sensing the commercial cables modified as making the outer conductor spirally wrapped to the dielectric layer and spray coated for continuity at zero strain (Fig. 5a). When a crack sensor is embedded into a reinforced concrete member, a crack intercepting the sensor creates a local separation between two spirals, generating a reflected electromagnetic wave when traveling through the coaxial cable, an electromagnetic wave guide (Fig. 5b).

The ongoing studies showing that well designed cable sensors can be used for proactive maintenance of structures. Their capability of sensing and locating a crack provides a remarkable potential for TDR to be a real time monitoring method of structures.

5. CONCLUDING REMARKS

TDR (Time Domain Reflectometry) is a rapid measurement method developed initially for finding the faults on electrical and communication cables. In recent years, the method has been applied to many civil engineering areas. TDR allows users to monitor the sensing mechanism remotely and continuously. Devices using TDR technology are usually integrated with a multiplexer, so hundreds of channels, connected to the datalogger, can be monitored simultaneously. Waveforms, produced with TDR, can be digitally recorded and easily analyzed with convenient software. Such features described above makes TDR a very suitable method for needs of real time monitoring issues in civil engineering.

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CHARACTERIZATION OF TIRE POWDER IMPROVED SWELLING SOIL

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***Abstract*—This paper presents the findings of tire powder stabilized expansive silt to be used as road base. A mixture with 30% tire powder by dry mass of soil proved to be a feasible mixture to be used as a stabilized road base. The tire powder caused reduction in unconfined compressive strength while the soil mixture gained ductility. It also reduced the swell potential and compression index significantly. Therefore, it is concluded that recycling of waste tire can be a cost effective and efficient method in the stabilization of expansive soils.**

Keywords: Swelling soils, tire powder.

1. INTRODUCTION

Although there are methods to reuse the waste tire material, still a high amount of tire remains unconsumed. Introducing new techniques for resolution of this problem is one of the challenging issues worldwide. The geotechnical application of waste tire, such as in backfilling of retaining walls and as a highway material in pavement design, are new approaches for recycling purposes. Waste tire can also be used to modify the mechanical properties of soils. Several research works have been carried out using different sizes of tire shreds and various types of coarse grained soil [1-3]. Shahin et al. [4] mixed bauxite sand with tire crumb and powder, the result of compaction test showed that by increasing the tire powder content the amount of maximum dry density decreased. Berrak et al. [5] studied sand and cement mixture with tire granulates and observed that increasing the amount of tire granulates decreased the strength of the mixture. Kim et al. [6] studied shear properties of 3 different tire powders with bottom ash in various ratios. They have observed that by increasing the tire powder percentage in various ratios of bottom ash, the amount of the shear strength and internal friction angle reduced.

Ho et al. [7] investigated that rubber chips as a soft clay stabilizer have not been effective in strength development but increased the failure strain, hence the ductility. Seda et al. [8] investigated the effect of small sized waste tire rubber on swell potential of Colorado expansive soil. The result showed that maximum dry unit weight reduced by adding 20 % tire rubber due to the lower specific gravity of the rubber material. Also, the swell percent and swelling pressure decreased and compressibility of the mixture increased with 20% tire rubber addition.

The limited literature reveals that there are still an inadequate number of investigations which have been carried out on the effect of tire improvement on fine-grained soils. This paper presents the findings of a study on the possible use of tire as an additive to stabilize the swelling subgrade soils in a semi-arid climate.

2. MATERIALS

In this study, silt with high swell potential has been selected which is used as a subgrade material on the Eastern Mediterranean University campus, Cyprus. Tire powder was used as an additive. Physical properties of the silt alone and its mixture with 30% tire powder by dry mass of the soil were determined.

2.1 Specific Gravity

Specific gravity of expansive silt and its mixture with tire powder was tested according to

ASTM D 854-10 [9]. Due to tire powder particles possessing lower density than water and its imminscible nature in water, gasoline was used instead in determination of specific gravity. Results of specific gravities are presented in Table 1.

Table 1. Specific gravity results.

Specimen	Specific gravity (GS) (gr/cm3)
Expansive silt	2.62
Tire powder	1.45
Expansive silt + 30% tire	2.11

2.2 Atterberg Limits

Atterberg limits of expansive silt and tire powder mixture were determined according to ASTM D 4318–10 [10], and presented in Table 2. With the addition the tire powder, cohesion of expansive silt particles decreased, hence reducing the liquid limit, plastic limit and plasticity index of silt. Thus these variations changed the soil type from fat silt (MH) to lean clay (CL) in Unified Soil Classification System.

Table 2. Atterberg limits.

Soil	liquid limit (LL)	Plastic limit (PL)	Plasticity index (PI)
Expansive silt	61	35	26
Expansive silt + 30% tire powder	47	24	23

3. RESULTS AND DISCUSSIONS

3.1 Compaction

The optimum water content and maximum dry unit weight of soils were determined by Standard Proctor test according to ASTM D 698-7 [11]. In order to achieve homogenous moisture content, the batches of soils were mixed thoroughly in mechanical mixer and samples were sealed in double nylon bags and kept for 24 hours preceding to each test. The dry unit weight and optimum water content of silt alone and the mixture with tire powder are presented in Table 3. The reduction in unit weight when tire is added can be attributed to the lower specific gravity of the tire powder, and therefore lower specific gravity of the mixture. As tire powder replaces the fines, the mixture gets coarser, and therefore the optimum water content is also reduced.

Table 3. Compaction characteristics.

Soil	γ_d (kN/m^3)	Water content (%)
Expansive silt	14.7	26
Expansive silt +30 % tire	12.57	22

3.2 Unconfined Compression Test

Samples compacted at optimum water content and maximum dry unit weights were tested in unconfined compression and the results are depicted in Fig. 2. Table 4 summarizes the test results which indicate that the tire powder addition to expansive silt decreased the cohesion between the silt particles while increasing the failure strain. This may be attributed to the resilient behavior of the powder. The stabilized soil has lower cohesion, lower stiffness modulus, yet is more ductile with an increased tensile strength.

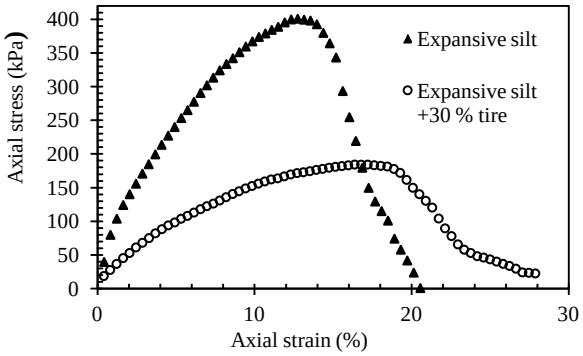

Fig. 2 Unconfined compression curves.

Table 4. Unconfined compressive test results.

Soil	q_u (kPa)	$\varepsilon_{failure}$ (%)	E (kPa)	C_u (kPa)
Expansive silt	400	12.8	3130	200
Expansive silt+30% tire	184	17.2	1070	92

3.3 One dimensional swelling –consolidation

Swell-consolidation test was carried out according to ASTM D 2435/D 2435 M [12]. Two samples were prepared at maximum dry unit weight and optimum water content. One-dimensional swell test was carried out under 7 kPa pressure, and the test results are presented in Fig. 3. Table 5 summarizes the swell test results

which include the percent primary swell, time taken for the completion of primary swell (t_s) and the rate of secondary swell. The results indicate that, tire powder reduced the primary swell by more than fivefold and the time of primary swell reduced by twofold. After completion of primary swell, swelling curves show an inclination to a gradual increase, the slope of which represents the rate of secondary swelling. It can be observed that the secondary swell rate of tire powder mixed soil is less than expansive silt. This behavior is explained by the reduction of swelling soil in the mixture.

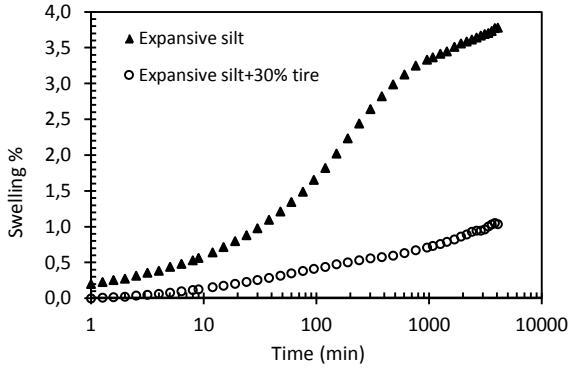

Fig. 3 Swelling curves.

Table 5. Swelling parameters.

Soil	Primary swell (%)	t_s (min)	Rate of secondary swelling
Expansive silt	3.4	930	1.60E-04
Expansive silt+30% tire	0.6	300	3.90E-05

Consolidation test results are plotted as void ratio versus effective consolidation pressure as shown in Fig. 4 and the parameters obtained from these curves are presented in Table 6. Consolidation curve of expansive soil is steeper than tire mixed soil concluding that the compression index of expansive soil is higher than tire powder mixed soil. This behavior is due to the elastic nature of the tire powder, resisting against compression. The preconsolidation pressure, which gives a measure of bonding between the soil and the tire powder, has almost remained the same indicating that the tire is not cementing the particles together. The swell pressure however has reduced significantly. This finding substantiates the reduction in swell percentage.

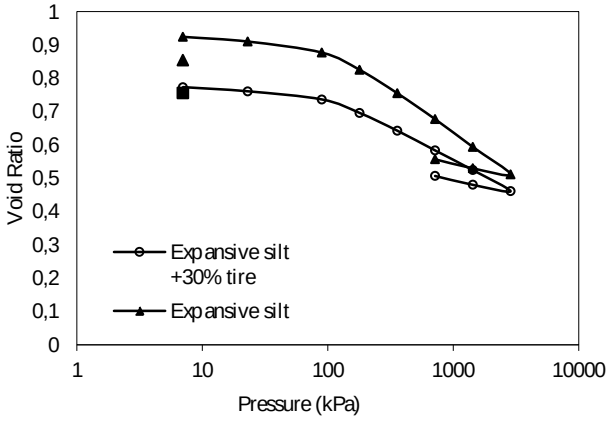

Fig. 4 Void ratio versus effective consolidation pressure curves.

Table 6. Compression test properties.

Soil	C_c	C_s	Pre-consolidation pressure (kPa)	Swelling Pressure (kPa)
Expansive silt	0.276	0.081	156	148
Expansive silt+30% tire	0.195	0.081	161	40

4. CONCLUSIONS

The test results proved that the tire powder technology can be applied to geotechnical and pavement engineering practices very efficiently. The powder inclusion reduced the swell potential significantly, while lowering the compression index as well, due to low compressibility of the material itself. The shear strength reduces, yet remaining within acceptable limits. The failure strain increases, which is due to the improved flexibility of the mixture. Finally, it can be concluded that, the prospective use of this technology will be very feasible as far as environmental and geotechnological aspects are concerned.

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PROJECT MANAGEMENT IN THE IRANIAN CONSTRUCTION INDUSTRY

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***Abstract*—Construction industry is an important industry and shares significantly in Gross Domestic Product in many countries. Enhanced share in GDP, increased number of employees, and annual investment growth are some factors amplifying the importance of private building construction in Iran. This necessitates the application of advanced project management principles to provide a sustainable growth in the sector. A survey was conducted to delineate the application of project management principles in private building construction in Iran. The survey results confirmed the instinctive implementation of fundamental management principles. It also revealed the low level of academic awareness and structured utilization of the management principles.**

Keywords: Construction industry, Iran, Private building construction, Project management principles, Quality management.

1. INTRODUCTION

Construction industry is one of the largest sectors in many countries. Construction industry's share in Gross Domestic Product (GDP) increased from 4.7% to 6.1% between 2001 and 2005 in Mazandaran province located in north of Iran [1]. The number of employees in construction sector increased by 215% and construction employment share rose from 9.7% to 15% form1996 to 2006. During this period, the annual capital investment increased from 50 million Dollars to 400 million Dollars. These figures highlight the crucial role of the construction industry to enhance the overall economy of Mazandaran and consequently the national economy of Iran. Mazandaran construction industry is labeled as being backward because of its relative lack of use of the latest advances in technology, management styles, and procedures. A questionnaire survey was thus conducted to analyze and find out the potential problems in the sector.

It was obtained from the survey that, most of the companies consider some management principles within their company and projects instinctively. However, they generally use traditional methods based on their experiences. Although the nature of construction industry resists the adoption of management techniques that were successful in other disciplines [2], a cultural and behavioral shift in the mind-set of all participants in the construction process especially top management is necessary if the construction industry is to improve its performance and competitiveness.

2. CONSTRUCTION MANAGEMENT

Construction works can be broken down by type of construction into residential, commercial/institutional building, industrial, and heavy/highway segments [3]. The main focus of the present study is on residential building construction since the contributors to the survey mainly involved in this area.

It is widely recognized that construction project as a discipline is a combination of art and science. Although the technical aspects of construction are extremely important, it is also essential that, construction professionals must have knowledge of management aspects of the profession.

Implementation of a construction project is a complicated and complex process. It requires contributions of different parties, successfully developed project design, choosing best available materials among a large range, using suitable methods, having capable and motive labor, and good supervision [4].

Some main benefits and advantages of project management implementation in construction industry could be:

- Better project control,
- Better organization reputation,
- Increased efficiency/profitability,
- More stakeholder/client satisfaction,
- Increased competitiveness,
- More effective communication,
- Better multi-project coordination,
- Improved resource utilization,

- Improved organizational culture,
- More staff satisfaction, and
- Greater innovation.

Iran, as a developing country, needs to improve construction management practices. On the other hand, there is a lack of specific investigation on the current status of the construction sector especially in Mazandaran province. Thus, present study aims to analyze the existing general management principles and quality management application in the construction sector in Mazandaran and propose suggestions for its improvement.

3. QUALITY MANAGEMENT

There are four stages of quality management (QM): inspection, quality control (QC), quality assurance (QA) and total quality management (TQM). Fig. 1 shows the stages of QM.

Inspection and QC are retrospective; they operate in a detection mode, aiming to find problems that have occurred. QA and especially TQM aim to reduce and ultimately to avoid quality problems occurring. This means they can be used to bring about improvement. Arditi and Gunaydin [5] and Arditi and Gunaydin [6] clearly defined and explained QC and QA.

The construction end product is not a repetitive unit, but an endeavor that may be unique in its design and composition. Internal and external factors such as community response, construction cost, and time of delivery must be addressed in the design and construction of a building [7]. Arditi and Gunaydin [8] stated the considerable attention given to improving the quality of the construction process.

They further pointed out the high potential of minimizing quality deviations on saving cost and time [6,8]. Quality in construction can be viewed as a part of a triangle in which the contractor must attain the cost level as planned, meet the schedule deadlines, and achieve the required quality level. An equative balance among these three aspects is considered as ideal. However, quality may be the first of these components to be disregarded or sacrificed in favor of increased cost savings and time reductions [7,10,11,12,13,14].

The projects in construction industry have been criticized for common cost overruns, expensive delays, high-accident rates, ever-increasing litigation costs, and declining international competitiveness. There is a consensus among professionals and researchers that the solution of the problem lies in application of formal quality management at all levels of design, procurement and construction [15]. Another study carried out by Abdul-Razek [16] concluded that, the factors affecting quality are inadequate information, poor communication, poor care in workmanship, and lack of site supervision.

Company-wide quality management has become a serious concern for businesses all over the world. However, in the construction industry the application of quality management concepts and tools has been more difficult due to its reactive nature and the complexities of the construction process [17].

Literature shows the lack of quality management survey in Iranian construction sector. Therefore, the present survey included quality management principles as a part of the questionnaire. The awareness of construction companies about ISO 9000 quality management system and the advantages and difficulties of its application were further investigated.

4. MAZANDARAN CONSTRUCTION INDUSTRY

Mazandaran is a province located in the north of Iran with a population of more than 3 million people [1]. Its capital city is Sari which has the largest share in the population.

Residential building construction in Mazandaran is mainly undertaken by private sector and NGOs. The most important NGO is called “Mazandaran Association of Mass Builders (MAMB)”. MAMB was established in 2002, and initiated its professional career by focusing on the following items [18]:

- Promoting members to apply professional construction methods and management,
- Managing unskilled constructors operating in construction sector in order to improve construction quality and to save national resources,
- Exploiting national policies on resource optimization, expanding mass building, reducing construction costs, expanding new technologies, and enhancing construction process quality,
- Exchanging experiences among MAMB members,
- Developing building and construction databases,
- Protecting members’ social rights and professional credibility and addressing their problems and difficulties, and
- Establishing relations with other local, national, and international societies.

MAMB has over 270 members which almost 50 members are construction companies and the remaining are individual builders. The majority of private construction activities have recently been undertaken by MAMB members.

Some statistics published by Mazandaran Urban and Housing Organization are as follows [18]:

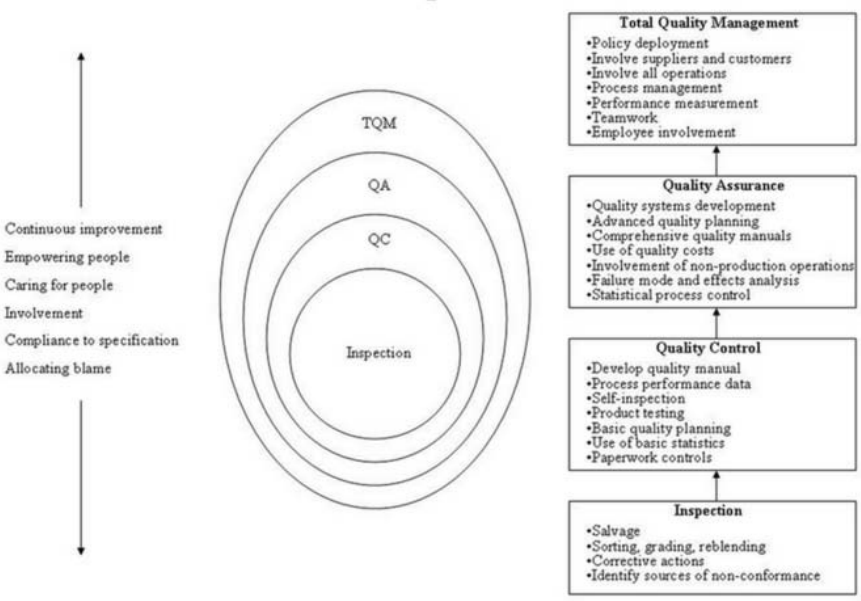


Fig. 1 Four stages of quality management [9].

- Number of existing houses in 1996 was 519,468 out of which, 60% were more than 10 years old, 24% were more than 20 years old, and almost 10% were more than 30 years old.
- Sari, Babol, Amol, and Ghaemshahr are the largest cities with populations of more than 100,000 people. Their shares in the province population are 0.17, 0.16, 0.12, and 0.10 respectively.
- Concrete and steel structures consisted only 7.4% of the total buildings in 1996 which increased to 36.4% in 2006.
- Private sector has built 78,079 buildings with the area of more than 19 million square meters between 1996 and 2006 which occupies almost 16 million square meters of land.
- 0.8% of the residential buildings were constructed by public sector.
- Construction industry's share in employment increased from 9.7% to 14.9% between 1996 and 2006 which places the industry in the second position after agriculture. It should be noted that northern region of Iran is the most important agricultural zone.

5. RESEARCH METHODOLOGY

A thorough literature survey was conducted to identify the important factors in construction management and quality management. A questionnaire was then prepared by considering the comments and recommendations of MAMB Education and Training Committee and submitted to the MAMB Board of Directors for approval.

The questionnaire consisted three parts. The first part included general questions about the company, its field of operation, number of employees, annual turnover, number of projects in a year, etc. The second part of the questionnaire was about fundamental management principles. The third part included ISO 9000 certification, its application difficulties, ISO certification advantages, etc. All contributors were asked to respond the first and second parts. The third part was asked to be filled only by the companies who had already been ISO certified or were in the certification process.

The questionnaire was administered to the companies who are the members of MAMB. The general managers of all companies were contacted by telephone to ensure the receipt of questionnaires. Out of 50 questionnaires, 28 companies responded which provided a response rate of 56%.

6. DATA ANALYSIS AND RESULTS

Data analysis process was initiated by first categorizing the data with the intention of discovering patterns and concepts related to different management knowledge areas. The data were analyzed by using SPSS software.

6.1 General Information

Regarding the companies participated in the survey, the following aspects must be noted for clarification purposes:

The majority of respondents were from Sari (53.6%) following by Amol (25%), Babol (14.3%), Ghaemshahr (3.6%), and Behshahr (3.6%).

20% of the companies were operating for less than 10 years, 32% between 10 and 20 years, 24%

between 20 and 30 years, and 24% more than 30 years.

40.7% of the companies undertook only private projects and 51.9% performed in both private and public projects.

96.3% of the companies were contractors and the rest were designers.

76% of the companies had grades from Management and Planning Organization (MPO) of Iran. These grades can be obtained based on the financial situation, available equipments and machinery, number of personnel and their education levels, and company work experience. Company grade governs the type and price of projects the company is allowed to undertake.

82.1% of the companies had less than 10 permanent employees. 24% of the companies had less than 10 and 48% had from 10 to 50 temporary employees.

The majority of the companies (71.4%) had annual turnover of $100,000 to $1,000,000 undertaking 2 to 5 projects per year.

6.2 Scope Management, Time Management, and Cost Management

The companies were asked about their projects' organizational breakdown structure (OBS) in an open-ended question. Although all companies stated that they have proper company level OBS and job responsibilities for their personnel, the answers to this question demonstrated the lack of awareness about OBS, different levels and positions in an OBS, job descriptions, and responsibilities.

89.3% of the respondents stated that they have project schedules and 96% of them declared they regularly review and update the schedules.

96% of the companies stated that they have proper cost analysis for the projects and they prepare monthly payment vouchers.

6.3 Communication Management, and Human Resources Management

Based on results, 92.9% of the respondents stated that they consider teamwork in their projects. 85.2% declared that they conduct periodic jobsite meetings.

76.9% of the respondents stated that they have some criteria for personnel performance measurement. The question was continued with an open-ended part to list the major criteria for performance measurement which resulted in criteria like commitment, timeliness, interest, creativity, client satisfaction, attendance, trust, proficiency, accuracy, and cost saving. The respondents were asked whether they have a motivation system for their personnel or not. 89.3% replied that they have. Although the main criterion for most of the companies is financial reward, some used other criteria including promotion, assigning jobs which they are more interested in, employ in future projects, and vacations and travels with family.

6.4 Procurement Management

The respondents were inquired about availability and utilization of proper contract forms for designers, subcontractors, etc. 89.3% replied that they have proper forms and 92% of these companies stated that they revise the contract forms after each project and update them accordingly.

100% of the respondents claimed that they have some criteria for selecting designers. They were asked to rank five criteria based on their importance. Success in previous projects obtained the highest rate, followed by friendship, least cost, friends' recommendations, and others (Fig. 2a).

The same criteria were checked for subcontractor selection and the same priority list was achieved (Fig. 2b).

Attending seminars, workshops, and fairs was another question which 92.9% of respondents affirmed that they attend at least three fairs and eight seminars and workshops on average in a year. They stated that some seminars and fairs are essentially practical and beneficial for their projects.

6.5 Quality Management

Quality management related questions were more included in the questionnaire comparing to other management knowledge areas.

44.4% of the respondents stated that they have quality department in their companies. Remarkable response was that quality department is only responsible for testing some materials and checking their conformance to standards in the companies.

Project supervisor was denoted as the most important person (64.3%) to perform quality controls in projects, followed by client (50%), and finally quality department (25%). Although 44.4% of the respondents declared that they have quality departments in their companies, only 25% affirmed that quality department undertakes quality related activities within the projects.

Respondents were asked to profess the method of ensuring materials' quality in their projects. The majority of companies preferred their experiences in previous projects (67.9%) as the main decision criteria for selecting materials. Almost 60% of the respondents stated that their computer utilization in daily activities is medium and lower.

Finally, the respondents were asked about ISO 9000 standards awareness. Only 46.4% of companies knew what ISO 9000 is and more interestingly, 50% of this 46.6% stated that it is a quality control system. Notably, 36% of the companies which claimed they have quality department, did not answer to this question.

7. DISCUSSIONS

Most of the companies consider management principles to some extent in their operations; however, they lack structured and scientific management skills.

Companies' general information illustrated that the majority of the contractor companies have less than 10 permanent employees and 10 to 50 temporary employees. The main site personnel are temporary workers who principally lack scientific management knowledge. This imposes negative impacts on cost, time and quality aspects of construction management. This problem can be alleviated by organizing training workshops for temporary workers.

Government and/or any other formal public authorities do not have any strict control on private companies in the sector. Unfortunately, this situation encourages the contractor to loosen the managerial discipline. It is emphasized by Mazandaran Housing Master Plan [1] that although material prices, labor wages, and land prices increased during 1996-2006, which caused about 1400 % increase in the cost of buildings, there was still a considerable interest to invest in this sector. People in Iran consider owning a house as prosperity and are eager to invest for it. Thus, governmental authorities are expected to involve more in promoting the application of project management principles in the private building construction.

Another important concern of Mazandaran Housing Master Plan [1] is governmental loans for construction sector. Almost 6% of the investments were covered by bank loans in 1996 which was increased to 122% in 2006 [1]. Thus, it is obvious that the loans were not invested in the construction sector but invested in other sectors like agriculture, tourism, food industry, etc. The longer duration of refunding the loans will reduce the profit to recover bank interests. Therefore, a proper time management and financial management would aid the contractors to make best use of the loans and increase their profit and also to direct the money back to the construction sector.

Since management principles were not scientifically applied and traditional construction methods were implemented, and there was not enough control over the building construction, some unprofessional builders entered to construction industry relying on their financial capacity [1]. According to Mazandaran Housing Master Plan [1], unprofessional constructors undertook almost half of the building construction which caused some conflicts between constructors and customers.

Population forecast shows the need for 272,999 new buildings in next 15 years [1] while the total number of buildings constructed by private sector during 1996-2006 is limited to 78,079. Government share in the same period was only 0.8%. Construction workforce productivity decreased from 25 to 17.4 square meters per person, investment efficiency reduced from 1 to 0.9, and physical capital efficiency decreased from 1.5 to 0.6 square meters over $100 in the same period [1]. It is clear that without proper time, cost, human resources, and quality management, not only the goals would not be achieved, but also they would be accumulated and transferred to the next generations.

It is interesting that considering all these data, Master Plan [1] expressed increase in management efficiency from 3.9 to 5 buildings per thousand square meters in the same period. It was considered as an improvement in management; however, there might not be an accurate definition of management efficiency and more appropriate criteria should be considered in calculating this important factor.

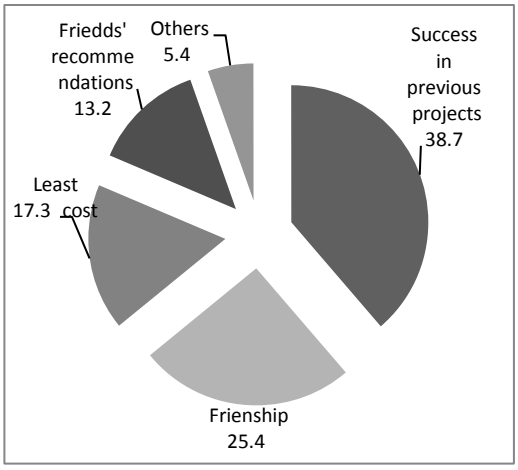
Different criteria used by companies for performance evaluation were praiseworthy, however, they might not have a proper structured method for performance measurement and their decisions are mostly subjective. On the other hand, a unified method can be established using the combination of motivation criteria to provide a basis for employee motivation for construction companies.

Arditi and Gunaydin [8] emphasized the popularity of TQM in the construction industry and its application in Japanese construction industry [6]; however, it seems that it is not yet scientifically implemented in Iranian construction industry. The responds to ISO standards awareness revealed that more than 50% of the companies did not know what ISO is.

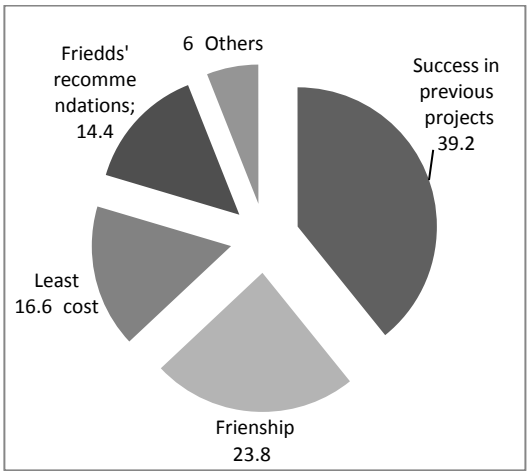
Concrete and steel structure increase and project completion time reduction in 1996-2006 period [1] revealed that constructors intend to use more industrialized methods to obtain higher quality, build the projects in a shorter time, and consider life cycle costing in the projects.

Mass building share in private building construction increased from 20% to 40% between 1999 and 2003. Mass building on the other hand optimizes land usage per unit building. Thus, MAMB can contribute to the construction industry by considering the following guidelines:

- Setting up more construction fairs,
- Proper training on new construction technologies and attempt to institutionalizing them,
- Organizing quality management workshops with the intention of imposing quality to all aspects of construction sector,
- Organizing investment appraisal and risk management workshops,



(a) Criteria for Designer Selection



(b) Criteria for Contractor Selection

Fig. 2 Importance of Selection Criteria for Designer and Contractor.

8. CONCLUSIONS

Construction industry is one of the most important contributors to GDP in Mazandaran province and consequently in Iran. Share to employment and investment makes it as second industry in the region following the agriculture.

A questionnaire was guided to analyze the construction sector in Mazandaran province. The questionnaire was sent to 50 companies and 56% of them replied. Regardless of company scope, annual turnover, and work experience, almost 70% of companies lacked a proper scientific management framework. This could be observed in all management knowledge areas including scope, time, cost, quality, human resources, procurement, productivity, and financial management.

Low level of computer exploitation in different operations was apparent evidence. Interesting project-level organizational breakdown structures proved the deficiencies in management knowledge. The criteria for personnel performance evaluation and motivation were found to be praiseworthy.

Although almost 50% of their operations and activities were instinctively performed based on the requirements of ISO 9000, more than 50% of the companies were unaware of ISO 9000 standards.

Finally, general feedbacks from questionnaires revealed that the great potential for management principles application to construction industry especially in the fields of quality, cost, time, and financial management are required to be improved.

- Establishing a framework or standard for construction project management based on regional considerations,
- Collaborating with other national and international professional associations,
- Creating databases for designers, contractors, sub-contractors, and material suppliers,
- Improving personnel work efficiency,
- Encouraging companies to use prefabricated materials in construction, and
- Educating labor force and grading them based on their experience and attendance to training workshops provided with certificates.

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UNDERSTANDING PROJECT COMPLEXITY IN CONSTRUCTION

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***Abstract*—The construction process may be considered the most complex undertaking in any industry. However, the construction industry has displayed great difficulty in coping with the increasing complexity of major construction projects. Therefore an understanding of project complexity and how it might be managed is of significant importance. This paper will present an overview of project complexity and will identify the complexity measures and severity factors driving project complexity in construction projects.**

Keywords: Project, complexity, construction industry, factors.

