Water Quality Assessment of Kan Taw Gyi Lake In Mandalay City

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ABSTRACT*:* This paper describes the water quality assessment of Kan Taw Gyi Lake in Mandalay City. Water samples of physico-chemical parameters from ten stations were carried out three times during May and June 2019. Water samples from upstream of Thingazar Creek, the main inlet of the lake were taken to analyze. Parameters that must be tested include temperature, pH, Electrical conductivity, salinity, turbidity, total dissolved solid, oxidation reduction potential, dissolved oxygen, Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), total nitrogen and phosphates and obtained results were compared to standard values. The pollution status was investigated on the basis of obtained results. Overall, the lake quality is affected by water hyacinth, rain and weather condition and the lake is in hypereutrophic state. Assessment of water quality can get great effects on maintaining the sustainability of the lake.

KEYWORDS: *Kan Taw gyi lake, physico-chemical parameters, standard limit, water quality assessment*

1. INTRODUCTION

Lakes are traditionally under-valued resources to human society. They are important sources of surface water and livelihoods to many region and urban communities. Water quality has become a global concern due to ever increasing population and developmental activities that are polluting water resources. The problem arises due to the pollution of water which effect the health and lifestyle of the human being. Declining water quality in fresh water lake is an increasing problem that threatens the ecosystem services in developing countries. One of the major causes of the decline in water quality is nutrient enrichment. Kan Taw Gyi Lake is one of the recreational areas located in Mandalay City Myanmar. It is located in Chan Mya Tharzi Township at 21º 55′ N and 96º 03′ E. The lake covers 322.43 hectares and 3657.6 m long from north to south and 975.36 m from east to west. The main inlet water resource is Thingazar Creek and the water of the lake is flowed into the Shwe Ta Chaung. The death of fishes, excessive growth of water hyacinth and turbidity of Kan Taw Gyi Lake were observed in last few years. The area is inhabited previously by the farmers and fisherman but nowadays, the surrounding area is filled with restaurants near the lake. To develop sustainability of the lake, the quality of water is needed to monitor and assess.

2. OBJECTIVES OF THE STUDY

The objectives of the study are as follow;

(1) To provide baseline data on some physic-chemical characteristics of the lake water

(2) To assess the water quality of the lake

(3) To determine the trophic status of the lake

3. MATERIAL AND METHODS

3.1 Methods Adopted for Collection of Sample Location

The grab sampling techniques was used to collect samples. 1.5liter polyethylene bottles were used to bring samples. Before collecting the samples, the containers were rinsed thoroughly with the water being sampled and after collection of the samples, the containers were closed with air tight and put in the cooler box. The water samples from 10 stations were collected to assess different physico-chemical- parameters for the determination of the water quality. Then these bottles were transported to water laboratory to perform analyses.

3.2 Location of Sample Stations

The study area is one of the lakes which are located in Mandalay region. As the lake provides a number of environmental benefits, it is important to keep water quality status to protect the water resource of the Mandalay urban area. Kan Taw Gyi is not used for water supply purposes but the lake is still existing for the recreational purposes and greatly maintains the climate of Chan Mya Thazi Township in Mandalay. In this study, 10 stations were sampled for the water quality assessment. Selecting of the sampling points are located preferably by the places of human contact and the area to cover the whole water surface of the study area. Inlet station is selected to know how the polluted wastewater is entering Kan Taw Gyi Lake and how much contaminants containing in it can cause the effect of pollution to the lake. The parameter of first 3 stations are obtained from Thingazar Creek Channel and that of Station 4 is the inlet of Kan Taw Gyi Lake while the rest samples stations are located in the lakes. The values of temperature, pH, pH mV, total dissolved solids (TDS), electrical conductivity (EC), salinity, turbidity, oxidation reduction potential (ORP) and dissolved oxygen (DO) are obtained from in-situ measurement with Horiba U-52 G multi-parameter water quality meter while biochemical oxygen demand (BOD), chemical oxygen demand (COD), total nitrogen (TN) and total phosphorus (TP) were achieved in laboratory analysis. Water sample stations and locations are shown in Table 1 and Fig 1.

Table 1. Location of Sample Station

Station No	GPS Locations	Description
1	21°59' 20.87"N 96° 3'57.96"E	Station near 22nd Street of Thingazar Creek
2	21°58'14.52"N 96° 3'41.98"E	Station near 35th street of Thingazar Creek
3	21°57'38.39"N 96° 4'8.36"E	Station near the 41st Street of Thingazar Creek
4	21°57'15.53"N 96° 4'3.59"E	Station at the end of Thingazar Creek (Six Sluice Gate)
5	21°56'52.52"N 96° 4'13.10"E	In-lake Station near Maharbawdi Datpaung Stupa Dhammarrone
6	21°56'38.46"N 96° 4'7.15"E	In-lake Station near Coffee Tower Restaurant
7	21°56'38.87"N 96° 3'53.75"E	In-lake Station north of Kan Pat Lan near Mandalay Restaurant
8	21°56'26.67"N 96° 3'51.71"E	In-lake Station near Kukko Island
9	21°56'4.79"N 96° 3'43.01"E	In-lake Station near Pyi Gyi Mon Royal Barge
10	21°55'35.51"N 96° 3'49.54"E	In-lake station at the outlet to Shwe Ta Chaung