1. INTRODUCTION

Project Management has operated in a management environment of chaos and complexity for decades [1]. However, even though projects and project management are usually associated with the concept of complexity [2], both practitioners and academics have difficulties accepting and treating projects as complex systems [3]. Thus, mastering complexity is not a new challenge, but an old challenge that is being increasingly recognized and accepted [4]. There is increasing agreement that understanding complexity is important for project management because of difficulties associated with decision-making and goal attainment which appear to stem from complexity [5].

There is no widely accepted definition of project complexity that is research-based and, therefore, defensible [6]. The project management industry is sort of taking the stand that "You will know it when you see it." Project ever growing complexity is an ever growing source of project risks. Identifying existing project complexity sources and levels of project complexity has thus become a crucial issue in order to assist modern project management [7].

With globalization, technological innovation and society evolution, project planning and implementation for human use are becoming larger and complex. The complexity from organizational openness, socialization and adaptability brings a new challenge for the project management discipline [8, 9, 10, 11, 12]. Measuring and managing the effects of complexity of interconnections is also a new area. The construction process may be considered the most complex undertaking in any industry. However, the construction industry has displayed great difficulty in coping with the increasing complexity of major construction projects. Therefore an understanding of project complexity and how it might be managed is of significant importance [13]. The purpose of this paper is to present an overview of project complexity in construction and to identify complexity measures and severity factors for construction projects.

2. PROJECT COMPLEXITY IN CONSTRUCTION

2.1 Nature of Complexity

The development of complexity theory has lead to the observation that organizations, including projects, can be complex [2, 9, 14, 15], and viewed as complex adaptive systems. Nevertheless current operational definitions of complex projects, based upon size and budget have been challenged [6, 16], who raise research questions about how complexity can be measured in a robust manner, that takes account of structural, dynamic and interaction elements. The concept of complexity is being used as an umbrella term associated with difficulty and inter-connectedness. Complexity is usually also associated with variety, so that complex systems consist of the interconnection and interdependence of distinct parts [2, 4]. Typically, the characteristics of a complex project would include difficulty, uncertainty [17], uniqueness [18], indirect communication among elements [19], dynamism [20] and lack of clarity on the goals of the project [21].

Most authors have tended to focus on uncertainty, [22]; difficulty, to do with technical or management challenges [21] or organizational complicacy [2, 17]. Others have used systems theory to help understand how these aspects affect the project as a system [2, 17, 23]. Taikonda and Rosenthal [24] and Pundir et al. [14], relate technological novelty to technological maturity of the organization, immaturity leading to task uncertainty [5].

2.2 Project Complexity

Although the project complexity has not been clearly defined [15, 25, 26, 27], it is regarded as one of the critical project characteristics that determine appropriate actions to result in successful project outcomes [2]. It has been widely recognized that project complexity will influence the project performance and eventually affect the success of a project [28, 29, 30]. Whilst disagreement exists about the relevance of complexity as a concept to project management [6], articles associated with complexity, chaos and uncertainty are steadily increasing in the PM research literature [5, 28, 31, 32, 33, 34, 35, 36, 37].

Some authors (Baccarini, 1996 [2]; Williams, 1999 [15]; Remington and Pollack, 2007 [23]) have attempted to arrive at a more precise definition of the word 'complex', as it pertains to projects, by appropriating concepts from complexity theory to describe what might happen in these projects which are more than just difficult. For these authors, a complex project can be described as one which consists of many varied interrelated parts and which can be operationalized in terms of differentiation and interdependency [5]. In terms of organizational complexity, differentiation would mean the number of hierarchical levels (vertically and horizontally defined), number of units, division of tasks, etc. 'Interdependency' would be the degree of operational interdependencies between organizational elements. Technology can be divided into three areas: operations, characteristics of materials, and characteristics of knowledge [2]. Jones and Deckro [38] add another aspect to technical complexity; that of instability of the assumptions upon which the tasks are based.

"Project complexity is the property of a project which makes it difficult to understand, foresee and keep under control its overall behaviour, even when given reasonably complete information about the project system" [2, 7, 28]. Remington et al. [5] define a complex project as one that demonstrates a number of characteristics to a degree, or level of severity, that makes it extremely difficult to predict project outcomes, to control or manage project. These characteristics include high levels of interconnectedness, non-linearity, adaptiveness and emergence.

There is a good understanding of some of the causes of project complexity [39]:

- Details—Number of variables and interfaces
- Ambiguity—Lack of awareness of events and causality
- Uncertainty—Inability to pre-evaluate actions
- Unpredictability—The inability to know what will happen
- Dynamics—Rapid rate of change
- Social structure—Numbers and types of interactions
- Interrelationships—Many interdependencies and interconnections exist.

2.3 Complexity in Construction

Construction projects are typically characterized by complexity, under time and/or cost pressure and requiring creativity and cooperation [40]. Construction projects are invariably complex and since World War II have become progressively more so. In fact the construction process may be considered the most complex undertaking in any industry. However, the construction industry has displayed great difficulty in coping with the increasing complexity of major construction projects [2].

Construction projects are initiated in complex and dynamic environments resulting in circumstances of high uncertainty and risk, which are compounded by demanding time constraints. Construction industry is one of the most dynamic, risky and challenging businesses but the industry has a very poor reputation for managing risk, with many major projects failing to meet deadlines and cost targets. Bertelsen [41] discusses construction as a complex system; he explains that the general view of the construction process is that it is an ordered, linear phenomenon, which can be organized, planned and managed top down. Bertelsen [41] states that the perception of the projects nature as ordered and linear is a fundamental mistake and that project management must perceive the project as a complex, dynamic phenomenon in a complex and non linear setting.

3. MEASURING PROJECT COMPLEXITY

3.1 Project Complexity Models

Managing project complexity is a critical factor impacting project success and the application of conventional management systems to complex projects is inappropriate [2]. Macheridis and Nilsson [42] developed a conceptual framework developed in order to summarize how project complexity effect project management. Leung [43] built up Construction Complexity Index (CCI) as an objective quantitative tool to measure the complexity of construction for building projects. Geraldi [4] suggests and tests a method for assessing the perception of complexity in projects

anchored with the concept of Pattern of Complexity. Hass [34] describes the unique nature of complex projects and proposes an approach to manage project complexities. It presents the characteristics of highly complex projects and introduces the Project Complexity Model. The model is used to diagnose the level of complexity in a particular project or projects within a program. Lebcir and Choudrie [44] developed a project complexity framework for construction projects and evaluated its impact on project cycle time through a System Dynamics (SD) simulation model integrating project complexity, project operations, and its time performance. The existing Project Complexity Models in the literature are shown in Table 1.

Table 1. Project Complexity Models.

Models	**Resource**
A framework for the effects of complexity on project management	Macheridis and Nilsson (2004) [42]
Construction Complexity Index (CCI)	Leung (2007) [43]
A framework to diagnose the elements of complexity	Hass (2008) [34]
Pattern of Complexity	Geraldi (2008)[4]
Effects of Project Complexity on Time to Complete Construction Projects	Lebcir &Choudrie (2011)[44]

3.2 Complexity Measures for Building Projects

Although the importance of project complexity to project management has been widely acknowledged, few if not none objective measures are available for assessing project complexity [30]. The measurement of building complexity will differ among clients, designers, project managers and construction managers. Leung [43] pointed out that an objective tool to provide a quantitative scale to measure the complexity of building projects is urgently required.

It is has been widely accepted that the project complexity should be objectively measured in order to provide continuous feedback to help control the process of project development [2, 30, 43, 45, 46, 47, 48]. Table 2. indicates the complexity measures for building projects.

3.3 Factors Driving Project Complexity

In the construction field, there have been some attempts to measure the complexity of construction projects. However, given the fact that project complexity is hard to be quantified precisely, many researchers focus on identifying factors/aspects relating to the project complexity [30].

Table 2. Complexity measures for building projects.

Measures	**Resource**
• technical complexity of task; • amount of overlap and interdependencies in construction stages; • project organization; • site layout; and • unpredictability of work on site.	Gidado and Millar (1992)[49]
• the employed resources; • the environment; • the level of scientific and technological knowledge required; • the number of different parts in the work flow; and • the interaction of different parts in the work flow.	Gidado (1996)[50]
• client's attributes; • site condition/site access problems; • buildability of project design; • quality of design coordination; and • quality management.	Chan (1998)[51]
• expected project organization; • type of structure; • site constraints; • method of construction and construction techniques; • scale and scope of the project; and • complexity of design and construction	Akintoye (2000)[52]
• complex processes of communicative and power relating among project actors; • ambiguity and equivocality related to project performance criteria (success/failure) over time; and • the consequence of time flux (change, unpredictability and the paradox of control).	Cicmil &Marshall (2005)[53]
• the workers; • material; and • tools used in carrying out the project activity.	Sinha et al. (2006)[47]
• project duration; • working spaces; • contract sum; • site area; • type of structure; • height of building; • site location; • client; • usage of building; and • total floor area.	Leung (2007)[43]
• Building functions & structure • Construction method • Urgency of project schedule • Project size/scale • Geological condition • Neighboring environment	Xia and Chan (2012)[30]

Although there is an implicit acknowledgement that projects come with varying levels of project complexity, there is still a great deal of confusion about the factors driving this

complexity especially in the context of construction projects [44]. Wozniak [54] operationalizes project complexity based on nine diverse 'difficulty' factors such as: criticality of project, project visibility and accountability; clarity of scope definition.

In order to elaborate upon the concept of project complexity, Baccarini [2] deals with the two types of project complexity most commonly referred to in project management texts, that is organizational complexity and technological complexity. A complex organizational structure is one containing differentiated parts so that the greater the differentiation the more complex the organization. Another attribute of organizational complexity in projects is the degree of operational interdependencies and interaction between the project organizational elements.

Technological complexity by differentiation refers to the variety or diversity of some aspect of a task, such as:

- Number and diversity of inputs and/or outputs.
- Number of separate and different actions or tasks to produce the end product of a project. Actions can be regarded as separate when they are differentiated by technology, time or territory.
- Number of specialities (e.g. subcontractors or trades) involved on a project.

Technological complexity by interdependency can encompass interdependencies: between tasks; within a network of tasks; between teams; between different technologies; and between inputs. It is proposed that project complexity be interpreted and operationalized in terms of differentiation and interdependencies. There is a well-established body of knowledge asserting that differentiation and interdependencies are managed by integration, that is, by coordination, communication and control [2].

Projects have two primary areas for complexity – the technical complexity aspects of the project with the degree of difficulty in building the project and the business scope or management complexity aspects such as schedule, cost, risk, and communications [13].

Danilovic and Browning [55], Alderman and Ivory [56], Cooke-Davies et al. [10], Aritua et al. [57] have all highlighted similar attributes namely inter-relationships, self-organization, emergence, feedback and non-linearity and have discussed these effects in multi-project situations.

Uniqueness, indirect communication among elements [58]; Kumar, et al. [59], dynamism [20] and lack of clarity on the goals of the project [21] are also cited. Vidal and Marle [36] argue that project complexity can be characterized into four families. All are necessary but non-sufficient conditions for project complexity. They are project size, project variety, interdependencies and interrelations and context-dependence.

Geraldi [4] takes a slightly different approach when she talks about complexity of faith, complexity of fact and complexity of interaction. Geraldi and Adlbrecht [35] have concluded that these factors vary over the life cycle of a project. Trust, as an organizational capability, is also suggested as a significant issue for complexity in IT related projects [60]. D'Herbemont and César [61], develop a matrix model for classifying projects comprising two categories, technical and human. Their technical category appears to be a dimension, while their human category could apply to any dimension of project complexity and therefore might be considered to be a severity factor.

Table 3. Complexity Severity Factors.

Factors	**Resource**
Difficulty	Williams (2002)[17]; Geraldi (2008)[4]
Non-linearity	
Inter-connected	Williams (2002)[17]
Non-linearity	Remington & Pollack (2007)[23]
Inter-relations	Danilovic & Browing (2007)[55]
Inter-connectivity	Alderman & Ivory (2007)[56]; Cooke-Davies et al. (2007)[10]; Geraldi (2008)[4]; Aritua et al. (2009)[57]
Inter-dependent	Vidal & Marle (2008)[36]
Uncertainty	Remington & Pollack (2007)[23]
Uncertainty estimates	Williams (2002)[17]
Uniqueness	Luhmann & Boje (2001)[58]
Communication	
Indirect communication	Luhmann & Boje (2001)[58]
Context dependence	
Dynamism	Kallinikos (1998)[20]
Org. culture/work practices/ processes	Remington & Pollack (2007)[23]
Context and history	Pundir, et al. (2007)[14]
Context dependent	Vidal & Marle (2008)[36]
Clarity	
Lack of clarity	Turner & Cochrane (1993)[21]
Trust	Geraldi (2008)[4]; Müller & Geraldi (2007) [60]
Lack of trust	Müller & Geraldi, 2007
Trust	Geraldi (2008); Müller & Geraldi (2007)[60]
Capability	
Human	D'Herbemont and César (1998)[61]
PM capability/sponsors support	Remington & Pollack (2007)[23]

Amongst the latest contributors are Remington and Pollack [23] who provide a starting point for categorizing complex projects into four types or dimensions, based on the source of complexity: Structural, Technical, Directional and Temporal.

They emphasize that a clear understanding of the source of complexity helps in selecting appropriate tools and approaches to manage the project. Structural complexity stems from potential non-linear, emergent behavior which can occur from interactions between many interconnected tasks. Technical complexity is found in projects which have design characteristics or technical aspects that are unknown or untried. Directional complexity is found in projects where the goals or goal-paths for the project are not understood or agreed upon at all levels of the project hierarchy. Temporal complexity refers to volatility over the duration of the project, where project durations are extended and where the environment (market, technical political or regulatory) is in a state of flux and can affect the project direction. These can be seen as dimensions of complexity.

Remington et al. [5] argue that the factors contributing to project complexity may be defined in terms of dimensions, or source characteristics, which are in turn subject to a range of severity factors. Project complexity models tend to focus either on severity factors, factors that exacerbate the complexity, or dimensions, factors that characterise the nature of the complexity or a mixture of the two. Complexity severity factors that appeared in the literature are shown in Table 3.

4. CONCLUSIONS

The intention of this paper is to provide an overview of the theory on project complexity, identify the complexity measures and severity factors driving project complexity and to stimulate debate in the topic. Researchers assess the project complexity from different perspectives. The identified measures include not only specific characteristics of construction projects but also conceptual aspects relating to the theory of complex system. As projects have become more and more complex there will be increasing concern about the concept of project complexity and its influence upon the project management process. It can be concluded that there is no consensus on the identification of complexity measures for building projects. It is considered that the concept of project complexity is worthy of further consideration.

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ADOPTING BIM AS A NEW TECHNOLOGY INTO CIVIL ENGINEERING EDUCATION

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***Abstract*—During the first decade of 21st century improving the virtual 3D software reached to revolutionary tools called BIM. This research aims to find out how to integrate BIM into Civil Engineering department curriculum. A questionnaire survey was conducted among Civil Engineering graduate students of Eastern Mediterranean University. Results showed that more than three quarter of respondents had no acquaintance with concept of BIM and all of them believed that BIM is useful to understand details in courses. Based on research findings, offering BIM as a course in last semesters of Civil Engineering undergraduate level is strongly recommended.**

Keywords: Building Information Modeling, Education, Civil Engineering Department, Eastern Mediterranean University.

1. INTRODUCTION

One of the most popular and widely teaching-course in civil engineering curriculum is two dimensional drawings across the world. However with the advances in the technology, it is time to transition from 2D plan into 3D virtual ones. Using new technologies in construction projects will influences and enhance all the phases from design to implementation and project delivery.
Also it gives benefits of saving in cost and time. Because of such reasons, construction industry intended to go toward using new technologies and employing them. Building information modeling (BIM) is one of the latest improvements in industry and it is favored especially in construction industry and architectural designing.

Building information modeling (BIM) is a quickly growing technology gaining traction in the Architecture / Engineering / Construction (AEC) industry [1]. "BIM is a successor to the computer-aided drafting (CAD) which started in the 1980s" [2]. National Institute of Building Sciences [3] defines BIM as "a digital representation of physical and functional characteristics. A BIM is a shared knowledge resource for information about a facility forming a reliable basis for decisions during its life-cycle; defined as existing from earliest conception to demolition."

Need for BIM enabled engineers by construction industry in one hand and using the advantageous features of it to educate civil engineering students in the other hand, makes construction education related institutes and civil engineering departments to undertake surveys and studies on how to integrate BIM into their curriculum.

Liu and killingsworth [4] studied on applying BIM into an estimating course and reported the effectiveness of using it as a teaching tool. Cooksey and Schiff [1] have recommended including BIM at different level of civil engineering studies curriculum and replacing current engineering graphic courses. They also were indicating the importance of the BIM for civil engineering students, especially structure students. "These students need to have knowledge of the capabilities of BIM and how to effectively use the tool as they enter the profession" [1].

Clevenger et al. [5] have investigated and studied on a pilot implementation of BIM seeking

to understand and accommodate the imminent transformational shift in construction management education and evolve curriculum to facilitate better learning and understanding through BIM. Barison and Santos [6] identified categories of BIM courses based on students activities and have presented a basic organization for a BIM-Enabled curriculum along with guidelines.

In the existing research, it is suggested to implement BIM education and integrate BIM into the curriculum of civil engineering department of Eastern Mediterranean University.

2. METHODOLOGY

To reach the aim of this study, a questionnaire survey was conducted. Respondents were civil engineering graduate students of Eastern Mediterranean University. The questionnaire was in form of hard copy and the survey carried out in presence of a PhD student for assistance and students one by one filled questionnaire in their free time. Whole questions were close-ended and Total numbers of 27 persons responded.

At the first of the questionnaire, students were asked about their level of study and job experiences to find out their academic and profession background which 72% were Master students and 28 percent were PhD level. Clouse to 67 percent had job experience and approximately 33 percent were inexperienced (Fig. 1).

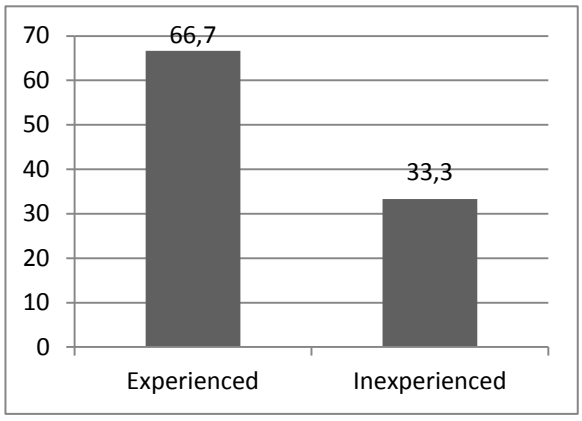

Fig. 1 Respondents percentage in terms of job experience.

Then they answered questions about their skills in drawing and design by hand, CAD software, and BIM software. Respondents selected one of five levels from "No Skill" to "Expert". Next part included questions about their knowledge of BIM. Finally respondents expressed their opinions on different aspects of offering subjected BIM course in civil engineering departments.

Questionnaires data one by one were imported to Microsoft Excel to statistically analyze students thought about BIM, based on their knowledge and experiences.

3. RESULTS AND DISCUSSIONS

According to questionnaire survey structure, the results were divided into three parts: drawing skills, BIM knowledge, and teaching BIM.

3.1 Drawing Skills

Drawing is one of the main requirements for civil engineers. To meet this need, about all of the civil engineering departments offer at least one drawing course. Although these courses emphasize on CAD (Computer-Aided Design) but hand drawing because of its role as a prerequisite of all types of drawing and design is teach prior to CAD drawing.

During the first decade of 21st century improving the virtual 3D software reached to revolutionary tools called BIM that not only affected design phase by 3D virtual models but also made evolution in construction and post construction phases (e.g. Facility management). Regarding to these changes, civil engineering departments are placed in transition process. Post graduate students skills in different types of drawing were investigated.

As shown in Fig. 2 students' skills in hand drawing and 2D drawing have same trend with different scales. Expert level of 2D drawing is more popular than same level in hand drawings. All students state that they can draw a plan for a 1 storey building with AutoCAD which prove the claim that there was no one without familiarity with these two skills.

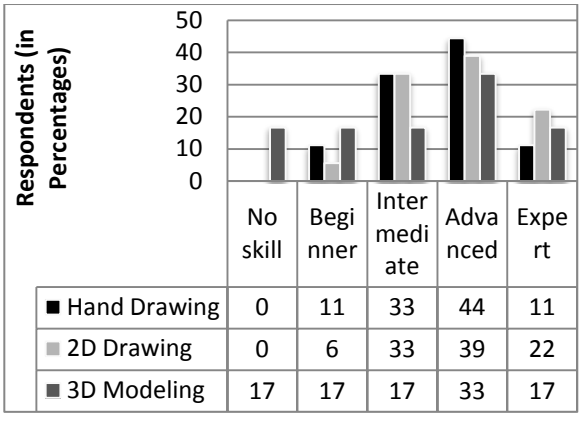

Fig. 2. Percentages of respondents in terms of different Drawing skills.

Distribution of respondents in terms of 3D modeling skills is quite different. 33 percentages claim that they are in advanced level and other percentages of respondents in other levels are equally 17%. Higher ratio of students with low levels and less number in high levels of skills in 3D

modeling in comparison with hand drawing and 2D drawing demonstrate that civil engineering department is in the middle of transition from hand drafting to Building Information Modeling.

3.2 BIM Knowledge

Weygant [7] in 'BIM Content Development: Standards, Strategies, and Best Practices' stated that "just as CAD (Computer-Aided Design) improved upon hand drafting, BIM is improving upon CAD. The difference is that BIM involves so many more project participants than just the architect." Close to 56% of students express that they have seen construction details in 3D modeling at any course nevertheless, less than half of this number (about 22%) among all students heard about BIM. These numbers show even though more than half of graduate students are familiar with 3D models, but more than three quarter of them have no acquaintance with concept of BIM.

According to widespread tendency of AEC industries' different parties to using BIM, increasing students' knowledge of BIM seems essential. It is obvious that because BIM was recently introduced to AEC industry and academia, graduate students did not have this chance to accustom with it. Civil engineering departments must make the opportunity for students to gain profits of BIM.

Regarding to low level of BIM's awareness among students, a brief definition of BIM was presented to them then, they were asked to answer further questions.

To find out respondents' opinion about BIM's usefulness, they answered to this question; "By Which extent 3D modeling could be useful to understand details in courses?" The answers were; 0%, 25%, 50%, 75%, and 100% (Fig. 3).

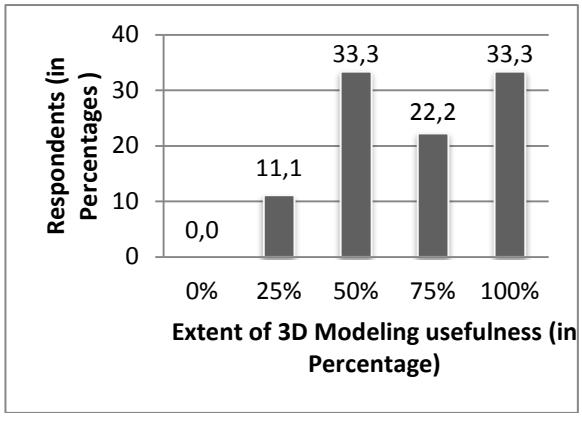

Fig. 3. Percentages of Students in terms of their opinion about extent of 3D modeling usefulness.

The results show that all of students thought that BIM is useful to understand details in courses, and about 90% of them chose extents more than fifty percent. Based on consensus of students on usefulness of BIM, some questions about offering BIM as a course in civil engineering department were asked as final part of questionnaire.

3.3 Teaching BIM

Some civil engineering departments have started teaching BIM as a separate course or among design or construction management courses, and some of them are planning to apply it in their curriculum.

95% of students thought there is a need to offer 3D modeling or BIM in civil engineering department. The question that come to mind is "in which stage BIM must be offered to civil engineering students?" about 72% of students thought undergraduate level is most appropriate level for offering such courese.

Approximately 22 percent said that BIM must be offered in master level and only little more than 5% preferred to such course offered in PhD level. Fig. 4 show that the most of graduate students think BIM course must be offered in undergraduate level. Opinions about offering BIM as mandatory or elective course do not sense while pros and cons were equally 50 percent.

As previously discussed graduate students preferred to had taken BIM course in their undergraduate education, it might be the reason why only 55% of them were interested to take such course if it will be offered in next semester and for the rest of them this course were unattractive.

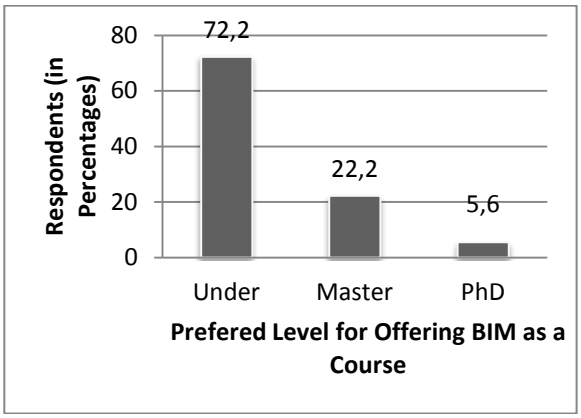

Fig. 4. students percentages in terms of their prefered level for offering BIM as a course.

As the final question, students were asked to specify their suggested academic year for teaching BIM in undergraduate level. Fig. 5 shows that 44% of respondents thought third academic year is suitable for teaching BIM and about 39 percent chose fourth year as their suggested academic year. First and second academic years were in next places with 11% and close to 6% respectively.

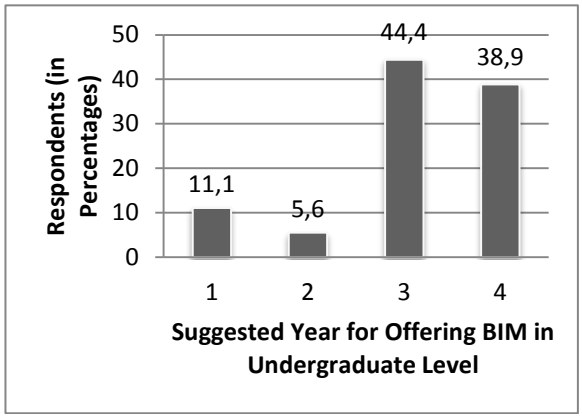

Fig. 5. respondents percentages in terms of suggested year for offering BIM in undergraduate level.

However, close to 85% of respondents suggested offering BIM in last two years of undergraduate level and understanding concept of BIM for juniors and seniors is easier, but introducing BIM in first semesters of study will help the students to improve their drawing and design skills and using this technological tool will assist teachers in presenting courses.

4. CONCLUSION

The intention of this research was to investigate on integrating Building Information Modeling into Civil Engineering curriculum of Eastern Mediterranean University. To achieve this aim, a literature review and a questionnaire survey among graduate students were carried out. The aim of questionnaire was to find out that how much graduate students are aware of BIM or ready to be trained.

Although, the respondents were PhD and Master students, but results showed that 3D modeling in all levels was lower than the same levels in 2D and hand drawing and only 22 percent of them had heard about BIM. About 95% of students believed there is a need to offer 3D modeling or BIM related courses in Civil Engineering department.

According to the results of this study, it is recommended to introduce BIM in first semesters of undergraduate study in order to help the students to improve their drawing and design skills. Moreover, using this technological tool will assist lecturers in presenting Civil Engineering main courses. Furthermore, offering BIM software as a separate course will prepare students for their future profession.

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INNOVATIVE STRATIGIES FOR TRANSPORT POLICIES IN INFRASTRUCTURE DEVELOPMENT

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***Abstract*—There are challenges and opportunities of deploying policies for transport infrastructure in developing countries. There needs to be a focus on developing innovative strategies for transport policies in infrastructure improvement. The objective of this paper is to outline the various innovative strategic approaches for improved transport policy and how to ensure that the strategies are well incorporated to solve the problems faced in the infrastructure sector in order to enhance improved economic growth.**

Keywords: Economic growth, Infrastructure improvement, Innovative strategies, Transportation infrastructure.

1. INTRODUCTION

Infrastructure can be described as the fundamental basis of all aspects and activities of an organized society in both social and economic dimension. It is the basic physical and organizational structures needed for the operation of a society and the economy. Infrastructure can be divided into; social and economic types [1]. The provision of infrastructure such as transport networks, water, sewerage, electricity and telecommunications play key roles in the development of efficient, healthy and sustainable environment [2]. Other urban facilities and amenities such as schools, health services, social services, markets, worship centers and recreation grounds are also important to development of any nation. The involvement of a wide range of stake holders is essential to the development of a shared and consistent approach of linking infrastructure investment projects and mega-projects to innovative strategic planning is also very important. The protection of such important infrastructure and services will influence response and reconstruction capacity, and minimize indirect losses, like disruption of flow of goods and services during rehabilitations and reconstruction. Therefore this will enhance the infrastructure improvement for developing countries [3].

The concept of innovation has been described as either ‘technical’ or ‘organizational’. Technical innovation involves either ‘product’ or ‘process’ innovation. Organizational innovation includes changes to organizational structure, introduction of advanced management techniques, and implementation of new corporate strategic orientations [4]. Innovation involves the use of creative knowledge for the introduction, development and growth of new products and services [5]. Innovation is a complex, lengthy and dynamic process. Hence, it becomes a strategic management technique involving the core stages of identifying of the need to innovate developing awareness, selecting appropriate innovative ideas, planning, and implementation which must be reviewed and modified to meet the challenging expectations of the problem [6]. Innovative strategies in the context of infrastructural developments involve bringing in new ideas drawn from local and foreign, from past and present, and future expectations. In today’s rapidly changing and globalizing construction market, innovation is

necessary because of the current inadequacies, such as, in construction infrastructure which has assumed critical dimensions in the following highlighted sectors: road, air, rail and inland waterways infrastructures in developing countries.

Innovative approaches for developing countries would be largely dependent on the methods and procedures used in developed countries. Introduction of advanced management techniques and implementation of new corporate strategic orientations are the key factors driving or hindering construction innovation. Attention to these factors by construction industry and stakeholders would be a key component of an effective innovation strategy and policy for any developing country [7]. Design practices and contractors work to create buildings and infrastructure works while on the hand are the myriad of individual construction tradesmen and small firms who operate in local markets, usually on maintenance works and domestic buildings. Construction activities are fundamental to economic and social development, while the outputs of construction – buildings, infrastructure works and transport networks – support all other economic and social activities, public and private infrastructure.

Economic and social infrastructure play a crucial role in the development of nations. They provide the basic foundation on which the superstructure of development and growth can be erected. If the economic and social infrastructural foundation is strong, development is therefore easily attainable and is also continuous, stable quantitative and qualitative [8]. The objective of this paper is to outline the various innovative strategic approaches for improved transport policy and how to ensure that the strategies are well incorporated to solve the problems faced in the infrastructure sector in order to enhance economic growth.

2. INNOVATIVE STRATEGIES AND ECONOMIC GROWTH

Innovation in construction is understood to be a proposed effective tool which aims to directly affect the process of improving performance of the construction industry productivity [9]. The past few years have seen many types of innovation in research and development in the construction industry [10]. The introduction of advanced management techniques and corporate strategic objectives are the key factors driving or hindering construction innovation [11]. An innovation strategy aims to mobilize the actors involved in a particular area of activity, whether an economic or social sector, in order to:

- review the situation in the domain or sector of activity in the region and compare it to that in competing regions;
- establish objectives and sectoral priorities, justifying these choices;
- identify and allocate financing available for each sector or area of activity;
- define public interventions under a multi-annual action plan to ensure continuity of action [12].

Fig. 1 below, shows the framework projecting a process of interdependent areas and dimensions, with the main aim of applying creative innovative strategies to improve competitive advantage and enhance productivity and growth. The identified branches of innovative strategies which cut across factors are such as Strategic Planning [Identifying and analyzing, and detail procurement of target areas], Customer Insight [Adopting strategies that will target users(user friendly)], Technology Planning [Strategic plan with technology to fit to required infrastructure deficiencies], Implementation [carrying out the strategies inline with planned scope], General Consulting Services [Repairs and Maintenance Services], Project Management [Planning, organizing, securing and managing resources to make sure strategies work] Local Government Services [Involving the local government in carrying out functions], Facilitation [Ensuring the strategies are running well]. This framework is a useful guide to decision making of those involved in planning and implementation of the proposed innovative strategies to be implemented.

Fig. 1 Innovative Strategic Framework.

The effectiveness of a strategy is strengthened by its duration which allows shared learning and ownership of the new approaches to develop and creates greater internal and external visibility. Strategic Innovation is not an end-state. Rather, it is a journey of open-minded exploration, experimentation, thinking, decision-making, action, results and learning, with the cycle then repeating [13]. Therefore, questions will lead to the implementation of innovative strategies and its applicability. An innovative strategy generates the creation of growth/development strategies, new process categories, services or business models that change the economy and generate significant new value for consumers, customers and the industry. The innovation process challenges the construction industry to excel above its already established operational boundaries and mental opinion models. An innovation must have a tight link to core performance competencies. The significant opportunities of partnering, outsourcing or acquiring new technologies and competencies must be considered in a well-structured innovative strategy.

Developing economies like Africa, is home to 13% of the world's population, and has only 39.1% of its population living in cities making it the least urbanized region in the world. Most of these urban and rural areas lack sufficient infrastructure to adequately support quality life. These deficiencies include lack of proper housing, sufficient electricity, sanitation, clean water and other essentials for life. Consequently, the lack of infrastructure in Africa is significantly greater for its population than the remainder of the world. A population of 373.4 million is currently living in urban areas of Africa, a figure anticipated to rise to 759.4 million by the year 2030 [14].

In many countries, anti-crisis public measures focus on higher public investment or investment in public objectives, like investment in infrastructure projects, as an important means to maintain economic activity during the crisis and support a rapid return to sustained economic growth. [15]. An instrument to realize these investments is the Public-Private Partnership (PPP) [16]. The scale of the challenge ahead is huge. But no country has a chance at developing economically and eliminating poverty until it puts in place the infrastructure necessary to support human activity [17].

3. CHALLENGES AND OPPORTUNITIES OF DEPLOYING POLICIES FOR TRANSPORTATION INFRASTRUCTURE

Achieving the transport policy goals will require the replacement of many of the currently existing transport systems and then replacing them with new ones within a relatively short period of time. Transport infrastructure will change. Modern infrastructure will increasingly incorporate new components, which make it smart (intelligent, ICT-enabled and automated), green (new light and recyclable materials) and intermodal (automated terminals, hubs, and equipment). It will integrate the provision of alternative, low carbon fuels and innovative management and operation systems. [18].

The analysis carried out by the Business Monitor International for developing countries Infrastructure revealed that the investment costs are not negligible; the market introduction of new solutions is mainly prevented by the lack of economic incentives for changes at ‘systems-level’, both for customers and investors. The deployment of smart, safe, and efficient transport solutions provides a tremendous opportunity to achieve environmental and climate objectives, as well as to increase construction market competitiveness [19].

Stakeholders and decision-makers need to declare where they stand on the balance between the different instruments, taking into account the urgency, acceptability and affordability of implementing the innovative solutions that are necessary to achieve our policy goals. User awareness should be increased and incentive measures could help to stimulate changes in consumer behavior that are coherent with the new technological innovation, thus facilitating market uptake and increasing demand for advanced infrastructure and services [20].

Stimulating innovation in mobility and transport will require mobilizing not only the major segments of the transport market but also blending them with existing or new emerging players from such fields as telecommunications, content-generation, financial services and the supply market. As a result, a clash between interests and entrepreneurial cultures may be created that is conducive to non-conventional and visionary thinking. [21].

Adapting the EU plan on innovation and strategic transport-technology will show the way for an effective development plan for any emerging economy. The strategic transport-technology plan will therefore also aim to:

i. Exploit convergences between distinct fields such as transport, energy, information and telecommunications services, territorial development, environment that can generate added value for the mobility of businesses and consumers and for wide range policy purposes, such as growth and employment. These should be pursued through a fresh approach based on new system-based concepts and pioneering ideas:

ii. Design a set of ‘out of the box’ operating principles and instruments as well as

interdisciplinary approaches to stimulating entrepreneurship, e.g. prize competitions, new venture-capital schemes, smart public procurement, etc., that can provide timely and adequate public responses to the requirements of the users' communities and the dynamics of the marketplace;

iii. Create a new dynamic of innovation in transport that can promote a renaissance of the sector at large, making it attractive to a new generation of talents, innovators and entrepreneurs. Coordinated investments in training and education and rethinking the skills they deliver might be necessary. Competitiveness of SMEs in the sector can be supported by better access to finance, easier access to European and international markets and less red tape [21].

There are key opportunities and challenges to the strategic transport infrastructure [22]:

- Benefiting from future economic and trade growth
- Changing policy objectives (competitiveness, green growth and a "greening of transport")
- Better gateway structures and organisation
- Improved strategic planning and evaluation processes
- The significant increases needed in infrastructure capacity
- The improvements needed in international connections
- Better funding and financing for gateways and inland infrastructure
- National policy frameworks better adapted to strategic transport infrastructure

National policy frameworks must set down how strategic infrastructure is to be planned, evaluated, developed and financed – as well as provide a solid basis for communication with stakeholders and the public. Most of the countries in which case studies were undertaken had good national policy frameworks; nonetheless, there is room for improvement [22].

The report of the OECD Futures Project on Transcontinental Infrastructure Needs to 2030/50 proposes a set of policy options to enhance the contribution of these infrastructures to economic and social development in the years to come. The options include recognition of strategic infrastructure (including gateways, hubs and key connections) in national policy frameworks and comprehensive measures to strengthen approaches and support the infrastructure development required.

National frameworks need to highlight the importance of strategic infrastructure. As the European Commission now recognizes, there needs to be a focus on strategic, multimodal "core networks" that can be funded and will be able to handle the major share of the future growth and transport tasks [22].

A recurrent concern is that many countries do not assign the same priority accorded gateway ports to the key inland rail, road and waterway connections required to move freight between the gateway ports and the cities and industrial areas in the hinterlands they serve. There needs to be a (new) "strategic infrastructure" category that actually includes the major international gateways and their key inland connections. The inclusion and linking of gateway and inland connection needs in national policy frameworks will be important for the downstream actions required, including reservation of land for gateway expansion, funding of the new corridors, and increasing capacity on the existing corridors needed for key inland connections [22].

4. CONCLUSION

This study addresses the innovative strategies to be adopted in the transportation infrastructural development and the contribution to developing countries. The problems areas identified, affect the economic, social and sustainable growth of the economy. There is a need for complete rehabilitation, new infrastructural installation and implementation of proposed strategies required for improvement. Further research should be carried out to assess the impact of the strategies proposed and to reveal their performance levels to measure for improvements in infrastructure development.

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CONCEPTUAL/THEORETICAL APPROACH TO EDUCATION IN CIVIL ENGINEERING

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***Abstract*—A proposal based on what are to be done to enhance education of civil engineering has been presented in this paper. "System of education" has been attributed to two main components: The first is to accept some fundamental lectures, such as physics, chemistry, mathematics, etc., as one and to employ them at relevant cases; alternatively to initiate conceptual lecture which can be introduction to theoretical lecture. The second is to lecture, in real site, in nature, after constituting a countrywide organization most of the time**

Keywords: education, civil engineering, academic, student, conceptual, theoretical.

1. INTRODUCTION

Education of civil engineering is so longstanding in Türkiye. In addition to existing School of German in the beginning era of the Republic, school of American has become included to system of education after sending academics to foreigner countries to be educated. These two different schools have been mixed both separately and along in Türkiye and so, this brought metamorphosed education methods along with as well.

Lectures, duration of lecture, format and methods of education have changed frequently. Experienced, and older, as well, academics who also have the position to philosophize express their displeasures, in symposiums, etc. On the other hand, employers and company owners also complain emphasizing insufficiency of education and schooling at universities, significant differences between universities, unrelated schooling at universities to applied engineering, insufficient education at universities in terms of human relations which causes ignorance of attitude in worksites, the low level of ability in managing groups.

How is/was education philosophy of Türkiye? In past, academies involved applied lectures, while faculties were theoretical. Function of academies of past is executed by vocational training schools today. When it comes to two main schools, ITU (Istanbul Technical University) and METU (Middle East Technical University), it can be said that teaching knowledge required to make projects is characteristic of ITU while teaching to find knowledge required to make projects is characteristic of METU.

Currently, in the frame of imperious education system, compulsory and elective lectures are carried out lecturer-oriented while projects are carried out lecturer-aided.

2. METHOD

2.1 Language Thematic

The issue of "language" must be handled primarily since it is the most significant topic in education. How to read and write the words must be reconsidered. Derivation, right utilization and selection of words must be cared. A determination must be indicated. The foreign (Ottoman, French, German, English, etc.) words of which have a synonym in Turkish must be eliminated since the assertion "more words mean an affluence" cannot be reasonable in this issue.

2.2 New Approaches in Engineering Education

The main questions which are under discussion to improve the topic are: Must the philosophy of education change?, Must the order of education change? Should an engineer be educated for market or must an investment be done for the future?, What may the affects of addiction to internet be?, Who must carry the weight, the lecturer or the student?, Does the fee-paying education acquire quality of education?, What may the affects of virtual schooling be? What is problem based schooling? [1].

Engineering consists of two parts, abstract and concrete ones. These parts, their contents and fundamentals are given in Fig. 1. All these require development of engineering skill.

Diversities in Engineering

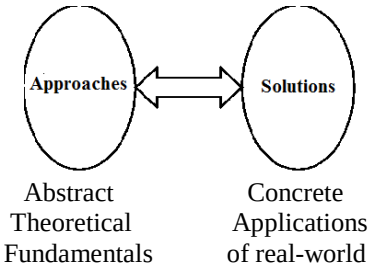

Fig. 1 Diversities in engineering.

2.3 A Transformation Example for Education Abroad

In a newly started model I observed during my visit to Southern Denmark Technical University in 2007 in content of an "Erasmus Activity", the weight was taken from the lecturer and given to the student. Duration of education has been reduced to 3 years from 4 and it has been aimed to educate students for markets and to make them concentrated on projects. The students have been diverted to group studies as well [2].

2.4 Loops in Engineering Education

While more superior professional talents, newer and more widespread talents show the cycles in education of engineering, the time interval that develops these properties does not change. Considering various schooling techniques in order to develop professional talents and selecting the convenient one between them seems the best way (Table 1).

According to Table 1, in order to learn better: 1. teaching to another; 2. learning by applying are the most convenient teaching/learning techniques.

Table 1. Regression in learning for various schooling techniques
(National Education Laboratory, Bethel Maine, USA)

Learning technique	**What remained of the learned one after 24 hours (%)**
Listening one-man show lecture	10
Audio-visual reading	20
Demonstration	30
Discussion group	50
Learning by applying (not applicable for all lecture-type	75
Teaching to another (not together learning)	90

2.5 Conceptual Approach

Trying to learn something without conceiving to what it serves, it only reminds of memorizing. The matter that has been done so far in all education systems nearly is this exactly. Human mind has not shown so important changes within thousand years. Mankind that has progressed due to issues such as genetic transfer, accumulation, experience, longer considering period and transaction has developed conceptual fundamentals firstly, and then theoretical ones. Structures built in antique world of thousand years ago are also evidences for that: mind firstly, calculation afterwards.

Two impressive examples of memorizing based education system have been given in Fig. 2 and Table 2. In Fig. 2 the students have failed in conceptual examination although they have passed the examination of lesson. The correlation between examination of lesson and conceptual examination has been summarized in Table 2. The transition between theoretical (intellectual) knowledge and applied knowledge has been shown in Fig. 3. As a result, conceptual schooling principle must be adopted first, theoretical one afterwards.

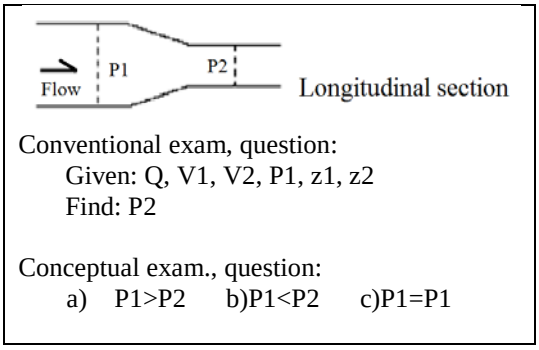

Fig. 2 Comparison of conventional and conceptual exams.

Table 2. Percentages between conception and passing ha the exam.
(Jacobsen at al. 1999, dyhr@mmmi.sdu.dk)

Situation		Conventional exam.	
		Failed	Passed
Conceptual exam.	Failed	25 %	45 %
	Passed	0 %	30 %

Note: Comment of lecturer: 60% of the students pass exams not aware of knowing "what's going on!".

Ability classes in engineering educational model

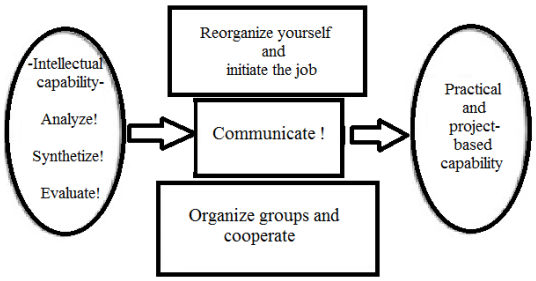

Fig. 3 Original information based ability series (*dyhr@mmmi.sdu.dk*).

2.6 Changing Educational Philosophy

The philosophy of education should be changed comprehensively, immediately, state offices aided, naturally/artificially experimented, with field excursions, conceptual/theoretical and project aided and the learning media of student should be organized. The student should be always in contact with object, group and project what combines of learning environment.

2.7 Learning Activities

All activities during learning can be summarized in four principles: Preparation of a motivating content; Activation of the students; Encouraging the conscious of group; Constitution of structural and related resource of knowledge.

The relation between weekly lecture hours and studying hours of students has been shown in Fig. 4. If lecturing hours in the classroom are more, homework should be reduced and when hours in the classroom are less, homework should be increased. Weekly lecture hours must not exceed 24 hours.

A continuous assessment must be carried out in a productive semester, duration and number of lectures and seminars must be compensated to project activities (Fig. 5). The scheme in Fig. 9 must be considered for the learning approach considered (Fig. 6).

Attracting student's attention; producing appropriate learning technique; promising for encouraging; including criteria of branch; giving hardworking students the worth and significance they deserve and defining the standards will mean to make an investment for the future.

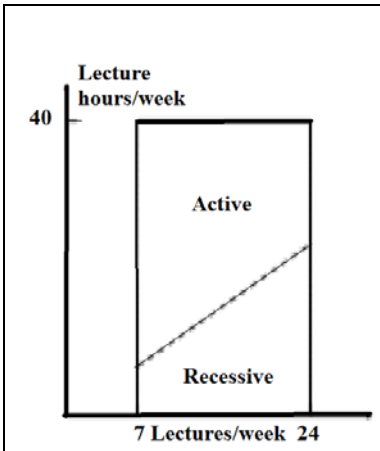

Fig. 4 Relationship between weekly lecture hours and working hours of the student (dyhr@mmmi.sdu.dk).

2.8 Changing of Roles

Professors should spend less time for lecturing and answering the questions but should guide and supervise the students instead. On the other hand the students should concentrate on searching solutions to open-end problems most of the time.

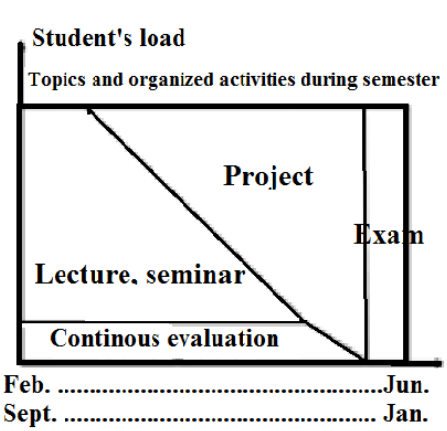

Fig. 5 Productive semester structure (dyhr@mmmi.sdu.dk).

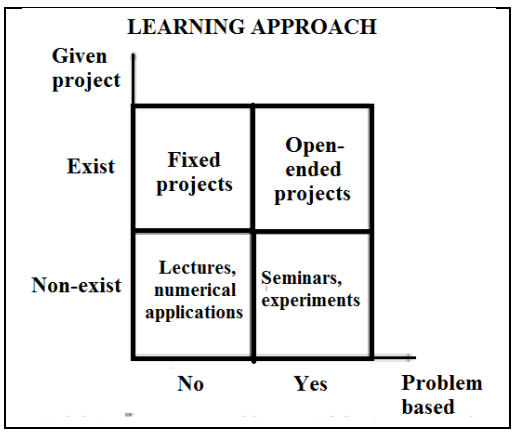

Fig. 6 Learning approach (dyhr@mmmi.sdu.dk).

3. MODEL OF CONCEPTUAL / THEORETICAL APPROACH

3.1 General

The difficulty in the perception of teaching for instance "Ricatti Differential Equations" to students who think they are educated for a vocation cannot be neglected. Lectures which seem like boring and weak to be sampled vocationally now should be removed; their example based general/specific knowledge should be given by mentioning in content of vocational lectures when it is necessary. So that, members of these branches will be fully free and have more time for their academic studies.

Vocational lectures should be divided into two groups: 1. Conceptual; 2. Theoretical. Conceptual lectures should accompany conceptual parts of theoretical lectures and theoretical lectures should accompany calculative aspects of conceptual lectures. By the way, conceptual preparations should be operated firstly. For that reason, textbook(s) on "Conceptual Civil Engineering" should be written (Conceptual Structural Engineering, Conceptual Geotechnical Engineering, Conceptual Water Engineering, and Conceptual Transportation Engineering etc.). An example on this issue has been carried out by Yanmaz and Usul [3].

3.2 Proceeding of Lectures

Number of students should be limited by 30 (1 bus; 1 trailer for the lecture* 5 coached, 6 sequenced* and trailer for the breakfast). Lectures should be carried out in field/nature, on bus, trailer, in state offices, in worksites, in laboratories (state office/school). Lectures should be in site and visual/audible (conceptual/theoretical based). For this aim, facilities of public and/or private sector should be benefited.

After this stage, the project should be done in the classroom, state archives and data should be utilized in planning stage for real projects, thus it should be aimed to carry out preliminary studies in a meaning to support state projects. In this case, will the education exceed its original duration? No! The space obtained by removing lectures like mathematics, physics, chemistry will be filled through conceptual lectures; in final 2 years project will be carried out and also theoretical lectures will be learned by the student under the supervision of the professors/lecturers (event based education, in some extent).

By creating small studying spaces in school, groups will be encouraged to use those spaces as if their houses; everybody will have mobile computer, the Department will hire them to student free of charge but obligatory, will take them back in graduation, will give upgraded ones to newcomers. Will the student adopt and support this system? "Yes!", with a great probability. Because they will realize that they will be learning new things even from the first day on.

3.3 Operation of the System

The system will be operated rotationally. Each professor will give the conceptual knowledge of branch they are specialized to students (week, two weeks, month based). How will the professor be hired? Professors will be schooled proper to this system and will earn day to day/excursion payments as long as they are in field. Will the existing personnel adopt this system? They will, with a great probability, as long as day to day/excursion payments are satisfactorily. How will the state support? It will supply for accommodations, give knowledge about the fields to be excursed, distribute documents. The same is desirable for the private sector; it will take the support of state, they will know the successful and hardworking students to employ them in the future in their firms.

3.4 Requirement of Changing for Student Profile

Educational profile of the student should be changed. It is too hard to reach any aim without changing profile of expedient student. The concept of expecting presentations from professors, also one-man show; listening-seemed students; random studying only during examination week; review only certain chapters instead all, if he catches "the big fish" then it is ok, if not, he blames on the lecturer. Percentage distribution of a 14 questions including small survey related to topic carried out in March 2009 in Civil Engineering Department of Engineering Faculty of Pamukkale University on 66 students studying "Scientific Writing" has been given in Table 3. This table shows explicitly the habits of the students by the selection of elective-courses.

Five impressive results of this survey can be summarized as follows:

1. Passing-warranty is most important condition.
2. Weekly schedule doesn't play any role by the election of the course.
3. No need any homework, quiz, class-room presentation, etc.!
4. Lecturing capability of the lecturer important if I know him previously.
5. The content of the lecture is important, less scope means greater success.

These results give an opinion about the way the student has been conditioned and the points in which these habits have to be changed.

Table 3. Reasons by the selection of the elective-courses.

Question	Accepting Ratio (%)	Question	Accepting Ratio (%)
I elect depending on the lecturer	80	I do not care the quota of student	98
I elect according to content of the lecture	85	I elect the lecturer who made me failed	73
I care whether pass is easy or hard	79	The schedule doesn't affects my election	79
I care the conduction of supervisor	83	I elect the lecture I guess I will pass certainly	68
I care the students previously followed same lecture	86	I do not elect the lecture that has syllabuses	61
I do not care the lecture my friend elected	77	I do not want homework, quiz, presentation, etc.	61
I plan the directions on my own	70	My professor does not affect my election	71

4. RESULTS AND PROPOSALS

The present higher-education of civil engineering over the world is far away from conceptuality in general, but it is based mostly on the memorization. The student passes the semester without not aware of "what's going on". Most of the lectures are in the classrooms, it takes place very limited site-trips, most the lectures are theoretical and not well-matched with practical engineering. The load and responsibility of the lecturers are heavy, so that they have no time to slot their investigations.

Civil engineering education has to be revised in its entity. A comprehensive transformation in the system is a due. The students must not have the feeling "I do not know anything" when they are graduated. For this aim, fundamentals of an education have to be established comprehensively, immediately, state - aided, naturally/artificially experimented, with field excursions, conceptual/theoretical and project aided. By the way, weight of the professor must be reduced (they must guide, supervise and evaluate only); activity hours of the student must be increased (diverting to projects); the student should aware of that school is his home (allocation of small lockable rooms to working groups, supplying computers, printers, stationary material etc. to every member).

Mutual compulsory lectures like mathematics-physics-chemistry must be removed, their spaces must be filled by conceptual lectures instead; these lectures must take place in content of theoretical lectures when they are met and required.

Lectures must be carried out where they should be learned, e.g. in field, site and real environment. A serious organization and process of transformation are needed for this aim.

Habits of students must be changed, the starving must be prevented, and their perception-measures must be changed properly.

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ENGINEERING ETHICS AND ITS MEANING AMONG DISCIPLE-STUDENTS

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***Abstract*—Engineering ethics has become a scientific branch during the last a couple of decades, which is already found place in consciences since ever , but its rules have been described not clearer long before and in between included newly in lecture notes and educational books. In this study, 200 students from Pamukkale University/Denizli/TR who are mostly not selected the lecture "Engineering Ethics" are asked the following questions: 1. What is ethics?; 2. Have you ever selected the lecture "ethics"?; 3. Should be due the lecture "ethics"; 4. Is the "ethic" a receipt that evaluates the rules truly and active rightly?; 5. What would you say if you express "ethics" with one word?. The most impressive result is that it is a receipt for "positive behavior" which is entirely wrong.**

Keywords: ethics, engineering ethics; engineer, receipt.

1. INTRODUCTION

Engineering is an important and learned profession. As members of this profession, engineers are expected to exhibit the highest standards of honesty and integrity. Engineering has a direct and vital impact on the quality of life for all people. Accordingly, the services provided by engineers require honesty, impartiality, fairness, and equity, and must be dedicated to the protection of the public health, safety, and welfare. Engineers must perform under a standard of professional behavior that requires adherence to the highest principles of ethical conduct.

Based upon the research available it is possible to say that ethical values have not been effectively taught and applied to education. Proof that this has not occurred is evident in the injustice, wars, crime, drugs, corruption, illegal businesses, an absence of fairness and trust, as well as the lack of peace that still persists worldwide (Gadner, 1990). Educational systems at all levels need to dedicate greater effort to find solutions for the preceding problems if they are to be effective (Rodriguez, 1996). Ethical values are roots that support the endless improvement of humankind enhancing reason, which distinguishes us from animals (Holmes, 2003). It seems, as Lockwood (1997) indicates, that there is not enough time for this extremely significant matter while educational systems focus upon more immediate concerns related to the ever increasing accountability requirements solicited by the state and federal government.

Philosophers throughout history have repeatedly provided an answer. Their message is very simple and may be summarized as "the lack of application of ethical values". Ethical values have been discussed across centuries including their influence upon education in many different ways as well as those values that people should apply.

Purpose of this study contained which investigate engineering students' perceptions of ethical values, including their importance, application, usefulness, origin, and benefits.

2. ETHIC CONCEPT

Ethics, as well as morals, are terms that have been very widely discussed from ancient times to the present.

Ethics, also known as moral philosophy, is a branch of philosophy that involves systematizing, defending, and recommending concepts of right and wrong behavior. It comes from the Greek word ethos, which means "character". It comes from the Latin word morality, which means "behaviors, habit". It comes from the Arabic word hulk, which means "moral". In general, the words ethics and morality are used interchangeably in everyday life is widely mistaken for each other.

Morality is the differentiation of intentions, decisions, and actions between those that are good (or right) and those that are bad (or wrong). Ethics is two things. First, ethics refers to well-founded standards of right and wrong that prescribe what humans ought to do, usually in terms of rights, obligations, benefits to society, fairness, or specific virtues. Secondly, ethics refers to the study and development of one's ethical standards. As mentioned above, feelings, laws, and social norms can deviate from what is ethical [5].

3. ENGINEERING ETHICS

3.1 General

Engineering is an important and learned profession. As members of this profession, engineers are expected to exhibit the highest standards of honesty and integrity. Engineering has a direct and vital impact on the quality of life for all people. Accordingly, the services provided by engineers require honesty, impartiality, fairness, and equity, and must be dedicated to the protection of the public health, safety, and welfare. Engineers must perform under a standard of professional behavior that requires adherence to the highest principles of ethical conduct.

3.2 Engineering Ethics

Engineering ethics is the field of applied ethics and system of moral principles that apply to the practice of engineering. The field examines and sets the obligations by engineers to society, to their clients, and to the profession. As a scholarly discipline, it is closely related to subjects such as the philosophy of science, the philosophy of engineering, and the ethics of technology.

In October 5, 1977; The World Engineers Union of ' meeting held on engineering ethics are as follows:

"Engineers, the accuracy of the engineering profession, the dignity and value of human beings using their knowledge and skills to live a comfortable, honest and impartial service to the public, and the loyalty of their business, trying to increase their own prestige of vocational and technical disciplines, promotes and advances".

Today, Engineering's the design during engineering decision results which affect the hunders of life. Therefore Engineers must work accordance with the ethical principles of their profession, without deviating from them, and without compromising. The main purpose of the determination of the principles of engineering ethics, must be provide guide duty for provide better services to the people of profession [1, 2,3].

3.3 Engineering Ethic Principles

3.3.1 Fundamental Canons

Engineers, in the fulfillment of their professional duties, shall:

1. Hold paramount the safety, health, and welfare of the public.
2. Perform services only in areas of their competence.
3. Issue public statements only in an objective and truthful manner.
4. Act for each employer or client as faithful agents or trustees.
5. Avoid deceptive acts.
6. Conduct themselves honorably, responsibly, ethically, and lawfully so as to enhance the honor, reputation, and usefulness of the profession.

3.3.2 Rules of Practice

1. Engineers shall hold paramount the safety, health, and welfare of the public.
2. Engineers shall perform services only in the areas of their competence.
3. Engineers shall issue public statements only in an objective and truthful manner.
4. Engineers shall act for each employer or client as faithful agents or trustees.
5. Engineers shall avoid deceptive acts.

3.3.3 Professional Obligations

1. Engineers shall be guided in all their relations by the highest standards of honesty and integrity.
2. Engineers shall avoid all conduct or practice that deceives the public.
3. Engineers shall not disclose, without consent, confidential information concerning the business affairs or technical processes of any present or former client or employer, or public body on which they serve.
3. Engineers shall not be influenced in their professional duties by conflicting interests.
4. Engineers shall not attempt to obtain employment or advancement or professional engagements by untruthfully criticizing other engineers, or by other improper or questionable methods.
5. Engineers shall not attempt to injure, maliciously or falsely, directly or indirectly, the professional reputation, prospects, practice, or employment of other engineers. Engineers who believe others are guilty of unethical or illegal practice shall present such information to the proper authority for action.

6. Engineers shall accept personal responsibility for their professional activities, provided, however, that engineers may seek indemnification for services arising out of their practice for other than gross negligence, where the engineer's interests cannot otherwise be protected.
7. Engineers shall give credit for engineering work to those to whom credit is due, and will recognize the proprietary interests of others.

4. NECESSITY OF ENGINEERING ETHICS

4.1 Survey Results

In engineering education, ethics as compulsory or elective course in many school studied in it. This is necessary, by such as the International Gauge Federation FIG and the US Engineering and Technology Equivalence Institute we strongly emphasized.

In universities ethical teachings, Although this means that the rules of the engineering ethic's behave engineers, students may experience difficulties in our professional life is critical to prepare against the dilemma [4].

The results of 5 selected questions for his study performed by 200 students from various faculties of Pamukkale University/TR are presented in the Figures 1 through 5 below.

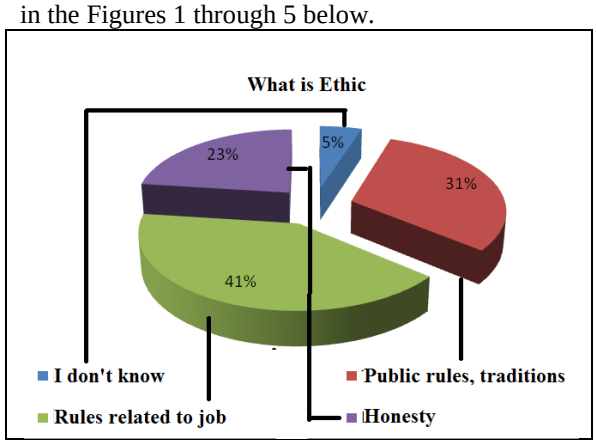

Fig. 1 Answer of the survey question “What is the Ethic?”.

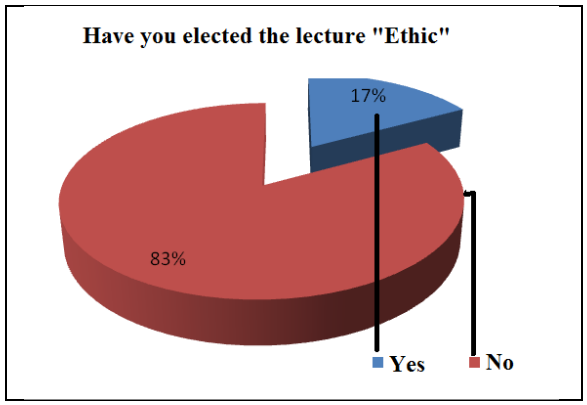

Fig. 2 Answer of the survey question “Have you ever elected the lecture Ethic”.

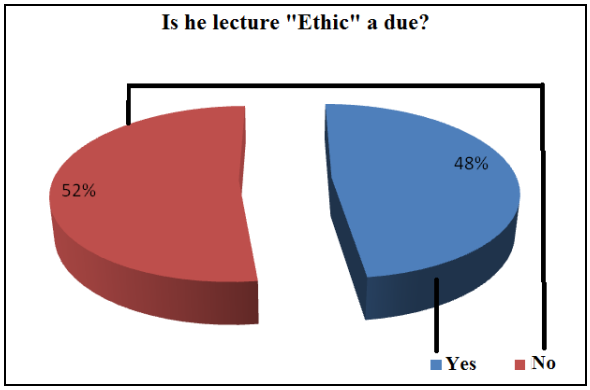

Fig. 3 Answer of the survey question “Is the lecture Ethic a due”.

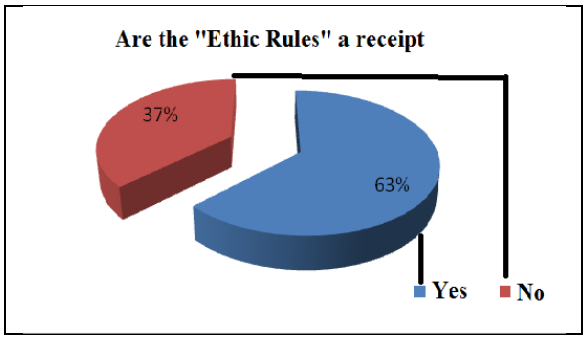

Fig. 4 Answer of the survey question “Is Ethic a receipt of right evaluation and proper action of an event.

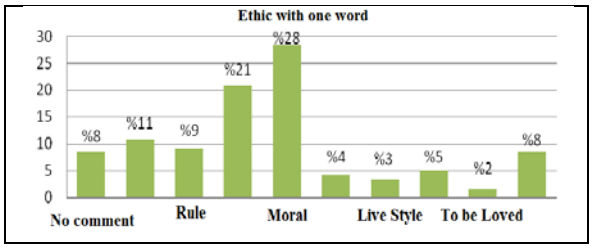

Fig. 5 Answer of the survey question “What would you say if you express Ethic with one word?”.

4.2 Results

Among the findings from the survey results:
1. Teaching of Ethics must be earlier;
2. Ethic is a chain of rules;
3. Absence of the lecture “Ethic” is not a deficit.
4. It is submitted e receipt for an proper evaluation of the concerned event which is not true;
5. It is not more than honesty and morality.

5. CONCLUSION AND DISCUSSION

As a result of the limited survey; the concept of ethics among students are not known and, they have no right information about it, even no approximate vision. There isn’t ethic course as

compulsory and/or elective course in many faculties and departments. But, it is expected that it should be a series of rules. It seems, that is a result of “memorization” and “ready-made” in our education system.

Today, problems in respect of ethical behavior show a gradually increasing tendency as a result of rapid development in science and technology.

Many countries in the world have created their own ethics rules by the various institutions of engineering. Each engineering discipline should adopt by itself to the general ethics rules of engineering. Education of engineering ethics should teach in a school as a compulsory course. During his whole professional life, the engineer must be responsible for application of the ethics rules.

The importance is great to educate engineers knowing rules of engineering ethics, smart and conscious. The basic ethic rules and behavior principles must be kept updated and renewed along with the entire engineering education.

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THE GRAVEL-BED RIVER REACH PROPERTIES ESTIMATION IN BANK SLOPE MODELLING

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***Abstract*—If no bed material data are available, an acceptable estimate of Manning's n may be made by using a simple power expression in which slope is the independent variable. When a simple power form of relationship is used to evaluate the friction factor parameter, $1/(f)^{1/2}$, the relationship between the friction factor parameter and Reynolds number is better than the relationship between the friction factor parameter and relative depth, d/D_g. This finding is at variance with the expected result based on widely accepted ideas related to the simplified case of two-dimensional turbulent flow over a hydrodynamically rough boundary.**

Key words: resistance, hydraulic geometry, gravel-bed rivers.

1. INTRODUCTION

Church and Kellerhals [2] point out the difficulty of adequately characterizing a gravel bed by a single grain size distribution for a relatively long river reach. Bray [1] indicated that the initiation of motion calculations gave as a result in which the gravel bed is immobile or at least not highly mobile at flows by flooding boundary layers. The basic data for each gravel-bed river reach are directly applied to a specified equation to compute the average velocity.

The knowledge about the hydraulic geometric parameters, width, depth and area of the river at the bankful discharge are required for solving a variety of problems related to river training, location of river constructions and navigation. To predict the average velocity of flow, the resistance offered to the flow by the boundary and air-water interface needs to be known. In methods for the prediction of width, depth, area and the flow velocity or resistance coefficient the results of the analysis of the available gravel-bed river data will be given.

2. METHOD

The resistance characteristics and the study of hydraulic geometry for gravel-bed rivers is the main method for finding all the hydraulic characteristics. The hydraulic geometry refers to the geometrical characteristics of the cross-section such as the average width w, average depth h and area A (=wh) at the bankful discharge Q.

The basic data for each gravel-bed river reach are directly applied to a specified equation to compute the average velocity. Then for each reach the percent deviation (PDEV) of the computed average velocity from the "observed" average velocity is computed. The distribution of the percent deviations associated with a specified equation is then determined for the different gravel-bed rivers reaches. A summary of the parameters to describe the distribution of the percent deviations for each of the specified equations is given in Table 1.

Some of the characteristics which differentiate gravel-bed rivers from the alluvial rivers are:

a) much steeper slope (0.001 – 0.02)
b) resistance is higher than the alluvial rivers

There is scope of using all the available gravel-bed river data and develop non dimensional relationships for the hydraulic geometry. In the analysis of river and channel problems it must be given a relationship between the average velocity U, the depth h or the hydraulic radius R, channel slope S and some coefficient which is related to the channel boundary. This is known as the resistance relationship [3]. The work of Lacey [4] about the sand-bed rivers has shown that for such rivers depth h or hydraulic radius $R \sim (Q/f_1)^{1/3}$, width W or wetted perimeter $P \sim Q^{0.50}$, Area $A \sim Q^{5/6}/ f_1^{1/3}$ where f_1 is Lacey's silt factor and is given by $f_1 = 1{,}76\ (d)^{0.5}$, d being the median size of bed material in mm. As regards the gravel-bed rivers Kellerhals and Bray [5] have related W, h, A to Q and sediment size d as

$$W = 2.08\ Q^{0.528}\ d^{-0.70} \quad (1)$$

$$h = 0.256\ Q^{0.331}\ d^{-0.25} \quad (2)$$

Table 1. Statistics for Gravel-bed River Reaches [1].

Equation (1)	Mean (2)	Standard Deviation (3)	Minimum Value (4)	Median Value (5)	Maximum Value (6)
Manning's Eq. n by modified Cowan	-3.3	29.6	-50.0	-7.0	83.2
n by Strickler $n=0.41\ D^{1/6}_{50}$	44.9	43.7	-18.6	31.8	181.9
$n = 0.038\ D^{1/6}_{90}$	37.5	40.9	-23.1	25.0	156.9
n by Limerinos	2.5	28.8	-41.8	-3.1	74.4
Keulegan's Eq.					
$k_3 = D_{50}$	54.2	46.1	-12.7	40.4	195.3
$k_3 = D_{65}$	47.0	42.7	-17.3	35.2	169.2
$k_4 = D_{90}$	32.9	38.3	-23.9	23.0	136.4
Lacey's Eq.	8.6	29.4	-26.6	-0.7	116.1

All such equations are based on the analysis of limited amount of data and are not dimensionally homogeneous. Only the dimensionless parameters W/d, h/d and $U/(\Delta\gamma_s\ d/\rho_f)^{0/5}$ and related them to the dimensionless discharge $Q/\ d^2\ (\Delta\gamma_s\ d/\rho_f)^{0/5}$ are given [3]. Hence $\Delta\gamma_s$ is the difference in specific weights of sediment and water and ρ_f is the mass density of water. Hence there is scope of using all the available gravel-bed river data and develop non-dimensional relationships for the hydraulic geometry. In the analysis of river and channel problems we need also a relationship between the average velocity U, the depth h or the hydraulic radius R, channel slope S and some coefficient which is related to the channel boundary. This is known as the resistance relationship [3]. The resistance relationship is expressed in dimensionless form as [3],

in Manning's Equation : $$\frac{U}{\sqrt{ghS}} = \frac{h^{1/6}}{n\sqrt{g}} \quad (3)$$

in Chezy's Equation : $$\frac{U}{\sqrt{ghS}} = \frac{C}{\sqrt{g}} \quad (4)$$

in Darcy-Weisbach Equation:

$$\frac{U}{\sqrt{ghS}} = \sqrt{\frac{8}{f}} \quad (5)$$

$$\frac{U}{\sqrt{ghS}} = F\left(\frac{h}{d}\right) \quad (6)$$

In the above equations n is Manning's roughness coefficient, C is Chezy's discharge coefficient, f is Darcy-Weisbach resistance coefficient and F is a function. These coefficients depend on the resistance, offered to the flow by the channel boundary and air-water interface [3]. The available data in Turkey at East Black Sea Basin have been analysed in a unified manner to obtain dimensionally homogeneous relationships for W, h, A and U.

3. DATA

A summary of data were classified as bankful discharge and variable discharge. The bankful discharge data were used to study the hydraulic geometry. The variable discharge data pertain to discharges other than the bankful in any stream. In order to study the effect of bed condition , each set of data were further subdivided into those with mobile bed, and those with paved bed. There is no

need to subdivide the data, because both sets of data behaved in similar manner.

4. ANALYSIS OF HYDRAULIC GEOMETRY

The dependent variables can be any two of the four variables average width W, average depth h, area of flow A = Wh and the average velocity U. The independent variable related to the flow is bankful discharge Q. The sediment representing the bed and the banks will be described by the median size of the bed material d, its geometric standard deviation σ_g and the difference in the specific weights of sediment and water $\Delta\gamma_s$ [3]. It is known that for a given stream the channel slope is related to the bankful discharge Q, the slope decreasing as Q increases in the downstream direction [3]. If we deal with the data from different basins, S and Q will not be related and hence S should be taken as an independent variable. If we ignore Q_B, because the gravel-bed rivers carry a small amount of sediment load, we can analyse as [3],

$$W, h, A, U = F(Q, d, \sigma_g, \Delta\gamma_s \rho_f, \mu, S) \quad (7)$$

With simplifications Garde [3] gave the dimensionless relationship for hydraulic geometry of different river basins as,

$$\frac{W}{d}, \frac{h}{d}, \frac{A}{d^2}, \frac{U}{\sqrt{\frac{\Delta\gamma_s}{\rho_f} d}} = F\left[\frac{Q}{d^2\sqrt{\frac{\Delta\gamma_s}{\rho_f} d}}, S\right] \quad (8)$$

If studied regime types of relations, we must plot W, h, and A against Q on log-log scale which yielded straight lines giving equations by [3],

$$W = 4.547\ Q^{0.507} \quad (9)$$

$$h = 0.293\ \ Q^{0.332} \quad (10)$$

$$A = 1.330\ \ Q^{0.839} \quad (11)$$

By comparing this equations, also the North Anatolian River Reaches will be investigated. The exponents of Q obtained in Eqs. [9], [10], [11] are very close to those obtained by Lacey [4]. Similar investigation was carried out using W/d, h/d and A/d^2 and determining their variation with $\frac{W}{d}, \frac{h}{d}, \frac{A}{d^2}, = F\left[\frac{Q}{d^2\sqrt{\frac{\Delta\gamma_s}{\rho_f} d}}\right]$ by plotting on log-log scales [3].

The relationships given by Garde [3] are,

$$W/h = 7.675\ Q^{0.448} \quad (12)$$

$$h/d\ = 0.504\ \ Q^{0.373} \quad (13)$$

$$A/\ d^2 = 3.872\ Q^{0.821} \quad (14)$$

5. METHOD

From www.terrasol.com the program for landslides can be estimated by TALREN 4 which is ideal for checking the stability of natural slopes, cut or fill slopes, earth dams and dikes. It takes into account various types of reinforcements, such as: anchors and soil nails, piles and micropiles, geotextiles and geogrids, steel and polymer strips. There is another new user-friendly graphical interface with:

a) In the program, definition of the profile using a mouse, rulers and a grid. Other features include pop up menus and choice of soil colours.
b) Ability to load background drawings (.jpg and .gif formats) and adjust to scale.
c) Several construction stages and calculation alternatives can be handled in the same file.
d) Tables illustrating main soil, load and reinforcement data.
e) Various output options for graphical display and tables (shadings, forces in reinforcements, detailed results for each failure surface, etc.)
f) Wizards and databases to help produce the best model and choice of input data (partial safety factors).

New calculation functionalities:

a) Automatic search option for circular failure surfaces (no need to define a manual grid).
b) No limit on the number of elements you can define (points, layers, reinforcements, hydraulic mesh, etc.)
c) Future upgrade option for TALREN 4 users: calculation method based on limit analysis theory.
d) TALREN still benefits from extensively used methods as limit equilibrium calculation along potential failure surfaces using the Fellenius, Bishop or perturbations methods.
e) Ability to take into account hydraulic conditions.
f) Seismic loads are taken into account by the pseudostatic method.

6. DATA UNCERTAINTIES

For estimation of landslides condition we require precipitation, stream flow, evapotranspiration and watershed morphology. The effects of data uncertainties must be considered in different ways:

(1) whether the model parameters are determined from calibration or from physical measurements and principles,
(2) whether the model is used to estimate real events (landslide forecasting), or to estimate synthetic events (design storms and generation of synthetic flows which reasoned the landslides. These issues are considered separately.

In Turkey the landslides can be seen in the Karst environment. Karst is a term applied to topography fdrmed in regions of limestone or dolomite bedrock by the vigorous solution work of groundwater. One recognizes karst topography by the presence of large numbers of sinkholes, solution valleys, disappearing streams, and landslides. The development of karst topography is enhanced by the presence of well-jointed carbonates or evaporites near the surface. It is also enhanced by rainfall. And sufficient relief to insure continuous movement of groundwater that will carry away dissolved matter. The term karst comes from a limestone plateau in Yugoslavia where solution features are well developed. Similar topography can be found in Turkey, Kentucky, Tennessee, Indiana, northern Florida, and Puerto Rico.

Types of mass wasting Earth materials on slopes shows the movement where it shows as the result of landslides.

Table 3. Types of mass movement (Levin, 1986).

Flow (movement distributed throughout material)	**Rate of Movement**	**Amount of Water Present**
Creep	Slow	Water not necessary
Rock glacier, rock, stream	Slow	Water not necessary
Solifluction	Fast	Water-saturated
Mud flow, debris flow	Slow or fast	Much water
Earthflow	Slow or fast	Much water
Slide (movement as one Mass on a slip surface)	**Rate of Movement**	**Amount of Water Present**
Debris avalanche	Fast	Wet or dry
Slump	Fast	Wet or dry
Landslide, rockslide	Fast	Wet or dry
Fall (free fall of rock or soil)		

Creep is a small form of land movement where the amount of water is not necessary and its measure is only a few centimeters in one year (Watkins et all., 1975). Creep can decrease if we can follow it through the earth surface and is a form of small earth flow. There are two types of creep, soil and rock creep which can be observed.

7. LANDSLIDE PREVENTION

Simple engineering techniques have been used to prevent the landslide, for example, by flattening the cut-slope angle the landslide movement of erosion can improve in an easy way by construction of infrastructure (Levin, 1986). Meandering environment shows us another way of landslides as an example, polygonal ground on the flood plain of the Kogosukruk River, Alaska. Scale of air photograph 1: 20.000 (Courtesy of U.S. Geological Survey).

8. RELATION FOR MEANDER TORTUOSITY

The relationship between the tortuosity ratio and other parameters can be expressed as

$$LR/LV= f(W, D, S, m) \quad (15)$$

This can be reduced to the dimensionless equation

$$LR/LV=f(W/D, S, m/D) \quad (16)$$

If mean velocity and discharge per length of channel width are assumed as two more relevant parameters, Eq.(3) can further be modified as

$$LR/LV=f(W/D, S, m/D, R, F) \quad (17)$$

in which $R=q/v$= the Reynolds number; $F=V/ (gD)^{1/2}$= the Froude number; q= discharge per unit length width; v= kinematic viscosity; V= mean velocity; and g= gravitational acceleration.

To investigate the actual relationship and its validity, river data or laboratory data for meandering flumes were needed for all the parameters involved. The study of the effect of parameters W/D, S and m/D individually on meander tortuosity, plots of LR/LV against these three parameters indicate that channels with low tortuosity ratio, i.e., more or less straight channels, have wide and shallow cross sections, steeper slopes, and relatively coarser bed material. A value of LR/LV equal to one indicates straight channels. With gradual reduction in the value of all three parameters W/D, S, and m/D, the tortuosity ratio increases, indicating that meanders become more and more acute.

Flow curvature creates superelevation and transverse flow across the section of a channel bend. The strength of the transverse current depends on boundary friction. In wide and shallow channels the ratio of roughness elements to flow depth is higher than in deep channels because of coarser bed material as well as shallower depth, the higher roughness ratio results in more frictional resistance and hence weaker transverse flow than in narrow and deep channels. In considering river patterns, channels can logically be divided into two main groups, single channel streams and multichannel streams, with a transition range between the two. Single channel streams can be further subdivided into meandering channels and straight channels with a transition between them. Meanders can be classified as regular or irregular, simple or compound, acute or flat, and sine, parabolic, circular or sine-generated curves.

Meandering channels are formed if the flow dynamics corresponds with the channel morphology. Braided channels occur if flow dynamics and channel morphology are incompatible. Alluvial channels are unstable because the stability criteria for the channel bed and for the channel banks are different.

Meander flow takes place in one single channel which oscillates more or less regularly with meandering river amplitudes that tend to increase with time. Meanders are found in beds of fine sediment with gentle slopes.

9. MODEL APPLICATION

The results of model applications were carried out for the same situations as the mathematical model at the Technical University of Berlin, Institute Wasserbau and Wasserwirtschaft (Yilmaz, 1990), started with flat bed, continued until $\partial z/\partial t=0$. Then the beds were solidified, and precise measurements of the bed configuration and the velocity were performed. Plan geometries of runs consist of a sine-generated curve and an asymmetrical meander loop, respectively. The latter is derived by a Fourier series analysis on several typical bends. The meso-scale bed configuration in alluvial streams is highly dependent on the width-depth ratio of the channel. The velocity measurements were made with small mechanical current meters fixed to a 1m high frame that rested on the bottom while measuring the lower points on the profile. The frame was suspended at different levels above the bottom to collect the data represented by the higher points. Velocity profiles are plotted semi-logarithmically with the dots representing field data and the smooth lines showing model predictions. The mean velocity was calculated from a fit to the entire data set, not for each profile. Smaller dunes (0.50 m high, 2 m long) were superimposed on the large sand wave. Smith and McLean (1977) estimated the roughness parameter for both the skin friction and the form drag due to the smaller dunes to be 0.141 cm^3. Three different perturbations are recognized :

1. **Alternating bars:** The bed configuration reached an equilibrium state after one hour, and the quantitative and qualitative agreements are given. Sensitivity analyses of each term in Eqs. 2 and 3 into the development of alternating bars were also carried out. The term in the Eq. (3) $\partial v/\partial s$ was found to play the most important role in developing alternating bars.

2. **Braided bars :** The calculated velocity vectors and bed configurations were given after one hour. Divergence and convergence of flow streamlines in a wide straight channel and the meso-scale bed configuration of braided bars can be clearly seen.

3. **No bars:** The calculated velocity vector and bed configuration were given after one hour. The velocity distributions are almost uniform and the bed configuration is two-dimensional with less scour and fill than in the case of alternating bars and braided bars.

Numerical calculations are performed using the hydraulic conditions as listed in Table.3.

10. OBSERVATIONS

If the sediment transport behaves as bed load, the sediment surface at meandering channels will deform into transverse waves. These bed forms can have a variety of scales ranging from ripples through small dunes to fully developed dunes or sand waves. Smith (1970) gave that, under pure bed load transport, a flat sand bed is unstable at all wavelengths to small perturbations in boundary topography so that with sufficient time all infinitesimal undulations will grow in height. His analysis predicts that, for bed features of finite wave number, a fastest growing wave exists only when there is a lag between the boundary shear stress and the sediment transport rate; this is the ripple instability. The tendency for larger bed forms to have a seemingly discrete wavelength distribution, and a wavelength associated with fastest growth, is not explained by such a primitive stability model, so Smith (1970) suggested that wake affects also had to be taken into account.

Table 2. Shear Stress Distribution by landslides at mendering channels experimental set-up.

x/L (Distance)	Elevation from bottom (mm)	Run.1 (τ_{max}= 100 mN/m^2)	Run.2 (τ_{max}= 239 mN/m^2)	Run.3 (τ_{max}= 300 mN/m^2)	Run.4 (τ_{max}= 390 mN/m^2)
0	-5.00	0.63	1.06	1.27	1.5
0.05	-4.00	0.64	1.17	1.32	1.58
0.1	-3.00	0.655	1.30	1.40	1.77
0.15	-2.00	0.84	1.40	1.50	1.89
0.20	-1.00	0.92	1.5	1.67	1.96
0.25	0.00	0.97	1.58	1.78	2.06
0.30	1.00	1.09	1.70	1.93	2.2
0.35	2.00	1.19	1.80	2.00	2.3
0.40	3.00	1.25	1.85	2.08	2.3
0.45	4.00	1.25	1.92	2.1	2.4
0.50	5.00	1.27	1.94	2.1	2.4
0.55	4.00	1.27	1.95	2.1	2.4
0.60	3.00	1.14	1.87	2.0	2.4
0.65	2.00	1.00	1.76	2.0	2.36
0.70	1.00	0.79	1.67	1.97	2.29
0.75	0.00	0.71	1.50	1.87	2.2
0.80	-1.00	1.0	1.58	1.80	2.1
0.85	-2.00	1.0	1.40	1.58	1.9
0.90	-3.00	0.72	1.25	1.48	1.7
0.95	-4.00	0.62	1.10	1.20	1.5
1.00	-5.00	0.556	1.10	1.19	1.4

Table 3. Experimental Condition for Alternating Bars.

Run Number (1)	Width Of Channel B (m) (2)	Size of Bed Material Dx10^{-2} (3)	Average Bed Slope (4)	Flow rate Q x 10^3 (m^3/s) (5)	Average Water Depth h (m) (6)	Froude number (7)
10	0.50	0.30	1/75	1.00	0.70	>1
8	0.50	0.30	1/100	1.25	0.80	”
5	0.50	0.30	1/120	1.15	0.69	”
9	0.50	0.30	1/60	1.20	0.75	”
7	0.50	0.30	1/80	1.00	0.70	”
9	0.50	0.30	1/90	1.10	0.73	”
11	0.50	0.30	1/110	1.20	0.75	”
5	0.50	0.30	1/70	1.00	0.70	”
8	0.50	0.30	1/50	1.10	0.74	”

It was showed that once perturbations are of finite amplitude, the larger stresses at the crests cause the crests to propagate faster than the troughs, thus imparting asymmetrical shapes to the waves. When the asymmetry is strong enough, the flow will separate, which creates a momentum deficit downstream of the wave crest much like that found in the wake of a circular cylinder. At the point of reattachment, the near-bottom velocity and stress are both zero. Downstream from this point, an internal boundary layer must develop beneath the momentum defect, or wake, region. The internal boundary layer adjusts to the velocity of the wake region above it, which increases downstream due to the flux of momentum into the wake from the interior. This produces two competing processes that are critical to determination of the boundary shear stress. They are : accelerating effect of an outward diffusing velocity defect; and the decelerating effect of a thickening boundary layer. In the near-field, spatial acceleration of the fluid in the wake dominates the decelerative effects of the internal boundary layer, but in the mid-field the opposite is true, and the net result is a decrease in the near-bed velocity in this region. In the far-field the boundary layer ultimately engulfs the wake entirely, and the boundary shear stress asymptotically approaches equilibrium. The essential features of this response to separation are preserved over an upsloping surface such as the stoss side of a bed form. Consequently, the resulting maximum in the stress profile has important consequences for both bed deformation and bed-form growth.

11. CONCLUSIONS

The flow resistance in a meander bend is considerably increased due to the form resistance of the patterns about which much is not known. It depends on a number of factors including grain friction, form resistance of two- and three dimensional patterns, skin friction of the non-separated oscillatory component and the sediment transport rate.

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POPULATION ESTIMATION OF ANCIENT CITIES BY USING METHODS OF WATER ENGINEERING

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***Abstract*—Human being has always been curious about populations of ancient cities. It seems this is the conclusion of demand of setting up a direct proportionality between the population and military and economical power of that city. Instead of asking questions like "which population?", "which year's population?", generally a unique population, probably the one reached maximum, is emphasized. Populations of ancient cities are generally exaggerated. It is possible to say searching a powerful city is the main reason to do that. "Size of a theater" is the most common method in population estimation so far. It has been claimed that a definite percentage of number of theater benches are concerned to population. Such approaches can be done considering area of city walls and number of people in unit area in cities which do not have theaters. Through approaches asserted in this study developed using water engineering tools, it is aimed to look into the topic from a different point of view, from water engineering's. Approaches can be summarized as: 1. Assumptions for channel capacities and daily water consumption per capita; 2. The approach based on filling/discharging/re-filling durations of cisterns and/or reservoirs benefiting from precipitation and amount of runoff; 3. The approach based on numbers and sizes of bathes/latrines; 4. The approach based on correlativeness between the main transmission pipe and distribution pipes in the city; 5. The approach based on size of irrigated agricultural lands and amount of bread produced for cities which have water storage structures; 6. The approach based on numbers and capacities of fountains in the city; 7. The approach departs from wastewater channel capacities and reaches to population; 8. The approach related to capacities of ports/wharfs. Some of these approaches have been experienced and applied to some ancient cities. The points related to topic chosen from tens of diploma studies and three Master of Science theses have been discussed in this study.**

Keywords: population, estimation, water supply, ancient, reservoir, cistern, fountain, theater capacity.

1. INTRODUCTION

Estimation of population of ancient cities seems necessary in terms of three main points: 1. Gives traces of military power of the city; 2. Gives traces of commerce, and wealth consequently, of the city; 3. Curiosity of the human being. But which population is this? Probably the maximum population! However, it can be inconvenient to mention about such a population. Because negative impacts of constant wars, migrations and contagious diseases will mislead estimations on population. Population is related to commercial life naturally. Over-population will bring over-production, employment of foreigner workers and probably exports.

It can be more rationalistic to accept the population in a range rather than a unique value for the foregoing reasons. However, it is nearly impossible to estimate the true population (which one?). Classification of the population must be done considering layers of life, such as archaic population, Hellenistic population, Roman population, Byzantium population etc.

The most common approach for population estimation in archeological community can be summarized as "multiply the theater's capacity with a definite coefficient". This coefficient is commonly "1/5". Maybe it will not be realistic to

base assumptions on such approaches since it is evident that theaters are signs of magnificence and power.

In this study, mostly essential knowledge, correlation, philosophy, number and formulae of knowledge of hydraulics, which are directly related to the topic, have been represented in order to guide archeologists who are relatively unfamiliar to this branch of science. Thus, it has been pointed out that it is possible to make an approach to population estimation through this method.

2. PREVIOUS STUDIES

Approaches based on capacities of theaters have got the main share in population estimation. In studies that have been carried out/supervised using water engineering methods in past by the authors of this paper, various methods have been represented about population estimation of ancient cities. Capacities of stadiums can be another data for population estimation. Number of fountains, capacity of each fountain and the area it serves can be another way of approach. However, since any studies have not been carried out for these last ones, only some short traces have been given.

Consideration of contemporary criteria is a must to discuss the assumptions in the methods. These criteria can be changed through definite projections. Studies can be concluded more accurately if some typologies can be established.

Studies carried out/supervised by the authors and studies relating to ancient city of Miletos are the essentials for this paper.

Two references discussing engineering in ancient ages have been supplemented [1,2]. A general study including water engineering and its samples in Anatolia that have to be updated day by day has been supplemented as well [3]. A study which is directly related to the topic and mentioning population estimation in one of its chapters [4] and three master of science theses which include numerous examples of application have been also supplemented [5,6,7]. Another interesting study discussing the relation between production of cereals and the population has been carried out [8]. The study which projects capacities of ports and wharfs can be considered in population estimations can be extended by adding sea transportation structures of ancient era to content [9,10].

3. POPULATION ESTIMATION METHODS FOR ANCIENT CITIES

3.1 Capacity of Theater (Stadium)

Determination of capacity of theater is the widely used method in population estimation so far. It is because there is a theater in every city nearly. It is based on a suggestion that claims a definite population may attend theater and this can be 1 person from a house containing 5 people totally. This means multiplying benches of theater by 5 will give the population. However it is not clear what year's/decade's/age's population it is. Theaters are tools for developing relations between cities beside their role in emphasizing the magnificence. Thus, maybe it is not so rationalistic estimating population of a city from theaters. The same approach can be considered for cities which have stadiums, but it is very rare. It can be more convenient to make an induction by discussing capacities of theaters and stadiums together for the cities that have both theater and stadium.

3.2 Size of the Settlement Area (Area of Interior of City Walls)

As it is known, cities have been situated on higher grounds (acropolis) for reasons of defense, then spread towards lower lands in more powerful and safer times, so they have developed towards plains and plateaus. City walls are thought remain from establishment or decline periods of those cities, the times in which cities are the least powerful. For that reason, population estimation of interior of city walls will be concerning with one of these two periods and not give desired information. Attributing on this approach, for population of the city for its glamorous times, the population estimation can be done by multiplying population per unit area to by development area for definite times.

Maximum 40% percent of a land is reserved as green area according to current Law of Public Works. Coefficient of Storey Area (CSA) and Coefficient of Bottom Area (CBA) at a definite percentage are allocated after expropriating this amount for using as roads, junctions, parks, public buildings, religious areas, etc.

If the same method is implemented to ancient cities, the area reserved for housing can be calculated by a concept: "the remaining area after subtracting a definite percentage area of interior of city walls must be the area of housing". If an assumption is done for number of people in each house, an approach can be obtained for population estimation by this method. As a suggestion, population can be calculated by multiplying averagely 70% of area of interior of city walls by 5 (5 people in each house). This amount has been calculated as 330 capita/ha for Pompeii, 550 capita/ha for Miletos [4].

3.3 Numbers and Sizes of Latrines and Baths

As it is known, both latrines (also visited by visitors, merchants, etc.) and toilets in houses of wealthy people were in use in ancient ages. Current criteria like the ones proposed by Bank of Provinces of Türkiye (BP) and Ministry of Public Works and Settlement of Türkiye for toilets of schools, etc. can be employed in this approach.

3.4 Quantities and Capacities of Cisterns, Wells, Monumental and Street Fountains

Population estimation is possible through numbers, sizes filling/discharging durations of cisterns; through depths, and static water level if there is still water, of wells; through inner volumes, flume heads and location grids of street fountains; through capacities and length of tap outlet pipe in monumental fountains.

For this purpose, water discharge capacity of fountain can be calculated using orifice formula. Ottomans have frequently used the formula given in Eq. (1) in order to standardize water distribution. If it can be determined in what extent the monumental fountains are benefited, a possibility may appear to make approaches to population.

$$Q = c*A*(gh)^{1/2} \quad (1)$$

Q: Discharge of the fountain (m^3/s)
c: Coefficient of discharge (for instance, 0,82 by Ottomans)
A: Wetted cross-sectional area of the orifice outlet (m^2)
h: Water head behind the axis of the orifice outlet (m)

In semi-arid regions like Anatolia, precipitation is much in months of winter and small in months of summer, in a year. These fluctuations have to be considered in the calculation of capacity of water storage. Precipitation has to be stored to meet the urban requirement. The filling-discharging number of existing cisterns can be calculated using the ratio of annual mean precipitation to runoff. In this manner, if the water requirement per capita can be predicted a definite approach can be obtained to population.

Another method is "mass curve", also called "Ripple diagram". Use of this method for urban reservoirs is based on this concept: Total volume of one or more reservoirs of storage has to be in such an extent that the water must not be wasted instead of storing. By this way, if water storage volume, that is not available now but might have existed in past, can be calculated, an approach can be done for population estimation. Water volume input to reservoir is calculated after calculation of precipitation and projection on coefficient of flow. Mass curve values are obtained if cumulative values of water volume input is signed on vertical axis and months are signed on horizontal axis. For a total storage case, sum of successive lacks and surpluses give the active reservoir volume. Thus, volume of the reservoir is obtained as $V_H = |V_1| + |V_2|$ (Fig. 1). The condition $V_1 + V_2 \leq \Sigma V_{cistern}$ must be satisfied.

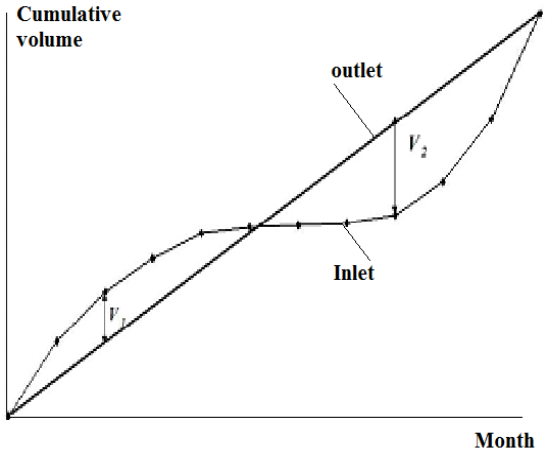

Fig. 1 Mass curve diagram.

3.5 Water Consumption per capita and Capacities of Channels

Physiological water requirement of a human being can be referred in order to form an opinion about water consumption per capita in ancient cities. If sub/super limits are defined for this aim, a population range is obtained instead of a unique population value. Population estimation of that period in history can be approached if wasted, irrigated and domestically used water amount and date of water supply are identified by dividing supplied water to those numbers [5,6,7].

Because the water is the most important component of life, water supply for settlement zones bear a great significance as it is today. However, since there is not any evidence on amount of water consumed per capita in ancient cities, the topic has been tired to be explained through requirement of today's human being.

Another point is existence of fountains. It is thought that water has been supplied to flow constantly. Generally there was not fountains/taps in houses, although some fountains have been met in private houses, such as the ones in Pompeii. A way based on constant flow or releasing water at defined hours must have been followed.

3.5.1 Daily Consumption per capita

Daily average water consumption of a modern man has been defined by BP in Guideline No.3 depending on population of settlement zone (Table 1).

Table 1. Daily consumption per capita.

Population (N)	**Daily Consumption per capita per day (Q_{HI}, l/capita/day)**
N < 3.000	60
3.000 < N < 5.000	60-70
5.000 < N < 10.000	70-80
10.000 < N < 30.000	80-100
30.000 < N < 50.000	100-120
50.000 < N < 100.000	120-170
N > 100.000	Subject to approval of BP*

*Bank of Provinces (BP)

According to Table 1, the daily water consumption per capita increases significantly depending on the concentration of the population. The values in the Table 1 must be multiplied by 1,5 for summer, since they are the average values accepted by BP. Water amount given in l/caipta/day unit in Table 1 has to be converted to discharge using Eq. 2 in order to determine dimensions of pipes. If the average discharge is denoted as Q_{Ho} (l/s)

$$Q_{Ho}=(N*Q_{H1})/86.400 \quad (2)$$

Today, in the determination of water requirement of settlement zones components such as animal-breeding, industry and private consumptions for transmission discharge and fire and network losses for network discharge have to be also considered in addition to daily water consumption per capita.

Water Requirement for Animal-Breeding: Requirement for animal-breeding also bears importance in rural areas beside human's requirement. 50 l/day and 15 l/day have been proposed by BP for bovines and sheep respectively. Eq. 3 shows water requirement for animal-breeding where Q_{animal} is discharge of animal-breeding, N_b is the number of bovines, and N_s is the number of sheep:

$$Q_{animal}=(N_b*50+N_s*15)/86.400 \quad (3)$$

Water Requirement for Industry: Consumption of industrial water indicates notable fluctuations depending on type of industry, methods of production, price of the water, etc. Therefore, industrial plants have to be examined one by one in order to obtain healthful conclusions. If it is not possible to determine the water requirement for industry through these examinations, the values can be accepted as Q_{LIZ}=0,50-0,85 l/s/ha for larger industrial zones and as Q_{SIZ}=0,35-0,50 l/s/ha for smaller industrial zones. Also, 10~20% of the public water requirement can be accepted as water requirement for industry generally. Private Discharges: Projections of private discharges (liter) for ancient ages have been given in Table 2.

Discharge of Transmission: The sum of discharges that must be transmitted to meet human, animal-breeding and private requirements in the city is called discharge of transmission. Private water requirements are defined by BP in Guideline No.3 as well. So, discharge of transmission is calculated by Eqs. 4 and 5.

$$Q_T=Q_{Human}+Q_{Animal}+Q_{Industry}+Q_{Private} \quad (4)$$

$$Q_{Human}=1,5*Q_{Ho} \quad (5)$$

Table 2. Private discharges.

Type of Private Discharge	Water Requirement (liter)
For each bovine butchered at abattoirs	300-400
For each sheep butchered at abattoirs	150-300
To produce fabric from 1 kg of wool	1000
To prepare 100 kg of beet to be treated in factory	1500
To produce 1 kg of sugar	100-150
For each larger leather in tannery	1.000-3.000
Per capita in baths	300-350

Discharge of Fire: Water is generally utilized since it is cheaper although it is not the best material to dissipate the urban fires. It is especially employed for house fires. Discharge of fire gains importance in dimensioning pipes of secondary streets where the consumption is less. Because own discharges of these streets are much less than discharges of fires. The values for discharge of fire which are defined by BP depending on population and types of pipes and given in Table 3 must be taken into consideration in dimensioning pipes.

Table 3. Discharges of fire.

Population (N)	Main Pipe (l/s)	Actual Pipe (l/s)	Secondary Pipe (l/s)
N<10.000	5	5	2,5
10.000<N<50.000	10	5	2,5
50.000<N<100.000	20	10	5

Discharge of Network: Water requirement of the cities fluctuates within the day. Thus, sometimes smaller discharges are delivered, sometimes greater ones. In order to determine this fluctuating requirement, the discharge that is to be transmitted to the city is calculated by using Eq. 6.

$$Q_{Network}=1,5*Q_{Transmission}+Q_{Fire} \quad (6)$$

Because there are no data, neither references for ancient settlement zones, the calculation for water consumption can be based on such an assumption that accepts a lower boundary as 5 l/capita/day which is only enough physiologically and an upper boundary as 20 l/capita/day which is 1/3 of 60 l/capita/day which is accepted as the requirement of today's smaller settlement zones.

3.5.2 Population Estimation of the Cities that are Supplied from Springs through Transmission Line

The daily quantities of water transmitted to cities can be calculated by using Eq. 7 and 8 based on such assumptions as 1) the water which is to be

supplied to the city is between the values Q_{min} and Q_{max} which have been calculated through principles of hydraulics and defined properties of transmission line, 2) there is not any significant storage plants in the city, 3) so, the requirement is to be met from this water.

$$Q_{min}\,(l/day) = N*Q_{H1} \quad (7)$$

$$Q_{max}(l/day) = 1{,}5*N*Q_{H1} \quad (8)$$

where Q_{H1} is daily water consumption per capita and it has been also assumed that water resource is always able to supply the sufficient capacity and the water quantity is limited by the conditions of transmission. Daily water consumption of a person can be calculated through Eq. 9, accepting 5 l/capita/day for minimum and 20 l/capita/day for maximum, where Q_{city} and Q_{person} are in l/day unit.

$$N_{person}=Q_{city}/Q_{person} \quad (9)$$

Minimum and maximum boundaries of calculated population can be determined using Eq.s 10 and 11, where $Q_{maximum,person}$=20 l/capita/day and $Q_{minimum,person}$=5 l/capita/day.

$$N_{min}=Q_{min,city}/Q_{max,person} \quad (10)$$

$$N_{max}=Q_{max,city}/Q_{min,person} \quad (11)$$

This approach is developable to give more accurate conclusions in case such evidences are available like physiological water consumption difference between summer and winter, amount of water that was stored in houses or entire city that is to be determined after archeological studies.

Capacity of the channel has to be determined in order to calculate the water supplied. For normal depth, Manning's formula, given in Eq. 12, is utilizable for this aim.

$$Q = v * A \quad (12)$$

Q (m^3/s): Discharge
v (m/s) : Velocity
A (m^2) : Wetted cross-sectional area

Average open channel flow velocity here can be calculated with Eq. 13.

$$v = \frac{1}{n} * R^{2/3} * J^{1/2} \quad (13)$$

v(m/s): velocity
R(m): Hydraulic radius [R=A/U (wetted area (A) / wetted perimeter (U) [for rectangular channels: (channel bottom*probable water depth (can be projected from water traces) / (channel bottom + 2*water depth)]
n: Manning's roughness coefficient (Table 4)
J: Gradient of energy (can be accepted equal to the slope of the canal's bottom in open canals which has smaller slope between a few in hundred / thousand)

Table 4. Manning's roughness coefficients.

Manning's roughness coefficient	Orderly shaped soil channel	Rubble stones (joints plastered)	Channel graved into rock	Internally plastered channel	Baked clay pipe
n	0,018-0,020	0,017-0,020	0,025-0,035	0,013-0,018	0,014-0,018

3.6 Agricultural Production

If a storage reservoir exists in an ancient city, such as natural lakes, weirs, dams, and an irrigable land is also available, product(s) to be yielded from that land, led by wheat, are utilizable for population estimation by considering definite consumption data (Yaşar 2001). For this aim, the following assumptions may light the way for some calculations:

1 kg of wheat=0,7 kg of flour;
50 kg of flour=320 pieces of bread (200 g each and water, ferment, salt added);
1,5~2,0 tons (1,75 tons averagely) of wheat is yieldable from 1 ha land (hectare=10.000 m^2)
Human's consumption of bread: 1000 g/capita/day.

Irrigation modules in l/s/ha unit can be calculated fitted Blaney-Criddle method [11]. The required water volume and amount of agricultural product to be yielded by this volume can be calculated multiplying required water amount for unit area by total irrigation area. The next step will be calculation of the number of people fed by this amount of product [8].

3.7 Investigation of Sewerage/Wastewater System(s)

A definite amount of water diverted to city returns as wastewater. If channel network of wastewater system is measurable, using/adapting criteria of BP for example, the number of people produced this amount of wastewater can be determined. The water diverted to city in 24 hours is assumed to return to wastewater system in 12 hours by BP.

3.8 Port Capacity and Commercial Volume

Nearly all of ancient coastal cities have a port, or a wharf at least, certainly. Population estimations can be done under definite scenarios by considering port sizes, wharf lengths and ship sizes [9,10].

3.9 Other Population Estimation Methods Except Water Engineering

3.9.1 State Records Based on Allocations

The information in inscriptions and/or state records about amount of bread delivered to public at scarcities may give an idea about population. However it will be very limited information and can be employed typologically only (Tutthas 1998).

3.9.2 Numbers of Ships at Warfare

The population may be approached considering naval wars in history, sizes of ships in these wars, numbers of crews on ships and number of people in their families [4].

4. NUMERICAL EXAMPLES OF APPLICATION

Some of the numerical examples of application about foregoing population estimation methods are available [4,5,6,7,8].

5. CONCLUSIONS AND PROPOSES

The approaches considering size of theater (or stadium if exists), capacity of cistern, water consumption per capita, channel capacity, agricultural production (led by wheat), population of unit settlement area, port capacity and volume of commerce can make explanations for population estimation in a sense. Innovative approaches can be added to them by excerpts from other branches of science under different assumptions. The most important point is making decisions after a reasonably reasoning about conclusions obtained through various approaches. It must be adopted that exaggerated estimations based on today's cities, their settlement areas and populations cannot be true. Since periodical estimations bear significance, it must be always considered what population lived in what era in those cities.

Anatolia is home of a multi-layered cultural heritage. Statistics on Anatolian civilizations will reach a significant level in years by an increase in archeological digs. Then the topic can be concluded more reasonably in terms of population.

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EFFECT OF CLIMATE CHANGE TO THE COLLAPSE OF ANCIENT CIVILIZATIONS

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***Abstract*—War and unrest, and the collapse of many mighty empires, often followed changes in local climes. It seems, this is more than a coincidence. Civilisations have had to adapt or die. Climate changes lead to variation on precipitation, sediment and nutrient transport. As a result of these changes, it is being seen that changes also occur on agricultural-productivity, fishery, water potential and energy. The reason of these changes is rain, sun and wind gods in multi-theistic religions. Factors that control the climate changes over the world surface are volcanoes, meteors, ten-yearly fluctuations on Atlantic and Pacific oceans etc. Declines and collapses of many ancient civilizations overlap with the periods of climate changes. As seen from the data of European-countries, there are strong correlations between the collapse of civilizations and climate changes and, growth on populations and declaration of wars. These clues may give some idea about the periods, duration and intensities of climate changes along the history of humanity. Modern and complex civilizations exhibit relatively high resilience to inter-annual and to decadal droughts. But cultural responses to multi-decadal and to multi-centennial droughts can only be addressed by multi-branch and detailed archaeological and paleo-climatic records. Further studies of the past cultural adaptations to persistent climate change may provide valuable perspective on eventual responses of modern populations to future climate change.**

1. INTRODUCTION

Main reasons of climate change in point of science of climatology are “layer tectonics”, “periodical variations of world’s parameters, i.e. rotational, gyroscopic” and, “sun blasts”, while the anthropological aspect doesn’t accept this mechanism.

More than 1 million microorganisms live in each litre of marine water. These creations who are the primary kettle of the food cycle absorb “C” (greenhouse gas) for constructing their conchs while they release “S” gases (cooling gas) for controlling the temperature of the environment as the greatest balancing elements. The primary production is than formed the CO_2 by absorbing from atmosphere or marine/ocean water to organic compounds through photosynthesis or chemosynthesis.

Examples for global cooling/warming from history of humanity: 1. Noah’s Deluge; 2. Construction of Great Wall of China; 3.Freezing of Greenland; 4. Mayas, Mongols and Crusaders; 4- Construction of İzmir-Aydın/TR railway and cornice (ancient Cordelia) in İzmir/TR; 5. Redirecting of Gediz-delta (ancient Hermos). 6. Redirecting the creeks to prevent the harbors from sedimentation; 7. Changing of costal-lines (blocking of the mouth of ancient harbors); 7- John Steinbeck’s Grape of Wrath (economic depression in 1929’s America).

The main reason on discussions of global warming originated from anthropogenic behaviors: Modern complex societies exhibit marked resilience to inter-annual and/or decadal droughts, but cultural responses to multi-decadal-to-multi-century droughts can only be addressed by integrating detailed archaeological and paleoclimatic records. Four case studies drawn from New and Old World civilizations document societal responses to prolonged drought, including population

dislocations, urban abandonment, and state collapse. Further study of past cultural adaptations to persistent climate change may provide valuable perspective on possible responses of modern societies to future climate change.

2. SCIENCE OF CLIMATE

About 50 million years ago, during the Eocene epoch, some parts of the world were unusually warm. But no one knew why standard computer models can recreate an ancient climate with the right sort of temperature only by giving the planet an atmosphere with a very high concentration of the greenhouse gas carbon dioxide. Unfortunately, there is no geological evidence from rocks formed at that time to suggest that CO_2 levels were so high.

According to science of climate, there are different external impacts that creates earth's climate (Fig. 1). Also the certain movements of the earth is the reason of the nested climate changes (Fig. 2). The other reason is the relationship between temperature and precipitation, as shown in the figures below (Figs. 3 and 4). As long as the temperatures tend to arise, the same tendency is shown in precipitations which cause to climate change.

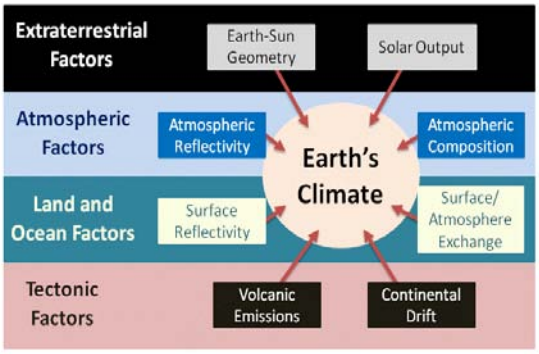


Fig. 1 Effects over the earth climate change [1].

3. CLIMATE CHANGES OVER THE WORLD

A faunal record of sea-surface temperature (SST) variations off West Africa documents a series of abrupt, millennial-scale cooling events, which points out warm period of Holocene phase. These events evidently resulted from increased southward advection of cooler temperate or sub-polar waters to this subtropical location or from enhanced regional upwelling. The most recent of these events was the Little Ice Age, which occurred between 1300 to 1850 A.D., when subtropical Sea Surface Temperature (SST) reduced by 3^{o} to 4^{o}C. These events were synchronous with Holocene changes in sub-polar North Atlantic SSTs, documenting a strong, in-phase link between millennial-scale variations in high- and low-latitude climate during the Holocene.

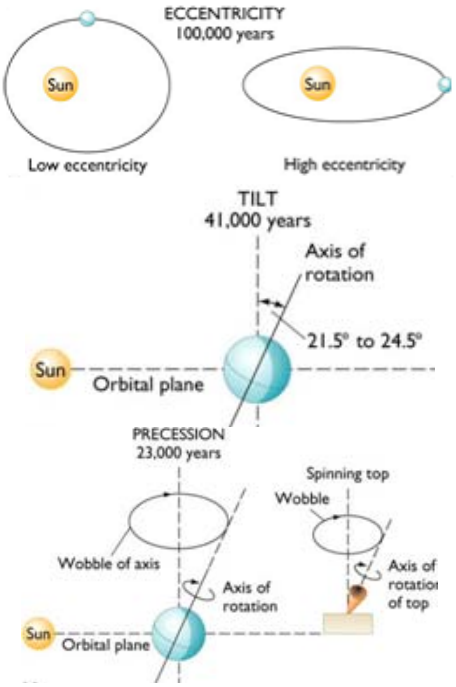


Fig. 2 Earth's movements [1].

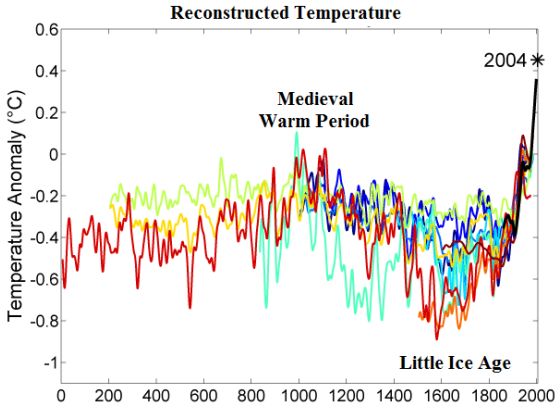


Fig. 3 Changes on average temperature related to 1961-1990 period. Little Ice Age, which occurred between 1300 to 1850 A.D., when subtropical SSTs were reduced by 3^{o} to 4^{o}C.

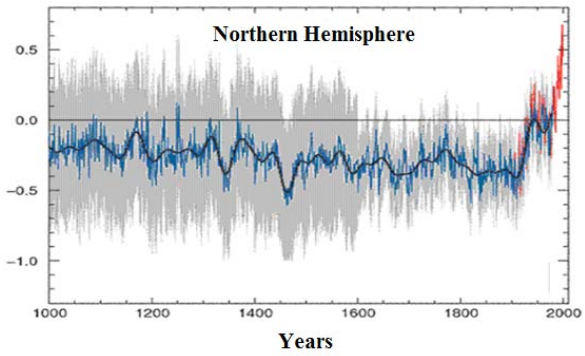


Fig. 4 Precipitations in northern hemisphere [1].

The disintegration of the Classic Maya civilization in the Yucatán Peninsula and Central America was a complex process that occurred over an approximately 200-year interval and involved a catastrophic depopulation of the region. Although it is well established that the civilization collapse coincided with widespread episodes of drought, their nature and severity remain enigmatic. We present a quantitative analysis that offers a coherent interpretation of four of the most detailed paleoclimate records of the event. We conclude that the droughts occurring during the disintegration of the Maya civilization represented up to a 40% reduction in annual precipitation, probably due to a reduction in summer season tropical storm frequency and intensity.

In the anoxic Cariaco Basin of the southern Caribbean, the bulk titanium content of undisturbed sediment reflects variations in riverine input and the hydrological cycle over northern tropical South America. A seasonally resolved record of titanium shows that the collapse of Maya civilization in the Terminal Classic Period occurred during an extended regional dry period, punctuated by more intense multiyear droughts centered at approximately 810, 860, and 910 A.D. These new data suggest that a century-scale decline in rainfall put a general strain on resources in the region, which was then exacerbated by abrupt drought events, contributing to the social stresses that led to the Maya demise.

4. CLIMATE CHANGE IN ANCIENT CIVILIZATIONS

4.1 Examples from the World

Famine is an extreme shortage of food that causes widespread, persistent hunger and death. It is one of the harshest conditions humans must endure. Many famines are initially caused by natural conditions, such as drought, but become real disasters because of overpopulation, mismanagement and frequencies of wars. A general map showing the fall of various civilizations and temperature-based changes in social life is given below (Figs. 5 and 6). As it's seen, the relatively more civilized latitudes (northern hemisphere, 35°-55°) had been affected largely and densely.

Fig. 5 General representation of the collapse of ancient civilizations and climate changes [2].

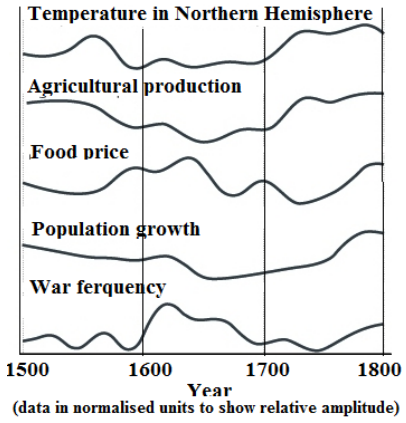


Fig. 6 Parameters in relevant with temperature [2].

4.2 Some Chronological Disasters

4.2.1 Egyptian Old Kingdom

During the time period of 2649–2150 B.C. Egypt's Old Kingdom at the Upper Egypt of today and having a population at Height 2 million had collapsed following of reduced flows in the Nile River. This drought lasted several years. A study of dust deposits from the Kajemarum Oasis in Nigeria revealed a short-lived cold-climate event that corresponds to this period.

4.2.2 The Harappan civilization

During the time period 3000 B.C.-1000 B.C., the Harappans civilization at hight population 100.000 where was located in India, Pakistan, Nepal, and Bangladesh had had an end caused by a decline in rainfall sometime around 1000 B.C. The people were forced to mitigate away from their cities as a result of a gradual drying out of the region. So, ending the sever floods that would have devastated farm activities and, this decline resulted in an agricultural collapse.

4.2.3 Mycenae

Little remains of the city of Mycenae now, but it was the centre of the first great Greek civilization, which thrived between 1600 and 1100 BC (Fig. 7). After this time, many cities were abandoned, trade ceased and their writing system disappeared. Other nearby civilizations, including the Hittites and the New Kingdom of Egypt, also declined around the same time, a phenomenon known as the
Late Bronze-Age Collapse.

Studies of climatic indicators such as stalagmites and sea sediments suggest the Mediterranean cooled at this time, resulting in lower rainfall over the next four centuries. Some researchers think falling food production led to a decline in population and thus to the decline of civilizations in the region [2].

Fig. 7 Mycenaean map [3].

4.2.4 Ancient Rome

Between 27 B.C. and 500 A.D., the fall of Rome with 60 million of population corresponds to a period of climate shift from 250 to 550 A.D. The tree rings indicate that trees were stressed during that time-frame, suggesting an unusually cold or dry periods. These poor growing conditions probably also impacted agriculture.

4.2.5 Chichen Itza

This statue represents the god Chac-Mool in the Mayan city of Chichen Itza, in what is now Mexico. Chichen Itza thrived until the 13th century, and there were still Maya living there when the Spanish arrived in the 16th. However, the heyday of the civilization was between 200 and 800 AD, when the construction of great monuments reached its apogee. After 900 AD, many cities were abandoned (Fig. 8).

Climate records show this decline coincided with a century of lower rainfall that would have dried out open water sources and severely affected food production [2].

Fig. 8 Chichen Itza map on contemporarily placement [4].

4.2.6 Chinese Dynasties

Chinese Dynasties during the time period 200 B.C-1643 A.D with a population at height 60 million and located in Southern China today had collapse as a result of cool temperature periods. These include the Han (206 BC-220), Tang (681-906), Song (960-1279) and Ming (1368-1643) dynasties. Climate fluctuations had interrupted the agricultural food supply, leading to social upheaval and rebellion.

During 4th millennium B.C., the earliest recorded famines occur in ancient Egypt and the Middle East. In 436 B.C., thousands of starving Romans drown themselves in The Tiber River, Italy. The country's second-longest river, it rises in the Tuscan Apennines, and flows south for 400 km, ultimately passing through the city of Rome before entering the Mediterranean at Ostia. Roman emperors will commonly withhold grain from starving peoples as a form of control.

In the 1200-1500 A.D., overpopulation, bad harvests, and epidemic diseases like the Black Plague help cause hundreds of famines in Europe during the Middle Ages in Britain alone. In 1235, some Londoners are reduced to eating tree bark to survive; 20,000 of them die of starvation [5].

4.2.7 Mayans

During the time period 250-900 A.D. the Mayans civilization at highest population of 20 million where was located in Mexico and Central America. The Mayan population grew to such an extent that they utilized all available water resources. It lacked the capacity to survive a severe drought, which is exactly what it faced, sometime around 800-950 A.D. The drought may have led to other problems, such as civil unrest and disease. They eventually contributed to the climate change by cutting the forests [6].

4.2.8 The Moche

Between 300 and 500 AD, the Moche thrived and built cities along the coast of Peru (Fig. 9). But their farmers depended on irrigation canals to grow their crops. By around 600 AD these channels had been buried by sand dunes. The survivors abandoned the coastal cities and moved inland.

Studies of ice cores suggest that an especially intense El Niño cycle around this time produced intense rainfall and floods, followed by a long and severe drought.

Famine also slowed population growth, appearing regularly around the world from the earliest times of recorded history. The Near Eastern and Mediterranean worlds were always susceptible to famine, as indicated by Biblical references [2].

Fig. 9 Map for Moche [7].

4.2.9 Western Roman Empire

The Romans built many aqueducts in southern France near modern-day Nîmes (Fig. 19). At its height, the Roman Empire controlled almost all of Europe, Western Asia and North Africa. But by around 400 AD it had splintered into the western Roman empire, initially centered on Rome, and the eastern empire based in Constantinople (now Istanbul/TR).

Rome was sacked by the Visigoths in 410 AD, the first time the city had fallen in 800 years. By the end of the century the western empire had collapsed. Civil war and institutional corruption contributed to its decline. There have long been suggestions that the climate also played a role and new evidence emerged last year. Records of temperatures and Western Europe over the past 2500 years revealed that between 250 and 550 AD the climate flipped from dry and cool to warm and wet from one decade to the next. Such unpredictable changes are devastating for farmers, and the resulting shortages of food may have contributed to the Empire's collapse [2].

4.2.10 Eastern Roman Empire

Famine was an integral part of Roman history, and closely linked to the final collapse of the Empire (Fig. 10). As its population grew, and environmental destruction limited local productivity, the Empire became dependent on foreign sources of grain. As the Empire contracted - and grain–producing lands in Germany, Egypt, and Britain were lost - Roman authorities were unable to provide the guaranteed distribution of food that had long maintained domestic stability. Between 400 and 800 A.D., the population of the city of Rome fell by over 90 percent, largely because of famine. Large-scale famine also occurred in the Byzantine Empire in 927. Millions died in these events, sometimes reducing local populations by as much as one third or more. Despite these setbacks, however, population continued to grow overall. By 1500, world population had reached an estimated 500 million. It was around this time that the era of western colonial expansion began in earnest, driven by the demands of more people for more resources [8].

Fig. 10 The Roman Empire and Germanic Migration (Eastern and Western Roman Empire) [9].

4.2.11 The Great Wall of China and the Sack of Rome

Growth of rapid population had been always a pressure by forming the human history. The connections are not always obvious, however, because they may be quite slow-acting. Consider how construction of the Great Wall of China ultimately led to the sack of Rome (Fig. 11).

In the third century B.C., a civilization of nomadic warriors flourished in Mongolia, but in the west, they became known as the Huns. As their population increased, the Huns gradually moved south, and began to raid China.

The Chinese responded by building the Great Wall. The Huns continued to raid around the Wall, and the Chinese ultimately had to extend their defenses. They captured and fortified the Tarim Basin all the way to foothills of the Pamir Mountains. Unable to penetrate this barrier, the Huns were forced west into Central Asia.

A few centuries later, another group of nomadic warriors arose in Mongolia, they are Avars. The Avars also raided south, were unable to penetrate the Great Wall, and were forced west.

In Central Asia, the Avars collided with the Huns. The Avars, with a major technological advantage (the stirrup, which makes fighting from horseback much more efficient), defeated the Huns and drove them further west. At the Dnieper River, the Huns collided with the Goths, who were being held out of the Roman Empire by a combination of treaties, bribes, and Roman Legions. When the Huns fell on the Goths, they slaughtered them in great numbers, and drove the survivors across the river into Roman territory [8].

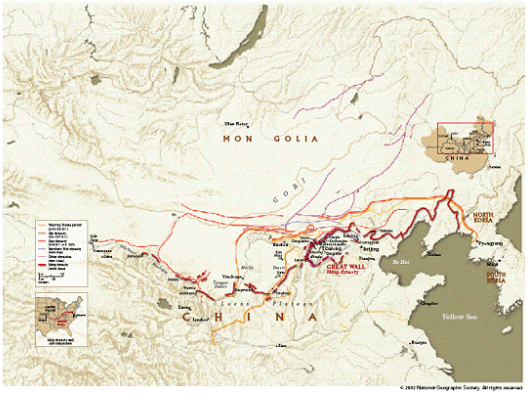


Fig. 11 Great Wall of China [10].

4.2.12 Scandinavians

The first known Viking raid occurred in 793 A.D., and over the next 400 years they discovered and settled the Faroe Islands, Iceland, Greenland which is green and suitable for agriculture at that time, and parts of Newfoundland (Fig. 12).

Moving east and south, the Vikings penetrated the Russia (they named them Russ) and founded city-states such as Kiev and Novgorod. They raided as far south as Constantinople (now Istanbul/TR) and Baghdad, and threatened French cities to the point that they were given huge tracts of land in what is now Normandy ("land of the North men") in exchange for peace. And they conquered much of the richest farmland of Northern England and Ireland [8].

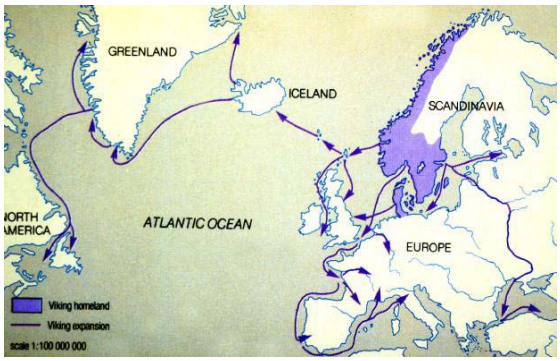


Fig. 12 Scandinavians' expansion [11].

4.2.13 Thirty-Years War

"Thirty Years War", which was continued between 1618 and 1648, was one of the longest and most destructive periods of war in Europe's history. During the 17th century there were many other wars, revolts and conflicts. This period of instability is known as the "General Crisis" (Fig. 13).

The General Crisis is usually attributed to social and economic factors, but some researchers think the real cause was climate change. A cooling of the northern hemisphere led to falling production and rising food prices at the time, causing famine, mass movements and, perhaps, a century of disruption [2].

4.3 Some examples from Anatolia

Major sources of sediments in ocean are terrigenous (or lithogenic, deposits derived from pre-existing rocks); biogenic (remains of once living organisms make up >30% of the deposits); hydrogenic (articles directly precipitated from seawater); Cosmogenic (meteorite fragments from space). Mutual impact between shoreline and ocean, as well as inner waters, may be expected as a result of earth's behaviors.

Sedimentation and shore-line changes because of sea-level fluctuations had been certainly appearances of climate instability in Anatolia (Fig. 14). Within the past of the civilizations that we are still observing their remains, they had taken some precautions or left the land to recover these unexpected occurrences. Some examples are given below (Figs. 15 and 16).

5. CONCLUSION AND PROPOSALS

A sudden decline in water resources is the most significant factor contributing to the collapse of civilizations. If we look at the history of wars, we see always they are from north towards south, from abrupt changeable climates to climatologically stabile countries. Social unrest and political upheaval often disguise the true destabilizing factor: an inability to produce enough food.

Fig. 13 Map of the 30 Years' War [12].

Fig. 14 Coastal-line of Aegean Sea at the end of the last major ice-age (1300 to 1850 A.D., when subtropical sea surface temperatures were reduced by 3^{o-} to 4^{o}C. and today [1]).

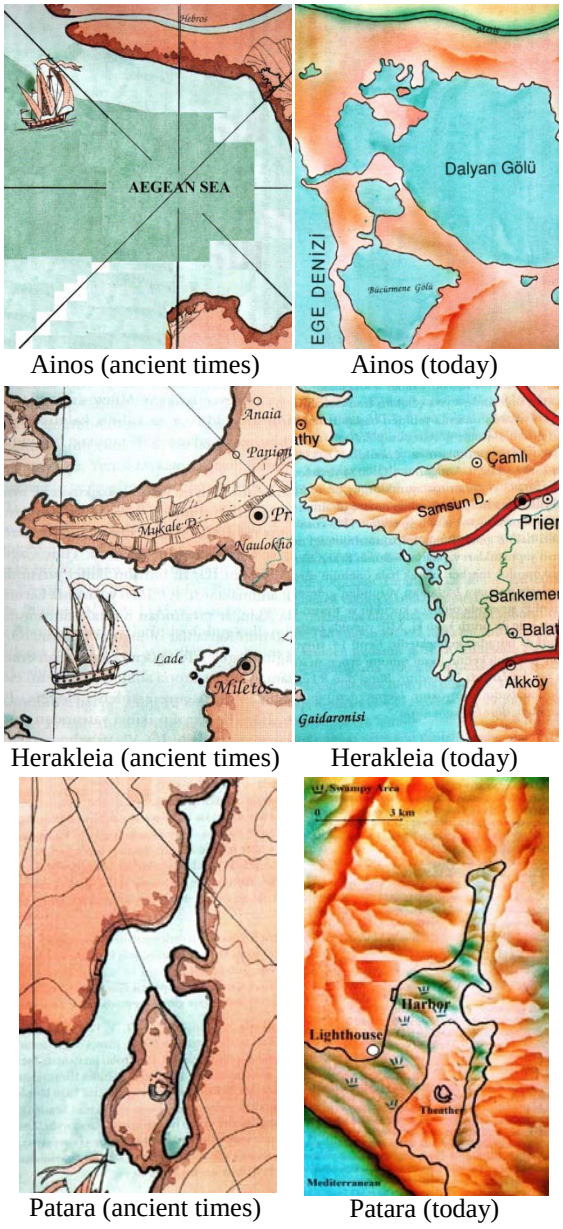
Archaeological and soil-stratigraphic data define the origin, growth, and collapse of ancient civilizations. Synchronous collapse in adjacent regions suggests that the impact of the abrupt climatic change was extensive. For instance, the downfall of the one of the greatest Chinese dynasties may have been catalyzed by severe changes in climate. The same climate changes may have simultaneously led to the end of the Maya civilization depicted.

Remains of ancient rainmaking fires are helping to date droughts in Iron Age. Villagers were also made to burn grain bins when they had planted "unlucky" during harvesting.

With the help of carbon dating and analysis of tree rings it has been discovered and dated previously unknown droughts.

Records of monsoons beyond the last 50 years are difficult to obtain. Looking for signs of monsoon trends in geological records going back 16.000 years helped solve drougth problem. It has been found a startling correlation between climate extremes and the fall of great civilizations. The records show that around the time that these civilizations went into decline, they experienced stronger than average winds in the winter and weaker summer monsoon rains. These weak rains would have reduced crop yields.

Also studying sediments deposited at the bottom of the lakes may provide an accurate historical record of the strength of the winter monsoon winds. The researchers looked at iron and titanium levels in a sediment core that was extracted from the lake floor. The oxidation level of the iron told them how much oxygen was present in the lake waters when the sediments were deposited, and therefore how much wind was stirring up the lake surface. Titanium in particles is non-reactive and the quantities accumulated in the layers of sediment provided another measure of wind strength. When they compared the 16,000 years represented by the mud core, the researchers found that years of strong winter winds corresponded very closely to strong summer rains and vice versa.



Ainos (ancient times) Ainos (today)

Herakleia (ancient times) Herakleia (today)

Patara (ancient times) Patara (today)

Fig. 15 Examples for ancient harbors filled with sediments and/or covered sea-sand [13].

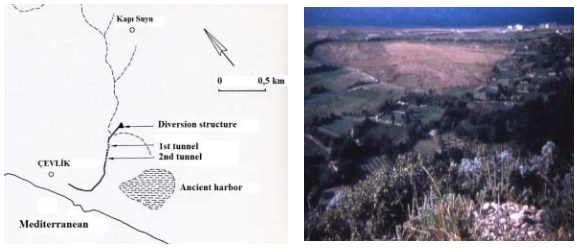


Fig. 16 Harbor-derivation of Seleucia de Pierra (Çevlik/Samandağ/Antakya/TR [14]).

Different from Mycenae and Mayas we have a very big advantage: we know at least what is being coming and how we will challenge with. Mitigation

of the future disasters originated from climate change is in our hands. We can retard it making radical changes in our social life. Unfortunately, we open the way to catastrophes by doing nothing up to now.

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FLOOD ZONING OF V-SECTIONS IN GIS USING HEC-RAS HYDRAULIC MODEL (CASE STUDY: GHALE CHAY RIVER)

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***Abstract* —One of the main methods in planning flood zoning is using non-structural plan. In river engineering, optimal and fast way in planning flood zoning is calculating and preparing profile of water level using math models in HEC-RAS software, and incorporating it with geographical information system of ARC-VIEW and HEC-Geo RAS. Here, first HEC-GeoRAS of river morphology is used, and cross sectional data were studied in ARC-VIEW. After collecting data to stimulate the model, flow hydraulic enters in HEC-RAS environment. In this research, a range of Ghale Chay river was used to prepare water level profile in HEC-RAS, by introducing maximum flood peak discharge with return period of 25, 50 and 100 years in constant flow. Results of HEC-RAS were transferred to ARC-VIEW, flood zone plan was prepared and flood management was determined. In V-form sections, flood zone and its changes are low in width and high in depth; therefore, according to results, velocity and depth of main variant flow is in comparison with return period of 20, 50 and 100 years. In V-sections, high flow velocity creates a very high power of destruction in coasts. Hence, flood management in these sections should be considered more important than sections with extended flood plain.**

Keywords: zoning plans, flood, flood return period, V-section.

1. INTRODUCTION

Annual statistic of natural disasters in the world represents that 32% of natural crisis is caused by flood. In other word, the first place of damaging natural disaster is devoted to flood. Increasing financial and physical damage of flood, time consuming and relative handy calculation required hydraulic engineers and other experts to find a new solution to control and manage this natural phenomenon using modern technologies and science. One of the main objectives of flood plain management is controlling flood and evaluating flood damages. To achieve this objective, hydrological studies and hydraulic analysis of water are required; various models have determining role in this context.

To zone the flood, Eftekhari et al. have changed manning roughness coefficient and stimulated Atrack River in HEC-RAS, and found that increasing roughness coefficient will increase flood zone [1].

ARC-VIEW or geographical information system plays very significant role in coordinating hydraulic model and finding relation between flood zoning information and their physical situation on earth. One of the advantages of using this system in hydraulic modeling is its high efficiency in extracting digital information of cross section from digital elevation model (DEM). Moreover, GIS not only extracts this information, but coordinates information in HEC-RAS and transfer HEC-RAS output as processed data to GIS to prepare zoning plans using HEC-Geo RAS. Digital model has a specific structure when in vector, called Triangular irregular network (TIN) [2].

By stimulating water level profiles in HEC-RAS software, Kaya et al. present flood management ways in Bornova valley [3]. In another

research, Kaya et al. investigate water level profiles in Boustanly valley using HEC-RAS software [4]. Daneshfaraz et al. (2008) studies organizing Deir Ali Chay stream using water level profile. They also investigate zoning and controlling flood of Mehraneh river of Tabriz in 2010 [5, 6]. By integrating hydraulic model and geographic information system, Rostami et al. (2005) study risk acceptance of flood plain of rivers [7].

In this research, flood zoning of V-section Ghale Chay River with return period of 25, 50 and 100 years was determined by integrating HEC-RAS hydraulic model and ARC-GIS.

2. SPECIFICATION OF WATERWAYS AND CATCHMENT BASIN OF GHALE CHAY RIVER

Basin of Ghale Chay River is considered as a part of Orumiyeh River. This catchment basin in placed in upstream in 37 degree and 30 minutes latitude to north 37 degree and 43 minutes, and 46 degree and 5 minutes longitude to 46 degree and 21 minutes. Table 1 shows more information about Ghale Chay.

Table 1. Specification of waterways and catchment basin of Ghale Chay River.

Maximum height of basin	3421 m
Minimum height of basin	1270 m
Impure Slope	9.5%
Average slope	3.6%
Average slope of basin	33%

3. MATERIAL AND METHODS

In this paper, HEC-RAS model is used to determine flood zone in V-form part of river. To acquire topographic information about sections, HEC-Geo-RAS was used in ARC-VIEW environment. After transferring topographic plan of earth to three-dimensional points and transferring to ARC-VIEW environment, section layers, central line layer, and coast layer were determined. HEC-Geo-RAS produces geographical file useful in HEC-RAS model. HEC-Geo-RAS can also arrange required data to prepare hydraulic model in HEC-RAS using SHP files of cross section layers, central line layer, and coast layer in ARC-VIEW environment. Layers made by these data are called RAS layers. Geographic information is acquired from calculations performed on these layers. Water level profile and output data of stimulation in HEC-RAS transfers to ARC-VIEW environment to plan flood zone using HEC-Geo-RAS. Studying water TIN and topography TIN in ARC-VIEW environment prepares flood zone. This process is as follow:

1) Processing topographic plan
2) Transferring topographic plan to three dimensional plan
3) Preparing digital elevation model in ARC-GIS
4) Preparing central line of flow in ARC-GIS
5) Preparing flow way layer and coast layer in ARC-GIS
6) Preparing cross-sectional layer in ARC-GIS
7) Transferring data to HEC-RAS
8) Processing data in HEC-RAS
9) Transferring data to ARC-GIS
10) Providing flood zoning plans

3.1 Preparing Digital Elevated Model

Extracting cross sectional data using topographic maps in ARC-VIEW environment is done through HEC-Geo RAS. Digital topographical model is prepared in TIN and is base of all geographical information extracted from this model. Complete accuracy is needed in its preparation.

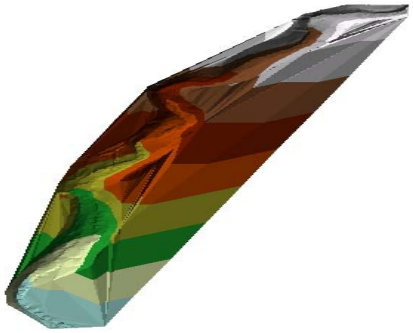


Fig. 1 Under study DTM in TIN format.

3.2 Preparing HEC-RAS Input File

After DTM and TIN, HEC-RAS input file is prepared using HEC-Geo RAS in ARC-VIEW environment. The layers include:

1) Central line of flow layer
2) Coastal layer
3) Cross sectional layer

First, central line of flow layer is prepared from upstream to downstream, then, coastal layers and finally cross sectional layers that show placement and interval of cross sections (Fig. 2).

3.3 Hydraulic Modelling Of Flow

In this step, RAS layers are transferred to HEC-RAS environment using HEC-Geo RAS. After completing processing data, water level profile is determined. Peak flow of the river is one of the parameters required for stimulation. In this regard, discharge with return period of 25, 50 and 100, use data of nearest hydrometer station to determine the range. To evaluate discharge peak of station, SCS method was used. Calculated flood volume is shown in Table 3.

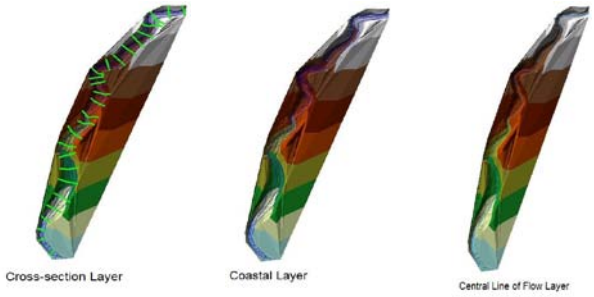


Fig. 2 Cross-section layer, coastal layer, central line of flow layer.

Table 3. Flood with Various Return Period Using SCS Method.

Return period	2	5	10	20	50	100	500
flood m^3 s	56	106	145	187	246	294	514

In this research, considering general situation and geology of river and using Kaon method, manning coefficient of main canal was 0.028 and coasts was 0.01. Waterway slope is defined as boundary condition in first and last interval, and expansion and contraction coefficient was determined based on HEC-RAS software tables [11, 12].

3.4 Determining Flood Zone

After preparing water level profile in HEC-RAS environment and transferring data to ARC-VIEW environment using HEC-Geo RAS, TIN layer of water is made and flood zone is determined by return period of 25, 50 and 100.

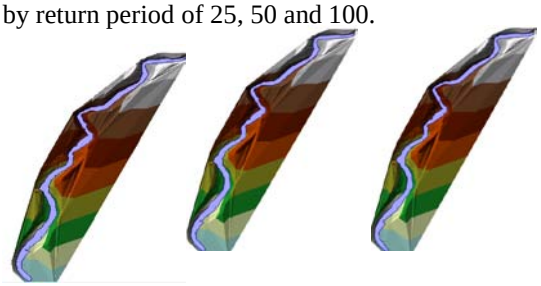
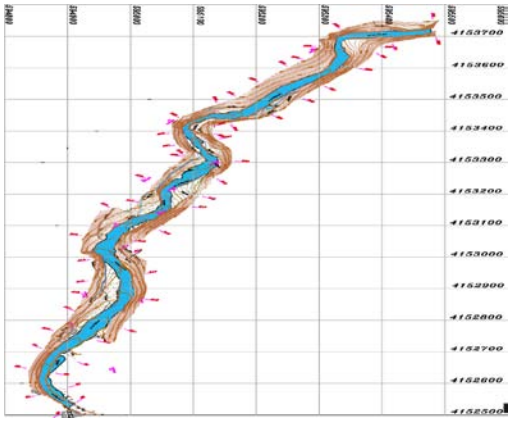


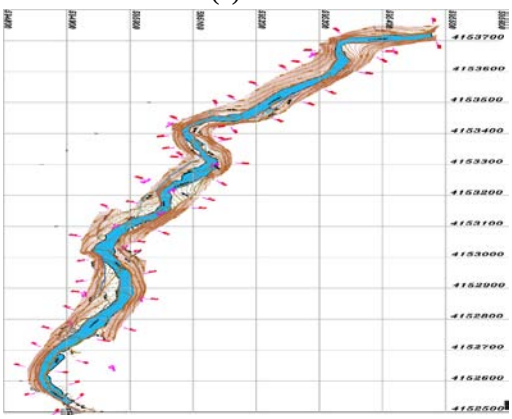
Fig. 3 Flood zone with return period of 20, 50 and 100 years (respectively).

Planning zoning using this method was common and incorporation of the software produced three dimensional, consolidated and practical tools. In this research, central line of flow, coastal line and cross sectional layer was drawn by Shap files in CAD and transferred to ARC-GIS. If the layers were drawn in HEC-RAS without using topographic plans, accuracy of work will decrease; the result found in this research. Preparing cross sections and geographic information was through coordinated topographic plans. Hence, flood zone is transferrable to early map with main coordinates. This step is done through AutoCAD software for return period of 20, 50 and 100 years:

Coordinated range of under study area			
X	594800	~	595800
Y	4152500	~	4153700



(a)



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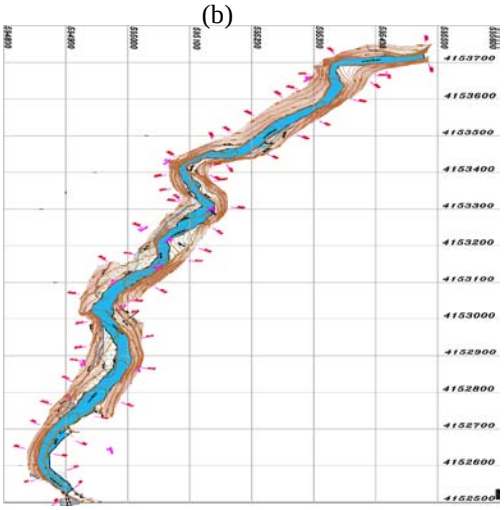

(c)

Fig. 4 Flood zone in coordinated topographic plan with return period of (a) 20, (b) 50 and (c) 100 years.

In case study, due to V-form of sections, it was observed that change of flood zone is less in width and more in depth of flow. It is obvious in Figs. 5 and 6.

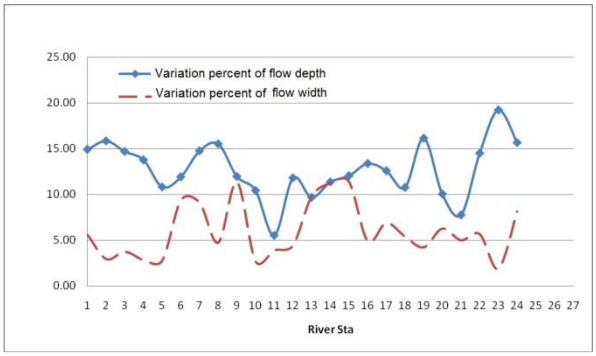

Fig. 5 percentage of depth changes and width of water level in various sections with return period of 20 and 50.

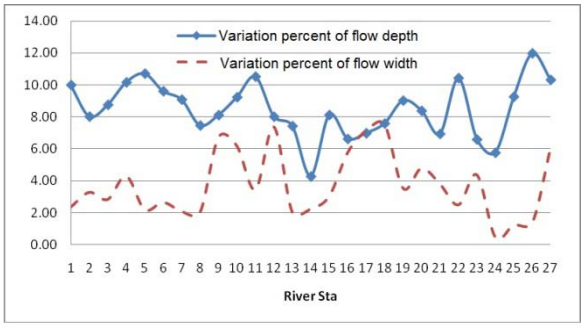

Fig. 6 Percentage of depth changes and width of water level in various sections with return period of 50 and 100.

4. RESULTS

Flood management in river engineering requires incorporation of tools to offer new scientific solutions. In this context, using mathematic modeling is inevitable. Coordinating these data and producing three dimensional flood zone necessitates using ARC-VIEW software or geographical information system due to the fact that flood is a three dimensional phenomenon and incorporated tool of HEC-RAS and ARC-VIEW using HEC-Geo RAS is a completely three dimensional, accurate and economic tool. This method can be used to automatically plan zoning and process a range of data as input in this set and make a link between them.

Assuming constant flow, hydraulic model of HEC-RAS perform calculation. Shortening the distance between sections, interpolation and entering discharge in upstream and downstream, and expertise software in hydraulic and geographic information makes the method very ideal, results of which can be considered accurate. In case study of Ghale Chay, due to V-form topographic sections, flood zone changes scatter less in width and more in depth, therefore, according to results, velocity and depth of main variant flow is in comparison with return period of 20, 50 and 100 years. In these sections, damage in coasts is more and these sections should be noticed more than flood plain sections.

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ENVIRONMENTAL WATER AND TURKEY'S APPLICATIONS

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***Abstract*-The most important parameters to be decided on the design of hydroelectric power plants (HEPP) are head and discharge. Head in HEPP's that convert the potential energy of water into kinetic energy than to electrical energy is restricted as it is directly related to topography. The choice of project discharge based on installed capacity is determined by economic analyses. These analyses generally favor the design which is dimentioned with the available water found in a little percent of time as the most profitable choice. This preference which is completely based on monetary benefit does not take environmental factors of HEPP into consideration. Designing projects only considering maximizing monetary benefits and not considering environmental relations causes environmental problems or causes cancellation of the project on the condition that environmental factors are not taken into consideration. Though HEPPs are known to be the most environmentalist energy producers, thermal and nuclear power plants which are polluters and risky are sometime preferred due to insufficient analysis. In this study, methods which are used for determining the amount of environmental water to be released without power production which is often issue for discussions The methods of environmental water flow calculation depend on countries specific conditions. In the practice including General Director of State Hydraulic Authority (GDSHA) approach, different methods have displayed ten times differences in results. In Turkey's practice, the environmental water value of six different homogenous stations has been studied and it has been shown that all the methods have displayed different results. Also the effect of choice of project discharge on environmental water for a potential power plant has been focused in the study and a new method has been offered.**

Keywords: flow-duration curve, environmental water, minimum discharge, natural balance, project discharge.

1. INTRODUCTION

Hydroelectric power stations which can be evaluated in two main branches as with storage and natural flow plants are one of the most important renewable energy producers Storage hydroelectric power plants, the flow which is random variable in time context is organized by storing and more energy is produced with this organization. 'Storage' term is being used for seasonal regulations. In natural flow hydroelectric power plants, owing to non existence of storage and head's being a stable value based on the physical(topographical) factors where the plant has been built, electric energy quantity that is to be produced from the plant becomes the function of the discharge that is there that moment in the river. These kinds of plants which contribute to the base load do not have the ability to increase energy production to satisfy the increasing energy demand.

In an area where a non-storage HEPP Project is to be planned, superficial water which is diverted into transmission canal is first directed to penstock, then transferred to the power plant and fed to hydraulic turbines. After its potential energy is converted into electrical energy, flow is returned to stream bed. In this case, amount of water which is lost in stream bed is approximately the flow of transmission canal. As a result of this, danger for living things which lives in the river and its beds emerges and plant and animal species partly or completely face extinction. To prevent this problem, 'environmental water' which can be regarded as a mistakable term is placed in stream bed without producing energy with the purpose of

keeping habitat alive and protecting nature and environment to provide the continuousness of species lives.

The most important problem in identifiying the environmental flow requirement is making a decision about the allowable extent of modifications in the river's natural regime. In this respect, more than 207 different methods have been developed in 44 countries within six different parts of the world [1]

In most countries, ecological balance and basin methods which are based on river velocity, depth and also habitat and hydrologic based methods are used to determine the discharge which is to be allocated to downstream. Some hydrologic methods are Tennant, Modified Montana, Lanser (Austria), Alarm Threshold Value (Czech Republic), Linearized Matthey, Carinthia Marine Search Method, Sawall and Simon (Germany-East Germany), Baden-Württemberg (Germany), Portugal Government hypothesis. Another methods gives importance on river velocity and depth. This method consists of Steiermark, Kamten method, Miksch Equality Method, (East Germany), Oregon Method(the USA) and Upper Austria Method. Apart from these methods, there are also ecological balance, basin and habitat based methods. Examples of these methods are Habitat Intersection Model (Germany), Rarefaction Method – Czech Republic (which is in fact habitat based but regarded as ecological balance based in literature context) and another ecological balance based method which is Basin Quality Index (Italy). Italian Method can also be counted as basin based method. All these methods have been studied according to the special conditions of the countries.

Although the number of resources concerning the topic is limited, resources give summaries of past studies, but basically Environmental Group Study is very comprehensive as it gives general summary of countries' general approaches [2].

Also in literature, the other well-known methods are the Tenant Method [3], the Flow Duration Curve Analysis (FDCA), and the 7Q10 Method [4]. Also there are some new efforts to show the importance of saving the full hydrologic regime of rivers after alteration. Botter et al. [5], emphasizing on saving the natural regime of rivers, could simulate the past natural hydrograph of some altered rivers in a catchment in Italy. Hydraulic Habitat Modeling has been developed to cover this problem [1].

The other way to calculate environmental water is calculating the wetted parameter.There is a direct relationship between wetted perimeter and usable aquatic environment [6,7].

It is very important to determine the proportion of water which is placed in downstream properly. Unsuitable decisions either destroy natural life or cause the cancellation of HEPP projects which are the most environmentalist production projects that reduce environmental damages. Today, getting clean energy resources is very important as there is an increasing energy demand, but defective designs prevent the projects from actualization.

In the study, summaries above mentioned methods that have been used to determine the discharge in question have been mentioned. Besides, it has been emphasized that the amount of environmental water on different combinations of project discharges on six different homogenous rivers display alterations. In the lights of the results, a method has been suggested for the process of choosing project discharge.

2. HYDROLOGIC BASED METHODS

2.1 General Overall

Environmental water quantity that is determined and offered by GDSHA is at least ten percent of average discharge for ten years. To calculate environmental water quantity, there are empirical approaches which are based on different essentials in different countries. There are hydrological, basin, water velocity and depth, habitat based methods to calculate environmental water.

To implement some hydrological based methods, flow duration curve has been drawn for the last ten years by using daily data and by this way, requested value is acquired via this curve.

2.2 Montana-Modified Tennant Method (USA)

This method is one of the methods which are often used. It is implemented by taking certain percent of average flow quantity which is calculated according to flow observations through long years. Tennant D.L.(1976). In river which is important for economy in terms of fishing, this percentage rate is between 40 and 60 percent. However, it can be said that this rate is 10 percent if there is no such economic importance for fishing.

2.3 Lanser Method (Austria)

According to this method, environmental water differs between 5 and 10 percent of flow average which has been taken through long years.

2.4 Alarm Limit Value (Czech Republic)

According to this method, required flow quantity for the continuation of ecological functions is $\%20*Q_{300.}$

2.5 Linearized Matthey Method (Czech Republic)

According to this method, if it is bigger than Q_{300}, 100 l/s, the lowest discharge is found with the formula of $RF=0,25*Q_{300}+75$ l/s.

2.6 Sawall and Simon Method (Old East Germany)

According to this method, low discharge is taken between 70 and 100 percent of flow quantity in August.

2.7 Carinthia Marine Surveys Institute Method

According to this method, after classifying monthly flow averages as groups, a value between 10 and 15 percent of the lowest value in the group is left as the lowest flow value.

2.8 Baden-Württemberg Method (Germany)

In this method, 33 percent of the lowest monthly flow is taken as the lowest flow value.

2.9 Portugal Method

Portugal government proposes 1/10 of the yearly average flow as the lowest discharge.

3. RIVER VELOCITY AND DEPTH BASED MINIMUM DISCHARGE METHODS

3.1 Steiermark- Karnten Method (Austria)

According to this method, flow speed differs between 0,3 and 0,5 m/s and the lowest flow value is the value which does not drop the depth under 10 cm.

3.2 Miksch (Sawall-Simon-Bai Germany, Old East Germany)

According to this method, the lowest flow quantity allocated for river's unit width is perceived as 30-40 l/s.

4. HABITAT BASED METHODS

There are a lot of methods for calculating environmental water. One of these methods is Diagnostic Model Method (Germany) (DVWK, 1999) [1]. This model defines the lowest ecological water nourishment and economical energy values. 'vital water' quantity is not to exceed the discharge which is needed for the lowest ecological balance and yearly flow average is supposed to be 4 percent of economical energy threshold value. Annual flow average is to be taken between 5 and 12 percent.

5. MULTIPARAMETER AND ENVIRONMENTALIST METHODS

5.1 Overall

These methods aim at protecting many parameters concerning water such as water pollution and heat for the benefit of ecology. The most important two of them are Czechoslovakia Rarefaction Method and Basin Quality Index Method and these two are to be applied in some circumstances.

5.2 Czech Republic Rarefaction Method

This method asserts leaving the lowest flow that can ten times rarefy concentration of elements and roots which are dangerous for ecological life and environment (PO4, N02, NO3,As, Sb, Cd and so on).This implementation is very useful for rivers which carry household and industrial waste.

5.3 Basin Quality Index Method (Italy)

There are two methods in this approach:1. 7 or 9 percent of average flow quantity which has been measure through long term 2. 50 -70 percent of Q355 are taken.

6. GDSHA APPROACH (HYDROLOGICAL BASIN BASED)

For today, GDSHA demands ten percent of last ten years' average flow to be taken for small HEPP projects as the lowest flow value according to Montana or Modified Tennant methods. Not taking rivers' underground-surface water relation into consideration, it is demanded that for the whole year a stable discharge should be left in the same quantity. However, a river has a regime that has been formed for thousands of years and the discharge which is suitable for the river's regime should be taken into account. The flow course line of a typical river is shown below (Fig. 1, İstanbul 2. International Water Symposium, presentation). Leaving discharge suited for this will not disturb regime and loss of property and death will be diminished as overflowing damage will be fading.

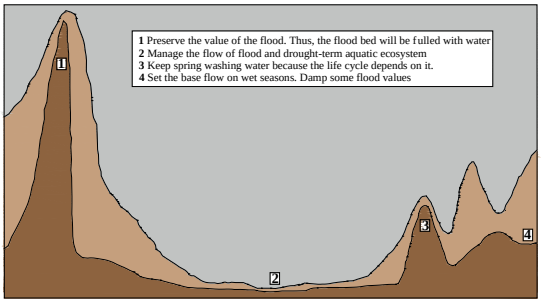


Fig. 1 A typical observed annual and reserved flows.

7. TURKEY'S APPLICATIONS

7.1 General

When determining rivers which will be studied on, their being of the same kind in terms of flow values (natural flow's not being spoiled) and their having stations which have had data through long years have been taken into account and also the chosen stations have been presented with the characteristics in Table 1.

Table 1. Stages in Turkey and their characteristics.

Station Number	**03-04**	**21-186**	**23-01**
Station Name	Osmanlar	Savran	Berta
River Name	Simav	Göksu	Berta
Basin	Susurluk	Fırat	Çoruh
Coordinates	28,33 E 39,25 N	37,80 E 37,53 N	41,981 E 41,173 N
Drainage Area	1254	713,5	1680
Mean Flow	7,45	12,78	25,66
Obsv. Years **Used For FDC**	1992 2001	1990 2001	1990 2001
Carstic Feature	Non-carstic	Non-carstic	Non-carstic
Station Number	**10-27**	**17-07**	**19-14**
Station Name	Suludere	Keşbükü	Mutluçukur
River Name	Bügdüz	Pamuk	Deliçay
Basin	Burdur	Doğu Akdeniz	Hatay
Coordinates	30,179 E 37,651 N	34,768 E 37,030 N	36,251 E 36,888 N
Drainage Area	214,2	599	161,5
Mean Flow	1,42	13,36	4,72
Obsv. Years **Used For FDC**	1983 2001	1986 2001	1990 2001
Carstic Feature	Carstic	Carstic	Carstic

Flow-duration curves which have been prepared by using last ten years' values in connection with GDSHA's suggestion have been determined whereas average monthly discharges of related stations have been presented in table 2.

For a potential HEPP which will be built in station areas, minimum and maximum discharges which have been calculated with different methods are presented in Table 3.

Table 2. Average monthly flows of studied stages.

Monthly Mean Flows of Turkey Stations						
Months	**10-27**	**17-07**	**19-14**	**03-34**	**21-186**	**23-01**
Oct.	0.56	3.29	2.03	1.13	4.81	11.84
Nov.	0.66	4.85	2.19	2.35	6.4	13.36
Dec.	0.76	7.79	6.86	6.8	10.77	10.8
Jan.	0.98	9.84	7.07	14.05	11.9	9.46
Feb.	1.45	12.53	10.37	19.76	12.77	9.47
Mar.	2.63	17.87	6.24	17.04	24.29	21.68
Apr.	3.08	31.36	7.55	17.14	33.27	63.68
May	2.73	34.46	5.89	6.96	19.34	84.89
Jun.	2.25	22.56	3.28	2.3	10.78	49.39
Jul.	0.9	8.14	2.21	0.86	7.66	17.04
Aug.	0.55	4.41	1.79	0.44	6.34	7.55
Sep.	0.53	3.4	1.63	0.6	5.21	8.3

As seen form the Table, there are up to 10 times differences between methods. In Turkey, in the last years private sector gives importance to hydroelectric power stations and it has been planned that small or big 3000 HEPP will be built. In these HEPPs, discharge choice based on installed power is being done with economical profit analysis. And these economical analyses have been mostly calculated with the belief that the plant which is built with the water that is found in the stream between 10 and 20 percent of time will bring maximum profit. However, in case designs which are built according to the discharge that can be observed in 30-70 days of the year come true, the stream is doomed to minimum water which is 10 percent of average discharge and environmental water for nine months.

Table 3. Applied methods and ranges of reserved flows.

Method	**10-27**		**17-07**		**19-14**	
	Min	**Max**	**Min**	**Max**	**Min**	**Max**
Modified Montana	0.11	0.42	1.11	4.43	0.41	1.62
Lanser	0.05	0.11	0.55	1.11	0.2	0.41
Alarm Limit Value	0.11	0.11	0.79	0.79	0.35	0.35
Matthey	0.21	0.21	1.06	1.06	0.51	0.51
Sawall and Simon	0.04	0.55	0.31	4.41	0.13	1.79
Karinthia	0.05	0.08	0.33	0.49	0.16	0.25
Baden-Württemberg	0.18	0.18	1.45	1.45	0.59	0.59
Portugal	0.11	0.11	1.11	1.11	0.41	0.41
Habitat	0.05	0.13	0.55	1.33	0.2	0.49
DSİ	0.11	0.11	1.11	1.11	0.41	0.41
Method	**03-34**		**21-186**		**23-01**	
	Min	**Max**	**Min**	**Max**	**Min**	**Max**
Modified Montana	0.6	2.4	1.29	5.15	2.68	10.7
Lanser	0.3	0.6	0.64	1.29	1.34	2.68
Alarm Limit Value	0.13	0.13	1.1	1.1	1.67	1.67
Matthey	0.24	0.24	1.45	1.45	2.16	2.16
Sawall and Simon	0.03	0.44	0.44	6.34	0.53	7.55
Karinthia	0.04	0.07	0.48	0.72	0.76	1.13
Baden-Württemberg	0.15	0.15	2.09	2.09	2.49	2.49
Portugal	0.6	0.6	1.29	1.29	2.68	2.68
Habitat	0.3	0.72	0.64	1.55	1.34	3.21
DSİ	0.6	0.6	1.29	1.29	2.68	2.68

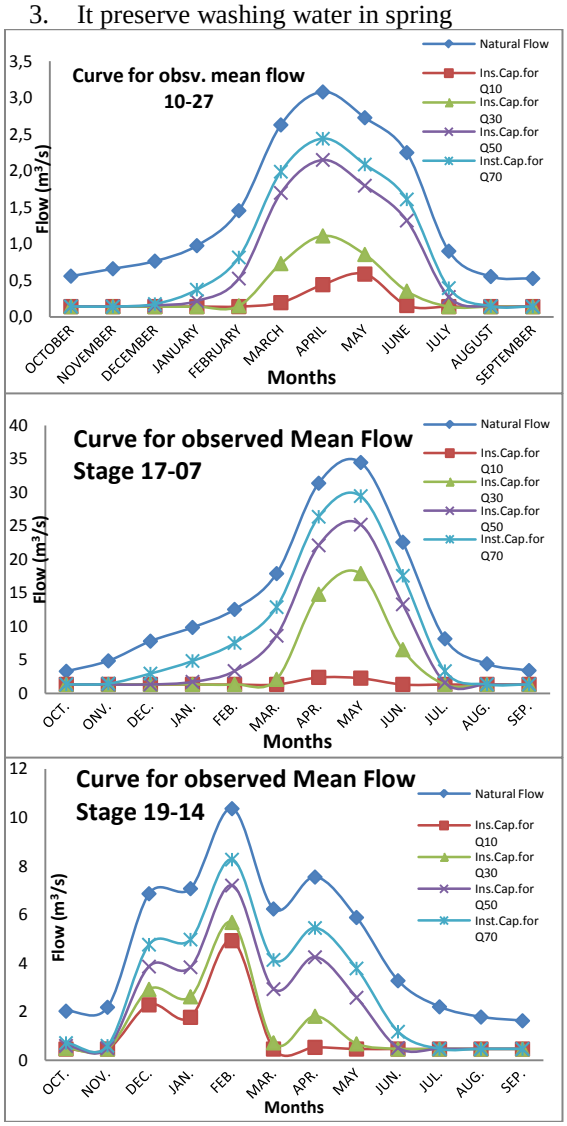
MQ : Average flow of long term
Q_{300} : Flow rate exceeding 300 days of duration
NMQ_{Aug} : Minimum mean flow in August
MMQ : Minimum value of monthly mean flows

In the study which has been carried out in Turkey in six different stations, besides the studies above showing natural flow which is drawn with monthly average discharges, flows which will pass from the stream in case of choosing 10, 30, 50-70 percent of the time of the installed power of a potential HEPP which may be designed in this axis according to the existing discharge have been presented in Fig. 2.

As it can be understood from the image, concerning discharge that is based on design, the discharge which is in 50 percent of time in the river (past studies accept that discharge which is under 50 percent of time may be the basic for discharge for the installed power) protect river regime better compared to the discharge in 10 percent of time which is potentially easy to be exceeded. Principals presented in Fig. 1 are now preserved.

According to this:

1. It preserves the values of the flood. Thus, the flooding area shall be filled with water
2. It is managed the wet and dry periods of the aquatic ecosystem
3. It preserve washing water in spring



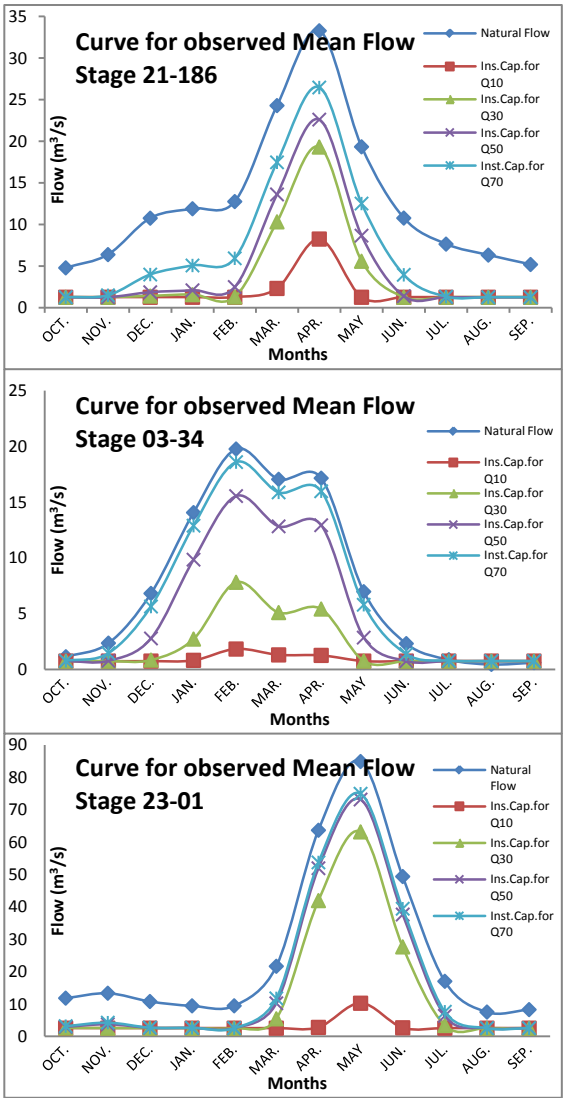


Fig. 2 Discharge course curve in working area stations; Q10, Q30, Q50, Q70 water amount that will be placed in river for design discharges.

On the other hand, in the event that project discharge becomes the discharge which has potential of being exceeded in 10 percent of time of the discharge existing in the river, flood plain is exposed to damage and only the lowest discharge is given to the river for nine months. In this sense, at the end of the study, another method is being offered for environmental water calculation methods.

7.2 Project Discharge Selection (PDS)

By **PDS**, water amount the plant will release into the stream during project phase will be brought under self control. While making a choice for discharge which is based on installed power, project engineer will decide by supervising water amount being released into the stream as the result of economic analysis instead of choosing the value that comes out only as the result of economic analysis. If devising with the discharge which is

found in ten percent of time in consequence of economical analysis condemns the stream into environmental water for nine months, installed power of the plant will be lowered until flood plain gives wash water value. And the government will not permit choosing such low potential discharges while controlling the calculation of installed power of these projects.

It is known that when project discharge is chosen as very high, environmental ecosystem is exposed to damage and projects are cancelled by courts because of environmental reasons. Thanks to this application, projects which are environmentally friendly and sustainable will be able to build by using lower power.

It should be asserted that environmentally friendly HEPPs which have lower power are more productive and meaningful than HEPPs which cannot be built because of environmental reasons and which have bigger power.

8. CONCLUSION AND PROPOSALS

Water played important roles in the development of civilizations throughout the history. And the populations settled down either on riverine and/or marine courses or springs since the early ages. Water has always been in service of humanity and it has always been the main character for development. However, increasing population and irrepressible increase in contaminating elements have made it compulsory to care more for water. Thus, it is one of our most important responsibilities for the next generations to provide the effective utilization of water of which we feel shortage even nowadays.

There is an increasing need for energy today and the most important way for obtaining maximum benefit from clean energy resources is to operate hydroelectric potential via sustainable politics. In this sense, environmental effects of HEPPs should be taken into consideration through the duration when HEPPs which have been developed and are in phase of being developed are designed.

There are a lot of methods on different basis concerning the calculation of lowest flow value (mistakably named 'vital water'). Each of these methods has been approved for rivers which have fulfilled specific conditions by different countries. There are measures which are dependent on climate, river, habitat and river use for each of the methods.

Calculations which are done according to the different methods used by different countries to calculate vital water which is one of the most important characteristics in terms of environmental aspect have been calculated in the scope of the study. In the study conducted on the same river, there has been ten times differences according to the results of different methods.

For today, in Turkey, General Directorate of State Hydraulic Authority demands Montana or Modified Tennant Methods to be implemented for calculation of lowest flow in non-storage HEPP projects. 10 percent of average flow values of the last ten years is taken into account as the lowest flow value and it is estimated as a stable value for the whole year. Suitability for the river regime is being neglected.

In this study, 'Project Discharge Selection Method'(PDS) which does not take part in literature has been proposed for calculating environmental water concerning HEPPs that are planned to be built on rivers. A HEPP which is planned by taking this method into account will have a self-control system that can protect habitat during operation. By this means, HEPP projects which have been completely planned or which are in the phase of being planned will not be cancelled because of the vital water reason and HEPPs that are one of the main clean energy resources will be able to be put into use.

After this study, what is proposed is to systematize this method. It is known that discharge that is based on installed power has a serious effect on vital water. If project discharge selection can be generalized for all the rivers, power plants which will be potentially built will have environmental precision concerning environmental water at the beginning phase.

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REMEDIATION OF HEAVY METALS IN CONTAMINATED SOIL BY USING ELECTROKINETIC TECHNIQUE

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***Abstract*—In this paper, the application of electrokinetic technique to remediate organic soil contaminated with heavy metals (Lead, Zinc and Copper) was carried out. The organic soil sample was taken from specific area that mainly involves in industrial and agricultural activities. The physical and chemical properties of contaminated soil sample were assessed. Electrokinetic remediation process was carried out for 5 days operation of each heavy metal. The results of zeta potential behaviour upon soil pH, voltage behaviour during the process and images produced by SEM-EDX, showed the success of EK process to remediate contaminated soil by those heavy metals.**

Keywords: organic soil, electrokinetic, remediation, heavy metals contaminant, zeta potential.

1. INTRODUCTION

Mining, manufacturing and the use of synthetic products can result in heavy metal contamination of urban and agricultural soils. Heavy metals also occur naturally, but rarely at toxic levels. The specific type of metal contaminant expected at a particular site would obviously be directly related to the type of operation that had occurred there. In recent years, the contamination of subsurface soils has generated enormous public concern in various sites such as residential areas, near industrial complexes and reservoirs of drinking water [1]. This has created an urgent need to find feasible solutions to the problem because they can pose a risk to both groundwater quality and the health of humans.

There are several soil treatments such as bioremediation, thermal desorption, soil vapor extraction, soil washing and soil flushing, however, it is expensive and time consuming and also has failed to show efficient results in removing heavy metals issues faced by environmental professional [2]. The properties of soil such as low hydraulic conductivity of fine-grained soils somehow contribute to the failure of those treatments [3].

Electrokinetic (EK) soil remediation methods have increased interest during the last decade due to many promising results obtained in both laboratory scales as well as in pilot tests and full scale actions [4]. The main interest in electrokinetic soil remediation in environmental cleanup operations lies in an attempt to concentrate and confine contaminants close to an electrode and remove them if possible. Due to the electric field present when a low-level direct current (DC) is passed between a pair of electrodes placed in a system containing charged particles (moist contaminated soils – organic soils), the pollutant species are driven towards one of the electrodes, from where they may be removed [5].

The objectives of this study are i) to investigate the behaviour of heavy metals removal, lead (Pb), zinc (Zn) and copper (Cu), in the contaminated organic soil samples, ii) to investigate the effect of zeta potential and charges in soil pH and iii) to examine the ability and efficiency of the Electrokinetic technique to remove the heavy metals contaminants from the organic soil.

2. MATERIALS AND METHODS

The organic soil samples were collected from Area X which is recently experienced intense development pressure for housing, industry and commercial activity. The development of Port X and its associated maritime and shipping industries features prominently in this area. Several studies on the presents of heavy metals in soil and surface sediment have been carried out [6, 7] around the area. Those heavy metals compounds are chosen for this study because it was the most identified species in hazardous waste sites such as hazardous waste landfill, industrial wastes site, abandoned mining wastes and exposure as a main risk factor for several adverse health effects [8].

There are two part of tests for this study; a) the soil samples were prepared to evaluate basic properties, i) physicochemical properties: Atterberg limit, moisture content, organic content, particle size distribution, specific gravity (BS 1377:1990) and specific surface area (BET technique); and b) for contaminated soil samples (by lead, copper and zinc), iii) electrochemical properties: zeta potential (ASTM D 4187), pH (BS 1377:1990) and a series of EK test was carried out.

The EK cell consisted of an acrylic tube, 150 mm long and with 69 mm diameter, where the osil was molded and connected at both ends to acrylic cylindrical chambers of the same diameter (Fig. 1). The EK injection technique was used by applying a constant electrical potential of 60V across the specimens. An acetate buffer solution for catholyte and magnesium sulfate ($MgSO_4$) for anolyte was used. The relationships of zeta potential value with pH, soil pH with normalized distance and voltage with time were established. Scanning Electron Microscope and Energy Dispersive X-Ray (SEM-EDX) equipment was used to identify the elements and contaminants present in the soil samples before and after the Electrokinetic test in order to determine the efficiency of the test.

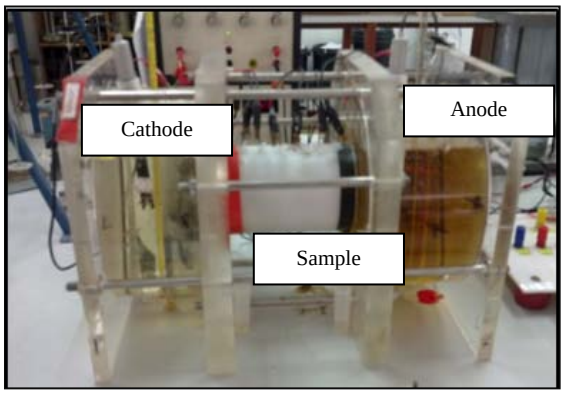


Fig. 1 Electrokinetic apparatus.

3. RESULTS AND DISCUSSION

3.1 Physicochemical Properties of Organic Soil

Table 1 shows the physicochemical values for organic soil. Sample can be categorised as high moisture content, 165.2%. The value for the organic content is 66.96%, specific gravity, G_S, is 1.38 and specific surface area is 2.35 m^2/g which is can be related to the electrical conductivity and chemical diffusion in soil [9]. The soil pH is in neutral range, 6 – 6.5.

Table 1. Result for physicochemical properties of organic soil.

Parameter	Value
Moisture content (%)	165.2
Specific gravity	1.38
Organic content (%)	66.96
Liquid limit (%)	78.00
Plastic limit (%)	70.35
Plasticity index (%)	7.65
Specific surface area (m^2/g)	2.35
pH	6 – 6.5

The value of plastic limit and plasticity index for the organic soil is plotted on the plasticity chart (Fig. 2). The sample appears to be in the range of inorganic silts of very high plasticity. Other materials such as dredged waste [10] and red mud [11] also fall in a silt range and have been classified as having a medium plasticity.

According to the particle size distribution chart in Fig. 3, it shows that the sample was classified as coarse grained soils and well graded which more than 50% has passed the sieve 4.75mm.

3.2 Zeta Potential and pH Relationships

Zeta potential and pH relationship was assessed using four concentrations of lead, copper and zinc (1M/L, 0.1 M/L, 0.01M/L and 0.001M/L respectively). The variations of pH is based on the NaOH that has been add to each of the contaminants sample related to the nature of electrical energy field in organic soils and dissociation of H^+ [12]. The soil surface charge was affected by pH; pH increase, the net negative charge increase, vice versa. At a certain pH, the soil surface charge could drop to zero, rendering a zero zeta potential which is called iso-electric point [13]. Negative surface charge of particles causes electroosmosis to occur from anode to cathode, while positive surface charge causes electroosmosis to occur from cathode to anode [14].

From the obtained result of zeta potential, the zeta potential of concentration 0.1M/L for all contaminant varied from -1.65 mV at pH 4.7 to -8.16 mV at pH 11.2. The zeta potential was almost zero at pH 5 for copper, pH 6.2 for lead and pH 10 for zinc from Fig. 4. The results for different concentration of heavy metals show the similar patterns. It can be concluded that as the pH increase, the zeta potential value tend to produced almost negative charge.

3.3 Electrokinetic Tests

3.3.1 Soil pH Relationships

Fig. 5 shows pH distribution of soil sections for lead, copper and zinc after EK experiments (day 5). The initial pH of soil is in the range of 6 -6.5 and soil pH for sample contained lead, copper and zinc were 5.99, 6.04 and 5.96. It can be seen that the pH of the soil samples after the application of EK was ranged between 3.5 and 1.5. The extraction of heavy metals was effective at pH condition below 4.0 [15]. Therefore, the removal process of heavy metal from the soil samples by EK can be concluded as succeed and efficient.

3.3.2 Voltage Effects

Fig. 6 shows the voltage profiles of the three contaminants against time. The voltage profile for the tests showed similar patterns; the resistance and voltage drop gradually across the soil cell.

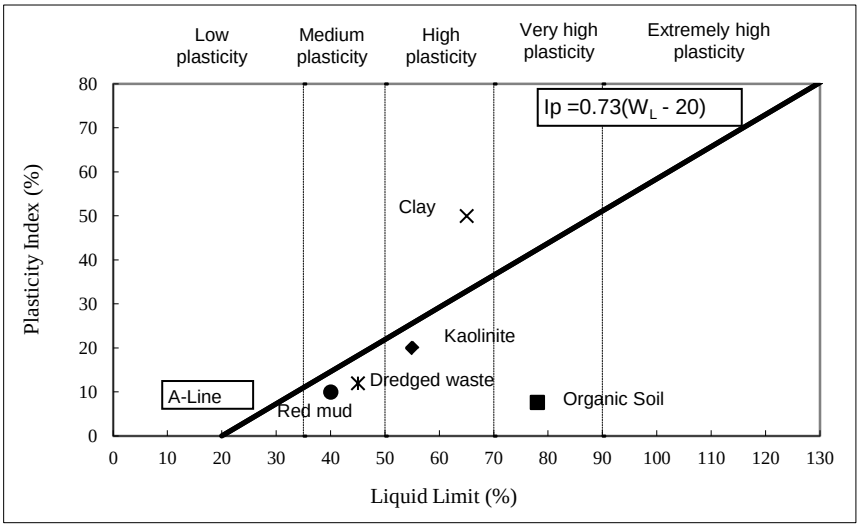


Fig. 2 Plasticity index chart.

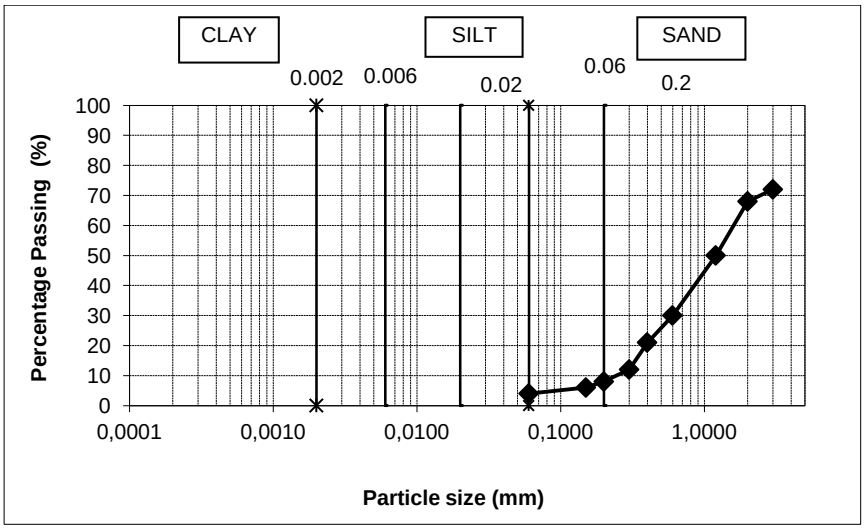


Fig. 3 Particle size distribution chart.

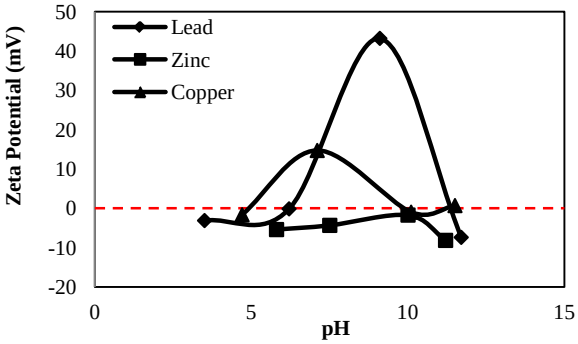


Fig. 4 Zeta potential for lead, zinc and copper values at concentration 0.1M/L versus pH.

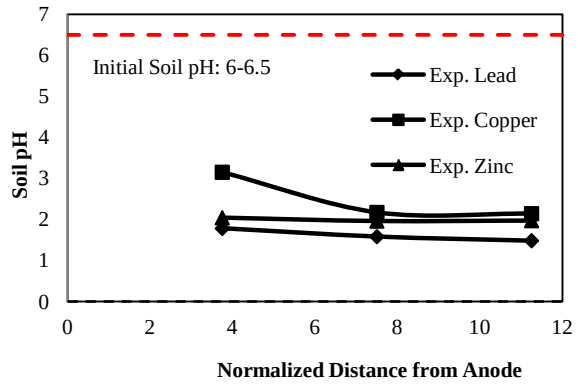


Fig. 5 Variation of soil pH at sampling points in the soil cell during the removal process of heavy metals.

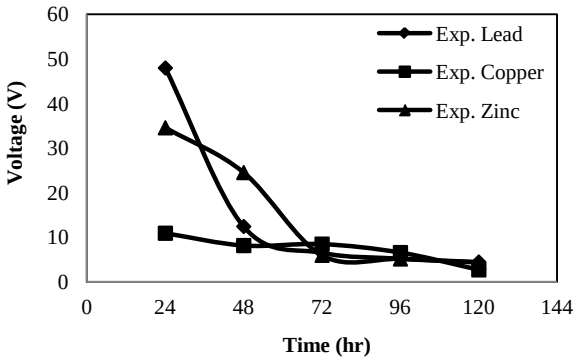


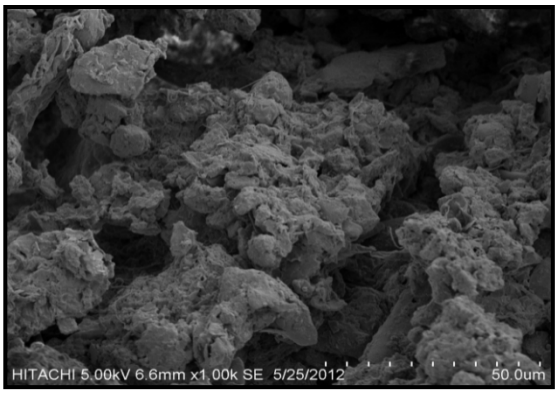
Fig. 6 Voltage profile in the soil cell during the removal process.

The organic soil used in all the samples had a much larger void fraction. As explained by [16], most of the hydrogen ions did not participate in desorbing species within this condition, but migrated toward the cathode compartment in the pore fluid. Continuous migration of the hydrogen ions in pore fluids and the increase of desorbed species meant the resistance in the soil cell decreased, resulting in a gradually decreasing voltage drop during the tests [16].

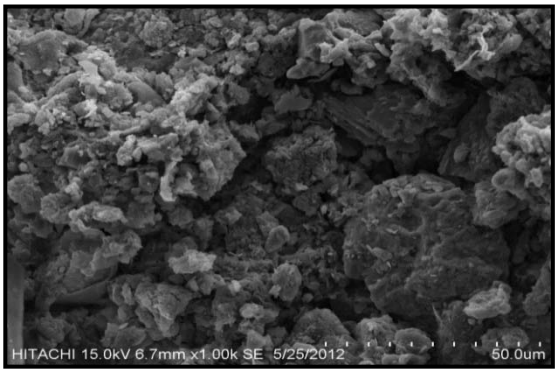
Structural Properties

The SEM micrographs of sample contaminated by zinc before and after treated by EK are shown in Fig. 7 (a, b). Before treated process, the particles are loose and there are many voids appear. However, the structure of soil sample is changed to closely packed and strongly bonded after treated by EK. The similar pattern occurred to the soil sample contaminated by lead and copper.

Using SEM-EDX equipment, a spectral image of contaminated sample by zinc before and after treatment was produced as shown in Fig. 8 (a, b). It can be seen the different of the heavy metals particles present (white dot) in soil samples before and after the treatment. After the application of EK tests, the heavy metal particles of zinc (white dot) reduced showing that by applying a direct current to remediate heavy metals from organic soil is succeed.

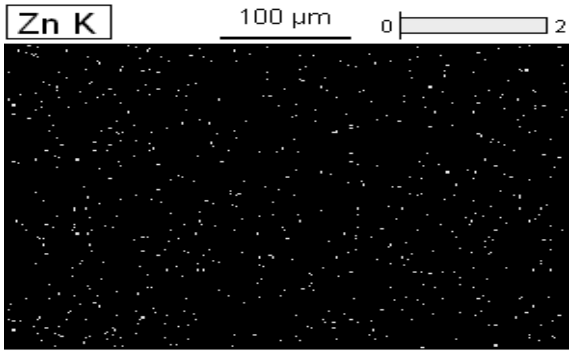


(a)

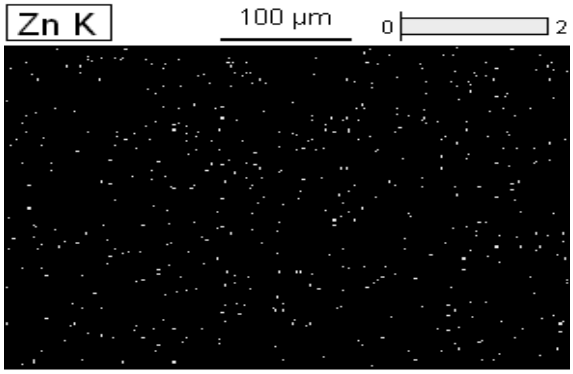


(b)

Fig. 7 (a) Soil sample before treated process and (b) Soil sample after treated process.



(a)



(b)

Fig. 8 (a) Soil sample before the treated process, (b) Soil sample after the treated process.

4. SUMMARY AND CONCLUSIONS

The application of electrokinetic technique to remediate organic soil contaminated with heavy metals (Lead, Zinc and Copper) was investigated. From the laboratory tests results, the following conclusions can be drawn:

- The moisture content, specific gravity, plasticity index, particles size distribution, organic content and specific surface area had shown the contribution in order to accelerate the remediation process.
- The variations in zeta potential with pH were related to the electrical energy in organic soils. The charge was affected by pH.
- The pH of the soil samples after the application of EK was ranged between 3.5 and 1.5 and the extraction of heavy metals was effective at pH condition below 4.0.
- The organic soil used in all the samples had a much larger void fraction causing the resistance and voltage drop gradually across the soil cell.
- Structural properties of contaminated soil shown that before treated process, the structures are loose and there are many voids appear. However, the structure of soil sample is changed to closely packed and strongly bonded after treated by EK.
- The spectral image of each contaminated soil showing the reduction of heavy metals particles present in sample after applying a direct current.

Elektrokinetic soil remediation is an emerging in situ technology with demonstrated efficiency to remediate organic soil contamination with heavy metals for example; lead, copper and zinc.

Based in the results and withdrawn conclusions, there is a great likelihood that elektrokinetic techniques could be used very effectively and able to resolve organic soils difficulties from the geoenvironmental viewpoint.

ACKNOWLEDGEMENT

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RENEWABLE ENERGY SOURCES AND SOLAR ENERGY POTENTIAL IN TURKEY

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***Abstract*—Nowadays, the use of renewably sourced materials have gained importance. Environmental problems caused by fossil fuels, and in line with the rapidly increasing global energy demand has also increased the demand for alternative renewable energy sources. Our country due to its geographic allocation in terms of the potential for solar energy is fortunate compared to many countries. This study explored the potential of solar energy in Turkey, solar energy advantages and disadvantages put forward.**

KeyWords: RenewableEnergy, Solar Energy, Solar Energy Advantage, Solar Energy Disadvantage.

1. INTRODUCTION

Solar energy is based on the technology of obtaining energy from sunlight. Emitted by the sun and the energy reaching our planet, at the core of the sun's radiant energy released by the fusion process from the fusion phase in the form of hydrogen gas into helium in the sun. The sun, our planet and other planets, the only one with a force of energy that can be called an infinite source of energy. Coal, oil, water potential, biomass and other energy sources such as wind, solar radiation is composed of substances effect on the physical [1]. The sun is a thermo nuclear reactor, 564 million tons of hydrogen each second as a result of fusion, and lost 4 million tons, 560 million tons of helium turns into a mass 386 million for EJ (Eksan joules) of energy is released. Solar, the world's orbital axis, 1.366 watts/meter² transmit energy. but it is slightly less than the amount of energy that reaches the earth. Intensity of solar radiation outside the Earth's atmosphere, more or less constant, and 1370 W/m values, but the values on the earth varies between 0-1100 W/m^2. Even a small fraction of this energy from the world of humanity, many times greater than current energy consumption. Studies on utilization of solar energy has gained momentum especially after the 1970s, solar energy systems decreased in terms of technological progress and cost, solar energy has maintained itself as an environmentally clean source of energy [2].

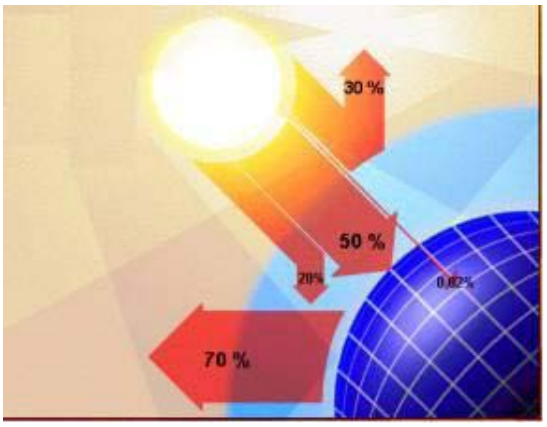

Fig. 1 The distribution of radiation from the sun [3].

2. TURKEY IN SOLAR ENERGY

Our country due to its geographical location in terms of the potential for solar energy is for tunate compared to many countries. Turkey Statistical Institute (TSI) was made in 2009 in Turkey, according to the monthly average daily total sunshine duration data Table 1 [2].

Table. 1 Average Monthly Daily Total Hours of Sunshine 2009 (Hours – Minute) [3].

Stations	January	February	March	April	May	June	July	August	September	October	November	December
Zonguldak	1.39	1.01	1.51	3.48	6.09	10.58	10.23	11.16	6.45	5.12	3.18	1.16
Samsun	3.01	2.43	4.18	6.51	7.15	9.04	8.46	9.39	6.03	5.27	3.34	2.45
Trabzon Meydan	3.22	2.29	4.03	5.19	7.04	6.24	7.58	6.44	4.33	4.34	3.03	2.33
Rize	2.28	2.39	4.06	4.56	6.15	6.09	6.01	6.30	4.32	4.21	2.09	1.39
Edirne	0.00				11.10	11.05	11.07	11.04	7.12	5.07	3.54	1.18
Tekirdağ		2.23	4.04	6.42	6.25							
Kastamonu						6.40	9.00					
Merzifon	3.01	2.29	4.33	7.40	8.09	10.19	9.42	11.25	6.46	6.34	4.03	2.18
Sivas	2.33	2.47	3.45	8.03	9.19	10.52	10.39	12.55	8.16	7.13	3.57	1.45
Erzincan	3.41	2.34	3.45	6.20	8.07	9.58	9.18	9.51	7.06	6.51	3.37	2.08
Kars	3.25	3.58	5.13	7.00	8.42	7.33	9.27	10.21	6.33	6.48	4.07	2.02
Çanakkale	1.16		10.18									
Eskişehir Anadolu	2.36	2.50	4.23	7.27	8.44	10.50	10.29	11.58	8.00	6.14	4.31	1.49
Kütahya	1.45	2.11	4.21	6.07	8.00	10.42	9.37	10.06	6.38	6.04	3.39	1.27
Kırşehir	3.43	2.54	4.21	6.45	9.15	11.45	10.36	11.33	8.39	7.16	5.19	2.22
Van	4.44	5.04	5.46	7.39	10.54	9.22	11.48	11.25	8.40	7.33	5.58	3.37
Kayseri	2.47	2.32	3.19	6.13	8.57	10.16	9.39	12.20	7.15	6.15	4.14	2.28
Malatya	4.15	2.55	4.28	7.55	10.00	10.10	11.06	12.58	10.00	7.10	3.35	2.12
Siirt	3.33	3.06	4.43	6.43	9.40	8.01	11.27	10.36	9.09	7.15	4.21	2.00
İzmir	3.51	3.55	5.30	7.00	10.28	11.34	12.10	12.09	9.18	7.25	6.19	2.33
Isparta	3.33	2.39	4.27	6.34	8.08	9.21	9.43	9.52	8.09	6.29	6.21	2.47
Gaziantep	3.22	2.32	4.18	6.26	8.04	8.10	8.52	9.26	8.06	6.25	5.13	2.18
Şanlıurfa	3.37	3.13	4.54	7.44	10.57	10.25	11.46	11.09	9.33	6.55	4.49	2.10
Muğla							10.07					
Antalya	4.30	4.22	5.04	0.00	0.00	9.49	11.01	10.17	8.39	7.45	6.44	0.00
Anamur	4.02	4.40	6.06	8.40	10.09	10.53	10.38	10.26	8.56	7.51	6.11	3.34
Adana	3.41	3.36	5.39	7.51	10.58	10.49	10.39	10.21	8.26	7.57	5.02	2.13
Antakya	3.03	2.11	4.48	8.28	10.36	11.00	11.30	11.24	8.49	6.13	4.42	1.44
Islahiye	3.10	2.19	4.12	7.03	9.41	10.20	11.01	11.19	8.13	6.53	5.07	2.03

Table 2. Turkey 's Distribution of Annual Total Solar Energy Potential [3, 4].

REGION	**TOTAL SOLAR ENERGY (kWh/m^2-year)**	**SUNSHINE DURATION (Hours/Years)**
South eastern Anatolia	1460	2993
Mediterranean	1390	2956
Eastern Anatolia	1365	2664
Central Anatolia	1314	2628
Aegean	1304	2738
Marmara Region	1168	2409
The Black Sea Region	1120	1971

Turkey's more solar energy in the region of South Eastern Anatolia Region, and it follows the Mediterranean Region.

According to a study done in previous years in the solar energy potential and the regional distribution of sunshine duration values in Table 2 is also given. However, this value is less than the full potential of Turkey, subsequent studies showed that Solar energy potential of Turkey is expected to be more than 20-25%.

3. ADVANTAGES OF SOLAR ENERGY

- Solar energy is an inexhaustible source of energy.
- Solar energy is a kind of pure energy. Gas, smoke, dust, no harmful substances such as carbon or sulfur.
- The sun, source of energy available to all countries of the world. In this way, countries will be eliminated in terms of energy dependencies.
- Another characteristic of the solar energy, without spending any transport may be achieved everywhere.
- Which is more or less where the sun is a little difference in yield, the mountains, hills, valleys, plains, or to take advantage of this energy.
- Solar energy does not require any complex technology. In almost all countries, thanks to the local industrial organizations to easily make use of this energy.

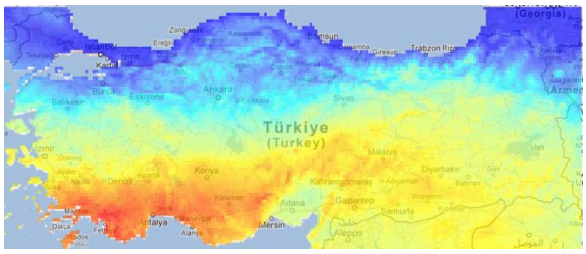

Fig. 2 Map of Solar Energy in Turkey [5].

4. DISADVANTAGES OF SOLAR ENERGY

- Solar energy density is low and constant. The desired density may not be available at any time.
- Mechanisms to take advantage of solar energy to the current technological stage investment costs are high.
- The amount of energy from the sun is not connected to our request, and we cannot control.

5. RESULTS

Develop in to other types of energy storage of solar energy and thermal, mechanical, chemical and electrical methods will be. Heat storage or cycle, specific heat capacity is high and easy to use cheap ingredients. They are pebble beds are among the water, oil, chemical storage hydrate salts are used. Electricity storage in the energy storage batteries is made. Solar energy cycles, or directly to: user-water heating, swimming pools, boiling and baking, drying of plant products, distillation of water, heating and cooling of buildings (classification climate); cooling, with a total production of electricity, heat and power systems irrigation water pumping, industrial process heat production, electricity generation and is used in many areas in order to realize the photochemical and photosynthetic cycles. You do not need to follow the sun, oriented to the south, and the sun's rays on a steep slope placed on giving these collectors, need to be adjusted seasonally.

Areas of use of solar energy and clean energy is more than the fact that due to the limited and non-renewable energy sources Given exhausted and considering most of the year the sun's rays reach the solar energy use are as should be extended to Turkey.

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COPPER REMOVAL FROM SYNTHETIC WASTE WATER WITH DIFFERENT DESIGN REACTOR BY ELECTROCOAGULATION

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***Abstract*—The performance of Cu removal from aqueous solution on an electrocoagulation reactor with iron electrodes was investigated on laboratory scale. Basically, two parameters were run in the experiment: current density and initial Cu concentration. The experimental results show that the removal efficiency is increased by increasing current density and the initial metal concentration. The initial concentration of 50mg/L was reduced to 1,08mg/L with the removal efficiency of 98% at 30mA/cm^2. Also it is observed that the residual concentrations were decreased from 20mg/L to 1.19, 0.22 and 1.80 mg/L at the current densities of 30, 40 and 50 mA/cm^2, respectively at the end of 90min EC operation.**

Keywords: Copper, electrocoagulation, removal, wastewater.

1. INTRODUCTION

Wastewaters containing heavy metals, which are coming from different types of industries, are discharged into the environment directly or indirectly. Heavy metals can be absorbed by living organism and are not biodegradable. Considering the food chain, heavy metals can be accumulated into human body and it must be treated before discharging [1-3]. Chemical coagulation and precipitation is the most widely used method for treatment. Also different methods such as adsorption, reverse osmosis, electrocoagulation and ion exchange can be used [3,4]. Among these methods, electrocoagulation provides important and useful advantages such as small retention time, easy operation, simple equipment, not needed chemical addition, high sedimentation velocity and possibility of complete automation [4-7].

2. ELECTROCOAGULATION (EC) PROCESS

Electrocoagulation is not a new technology. However, in more recent days scientists has renewed interest against process [6,8]. Since it is simple and efficient electrochemical method. EC is based on the stability of colloids, suspension and emulsions are influenced by electric charges. Thus, if additional electric charges are supplied to the charged particles via appropriate electrodes, the surface charge of the particles is neutralized and several particles combine into larger and separable agglomerates [9,10]. The basic mechanism is as follows;

At the anode

$$Fe \rightarrow Fe^{2+} + 2e^- \quad (1A)$$

$$Fe \rightarrow Fe^{3+} + 3e^- \quad (1B)$$

At the cathode

$$2H_2O + 2e^- \leftrightarrow 2H_2 + 2OH^- \quad (2)$$

Depending on the pH of the aqueous solution, different ionic species, such as Fe_{aq}^{3+}, $Fe(OH)^{2+}$, $Fe(OH)_2^+$,and $Fe(OH)^{4-}$ may also be present in the system. The pollutants can be removed from solution by suspended solid of iron hydroxides with mechanisms such as co-precipitation, sorption or electrostatic attraction, followed by coagulation [11].

3. MATERIAL AND METHOD

3.1 Solutions

Stock solution of 1000 mg/L Cu was prepared with dissolving required amount of $Cu(NO_3)_2.3H_2O$ (Merck). To increase the conductivity, 0,05M sodium sulfate was added to solution before operation. The pH of the solution was arranged with 1N H_2SO_4 (Panreac).

3.2 EC Cell and Operation

In this study the unique designed batch EC iron cell was used as seen from the Fig. 1. The iron cylindrical reactor that operated as cathode was of 10cm height and had an internal diameter of 11,3 cm. The mechanical stirrer (Heidolph RZR 2102) consisted of three iron blades with 10cm length and 0,9cm width and operated as the anode. Anode was dipped into the reactor containing wastewater and stirred the solution to maintain uniform composition at 100 rpm while acted as anode.

In each run, 0.5 L of solution was poured into the electrolytic cell, and the pH, conductivity, and current density were adjusted to the desired value. The reaction was started by switching the DC power supply (Statron 2257) on.

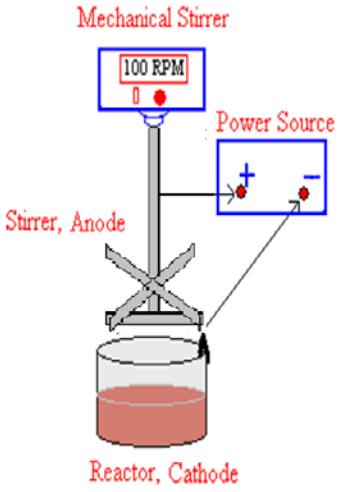

Fig. 1 Experimental set-up.

3.3 Measurement

Samples from the effluent were taken at 15-minute intervals and filtered. The solution was analyzed for Cu determination using atomic adsorption Varian Spectra A 250 Plus. All the samples were analyzed in duplicate to ensure data reproducibility, and an additional measurement was carried out, if necessary.

The calculation of removal efficiency (*RE%*) after EC was calculated using the equation:

$$RE\% = (C_o\text{-}C\ /C_o)\ x\ 100 \quad (5)$$

where C_o and C are the concentrations of Cu before and after EC, respectively, in mg/L.

4. RESULTS AND DISCUSSION

4.1 Effect of Current Density

It is known that current density has an impact on electrocoagulation processes. To determine the effect of current density experiments were performed at initial pH of 5 and initial copper concentrations of 20 mg/L. After 90 min EC the initial Cu concentration of 20 mg/L was reduced to 1.80, 1.19 and 0.22 mg/L with the removal efficiencies of 92, 93 and 98% at the current densities of 30, 40 and 50 mA/cm^2, respectively after 45 min EC treatment as seen from Fig.2. After that time, the residual removal is almost same. Thus, lower operation time can be run to reduce operational cost.

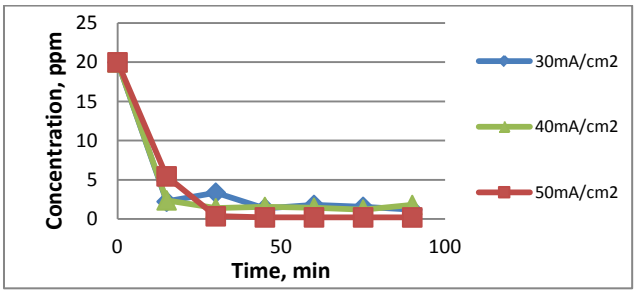

Fig. 2 Variation of Cu concentration with current density.

4.2 Effect of Initial Concentration

The effect of initial concentration from 10 to 50 mg/L was examined at current density of 30mA/cm^2 and pH of 5. Fig. 3 shows the variations of Cu concentration for different initial concentration as a function of time. As seen from the Fig.3 higher initial concentrations caused higher removal rates of Cu. The removal efficiencies were 74, 91 and 97% at the initial concentrations of 10, 20 and 50 mg Cu /L, respectively at the end of 60 min EC operation.

At the higher concentration the EC treatment is more effective because coagulant that produced during the electrocoagulation can come into the contact with pollutant with higher possibility. Additionally, higher initial concentrations were reduced significantly in relatively less time than lower concentrations.

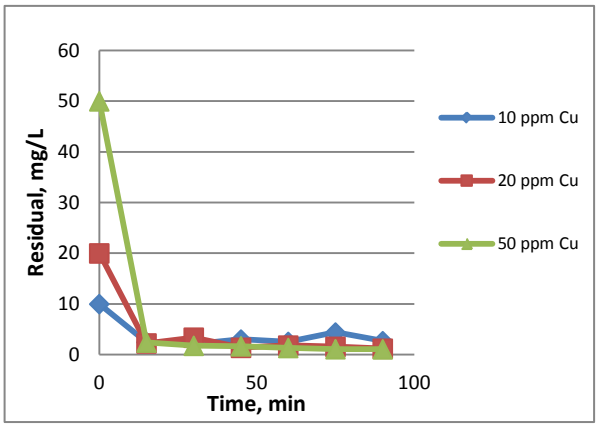

Fig. 3 Variation of Cu concentration with initial Cu concentration.

5. CONCLUSION

In this study, EC treatment using unique designed iron electrodes were used for the treatment of model wastewater containing Cu. The effects of current density and initial metal concentration on the removal efficiency were examined.

The Cu removal more than 92% was obtained at all current densities studied after 45 min electrocoagulation. It was seen from the experiments that EC treatment has higher efficiency at the higher initial concentrations.

In conclusion, electrocoagulation is a suitable process for removal of heavy metals from wastewaters using iron electrodes.

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THE PRODUCTION OF CITRIC ACID FROM WASTE MOLASSES BY YEAST

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***Abstract*—In this study, the citric acid production from sugar molasses was carried out by Yarrowia lipolytica yeast and the product was separated from fermenter broth with an anion-exchange resin packed-column. Two different medium was used for production of citric acid. First medium contained schlep as a micronutrient. The experimental results indicated that the schlep increased the citric acid productivity and sugar conversion from 1.085 g/l and 16.4% to 1.371 g/l and 17.2%, respectively.**

Keywords: Citric acid production; Fermentation; Separation; Anion-exchange resin.

1. INTRODUCTION

Citric acid is produced almost exclusively by fermentation, and widely used in the food, alcoholic and non-alcoholic beverages beverage, chemical, pharmaceutical, metallurgical, cosmetics, textile, tobacco, plastics, plating industries and many other industries [1]. A large amounts of citric acid (about 400,000 t per annually) are used by large-scale industrial processes. In view of the numerous applications of citric acid, its production by fermentation is increasing continually [2]. The study reported previously by different researchers that refined sucrose is the substrate most commonly used for producing citric acid by fermentation. However, production cost could be reduced if expensive refined sucrose from beet molasses was used instead, especially if the microorganism could produce citric acid directly from molasses, a valuable and economical fermentation substrate produced as a by-product of sugar manufacture [3]. In the present study, the production of citric acid was carried out by *Yarrowia lipolytica*. Moreover, product separation was made by anion-exchange resin from the fermentation broth.

2. MATERIALS AND METHODS

2.1 Microorganism

Yarrowia lipolytica was obtained from Greece. It was maintained on malt extract slants and stored at 4 °C and recultured every other month [4]. A loopful of conidia of *Y. lipolytica* was inoculated into a malt extract agar slant and incubated at 30 °C. After 6 days incubation, 5.0 mL of sterile physiological saline was added and shaken violently. The spore suspension was then collected and used as inoculums.

2.2 Medium

Molasses, obtained from a local sugar mill, was used as substrate. Prior to fermentation, crude molasses was pretreated by HCl. The molasses was diluted with distilled water to adjust the sugar concentration. Two different medium was prepared for citric acid production. The medium (A) contained the following (g/l): molasses 50; KH_2PO_4 7.0; $Na_2HPO_4.2H_2O$ 2,5; $MgSO_4.7H_2O$ 1,5; schlep 2 (as micronutrient). The medium (B) contained the

following (g/l): molasses 50; KH_2PO_4 7.0; $Na_2HPO_4.2H_2O$ 2,5; $MgSO_4.7H_2O$ 1,5; $CaCI_2.2H_2O$ 0,15; $FeCI_3.6H_2O$ 0,15; $ZnSO_4.7H_2O$ 0,02; $MnSO_4.5H_2O$ 0,06. The pH of the medium and temperature were adjusted to 6.0 and 28 ^oC, respectively.

2.3 Fermentation Conditions

Fermentation was carried out in a 250 mL erlenmeyer flask with 100 mL working volume. The surface culture was used and the incubating temperature was 28 °C. After six days fermentation, the broth was centrifuged. All experiments were conducted in duplicate and mean values were used in the analysis of data.

2.4 Resin Preparation

An anion-exchange resin (Ambersep 900 OH, Fluka) was used for citric acid separation from the fermentative broth. The resin was pretreated as described by Jianlong et al. [3]. First, saturating with distilled water for 24 h and then saturating with 2 M HCl for 24 h. Second, thoroughly washing with distilled water and saturating with 2 M NaOH for 24 h. Third, washing with distilled water to remove the alkali. The pretreated resin was filled into a column (4.0 cm × 30.0 cm).

2.5 Elution of Citric Acid From Resin

The citric acid adsorbed on resin was eluted by 1 N H_2SO_4. After six days fermentation, the broth was centrifuged and it was pumped into the column with a flow rate of 2 mL/min.

2.6 Analytical Methods

The fermentation broth was centrifuged in order to remove mycelia and the filtrate was analyzed reducing sugar and citric acid content. Reducing sugar was analyzed by the DNS (3,5-dinitrosalicyclic acid) method [5]. Citric acid was determined by the colorimetric method of Marrier and Boulet [6].

3. CONCLUSION

It is reported in the literature that citric acid production rates and yields are highly dependent on the type of microorganism, the type of substrate and the culture conditions [7, 8]. The most important factors to exert an effect on citric acid production are the type and concentration of carbon source of the fermentation medium, nitrogen and phosphate limitations, aeration, trace elements, initial pH, and temperature [9, 10].

3.1 Effects of Carbon Sources

Citric acid accumulation is strongly influenced by the type and concentration of carbon source as well as micronutrients. The type of the carbon source can be varied according to the microorganism used. Substrate profiles of *A. niger* and yeasts used for citric acid production can be extremely different from each other. For example, sucrose is the most favourable substrate among the easily metabolized pure carbohydrates for *A. niger*, followed by glucose, fructose and galactose. However, in citric acid production processes of yeasts; many distinct substrates could be used besides carbohydrates. Some yeasts are known to be able to produce citric acid from a wider range of carbon sources than fungi do [8]. In addition, molasses is often used as raw material for citric acid production by *A. niger*. Although fructose assimilation levels of some yeasts are reported as low, molasses or invert sugar mixtures can also be used for citric acid production by yeasts [8].

The changes in the maximum specific citric acid production rates with the two different medium for *Y. lipolytica* strain are shown in Table 1. Specific citric acid production rate was maximum at 1.371 and 1.085 g/L for medium A and medium B, respectively. The results showed that schlep had positive effects on citric acid production.

Although fructose assimilation levels of some yeasts are reported as low, molasses or invert sugar mixtures can also be used for citric acid production by yeasts [11]. In a study reported by Karasu-Yalcin et al. [12], growth and citric acid production characteristics of a novel endogenic strain *Y. lipolytica* 57 were investigated in comparison with a citric acid producer strain, *Y. lipolytica* NBRC 1658, in glucose and fructose media in a batch system. The best results for citric acid production were obtained when initial substrate concentrations were above 100 g/L. In another study performed with *C. lipolytica*, it was reported that high citric acid yields were obtained when glucose, fructose and glycerol were used as substrates [13].

3.2 The Citric Acid Separation by Ion-exchange Resin

The centrifuged fermentation broth contained residual sugar, proteins and inorganic salts as well as citric acid, which should be separated from the broth. The change of fermentation broth constituents after passing through the resin packed-column is shown in Fig. 1. The data summarized in Fig. 1 indicated that the fermentation product, citric acid, could be separated satisfactorily from broth. Moreover, the citric acid purification from the ion-exchange resin column was calculated as 84.7% and a slightly larger peak was obtained at 7 min. Previously reported study [3] had shown that residual sugar and NH_4Cl were not exchanged or adsorbed by the ion-exchange resin and their concentration remained almost constant before and after separation. However, KH_2PO_4 was separated from the fermentation broth, thus resulting in the loss of the nutrient.

Fig. 2 shows the FTIR spectra of the neat citric acid and eluted citric acid from the resin column. The vibration bands for the CA are found to be rather broad, as expected due to the strong intra- and intermolecular hydrogen bonding. The 1715 cm^{-1} peak assignable to the C=O vibration in neat and eluted CA is present as a broad band.

Table 1. Effects on carbon source.

Medium	Initial total sugar (g/L)	Residual total sugar (g/L)	Incubation time (h)	Biomass (g/L)	Citric acid (g/L)
A	32.2	29.3	144	7.95	1.371
B	32.5	26.0	144	6.63	1.085

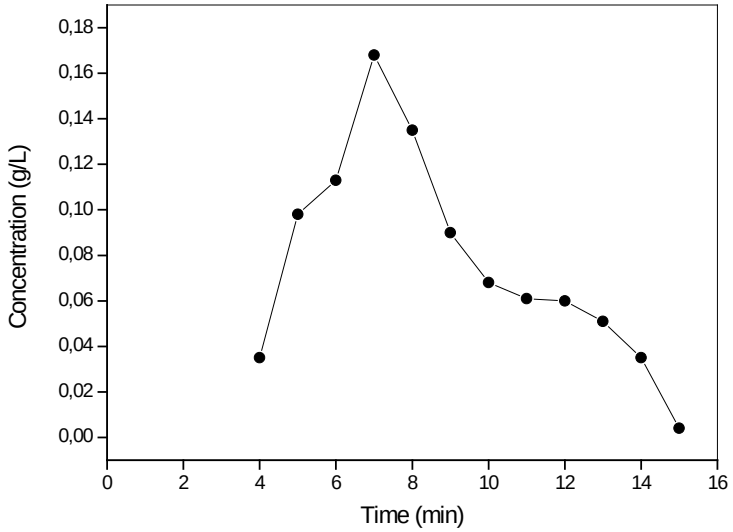

Fig. 1. The citric acid elution from ion-exchange resin.

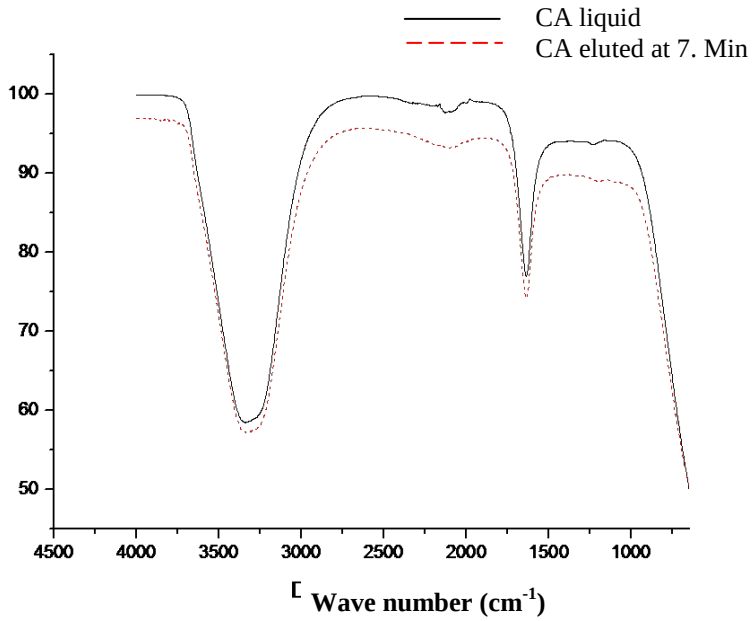

Fig. 2. FTIR of neat and eluted CA.

Fig. 3a and 3b show the raw fermentation broth contained citric acid and purified citric acid elute after passing the resin column. Additionally, the resin column removed color from a dark brown to a nearly colorless. Fig. 3b illustrates the collected elutes from resin column at 1., 2., 3., 4., 7, 9., 11., 13., 15., 17., and 24. min from left to right.

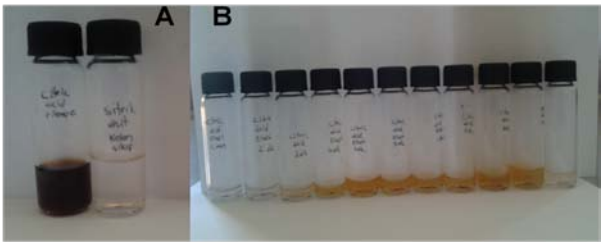

Fig. 3. Raw fermentation broth and elute collected from resin (a) and elutes collected from resin column versus of time (b).

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