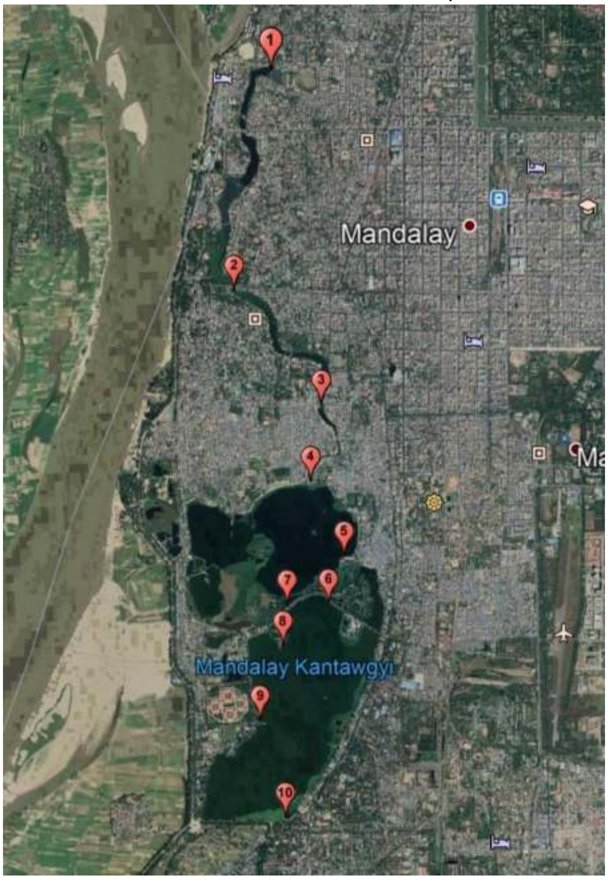

Fig 1. Water Sampling Stations of the Study Area

3.3 Parameters and its unit Consideration

Monitoring the quality of water in the lake requires many different parameters to be sampled. The fundamental characteristics for water quality are physical, biological, chemical and radiological parameters. These water quality parameters are compared with Intrim National Water Quality Standards (INWQS) Class IIB limit of Malaysia. Water quality parameters collected in the study area and its units are shown in Table 2.

Table 2. Parameters and its Units

No	Parameters	Units
1	Temperature	ºC
2	pH	
3	pH (mV)	mV
4	Electrical Conductivity (EC)	mS/cm
5	Salinity	ppt
6	Turbidity	NTU
7	Total Dissolved Solid (TDS)	g/l
8	Oxidation Reduction Potential (ORP)	mv
9	Dissolved oxygen (DO)	mg / l
10	Biochemical Oxygen Demand (BOD)	mg / l
11	Chemical Oxygen Demand (COD)	mg / l
12	Total Nitrogen	mg / l
13	Total Phosphorous	mg / l

4. RESULTS AND DISCUSSION

1) Temperature: Temperature is an important factor to consider when assessing water quality. For the study area, the value of temperature of the lake water is in the range of 28 to 35 °C as shown in Fig 2. Mandalay has a tropical climate and it can also be one factor of high temperature value in surface water of the study area. The temperature become increased as days go by. The temperature taken in 23rd of June is the highest in comparison with the other two. High water temperature enhances the growth of microorganisms and may increase problems related to taste, odor, color and corrosion according to World Health Organization (WHO) guideline 2010.

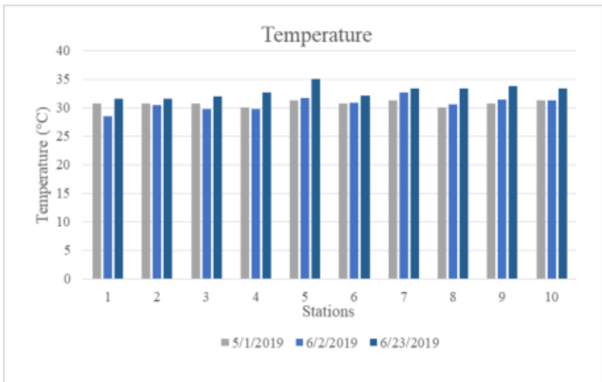

Fig 2. Temperature Variation of Kan Taw Gyi Lake and Thingazar Creek

2) pH: pH is a determined value based on a defined scale, similar to temperature. This means that pH of water is not a physical parameter that can be measured as a concentration or in a quantity.. pH can also fluctuate with precipitation and wastewater or mining discharges. pH levels outside of 6.5-9.5 can damage and corrode pipes and other systems, further increasing heavy metal toxicity. For the study area, pH at the inlet stations and lake stations are in the limitation of Intrim National Water Quality Standards (INWQS) Class IIB range of 6-9 except in Station 8 where the pH is 5.67. Station 8 is located near the restaurant. The pH level raises each time. The value of pH is shown in Fig3 (a) and its limit is shown in Fig 3(b).

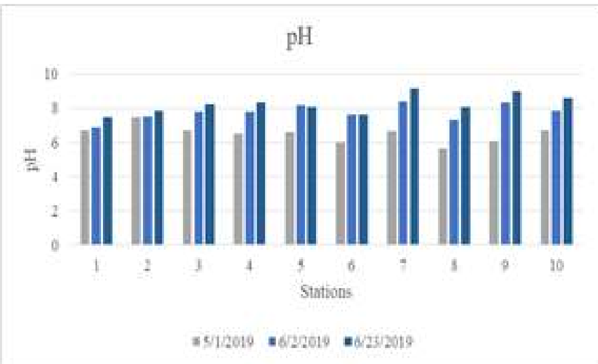

Fig 3(a) .pH Variation of Kan Taw Gyi Lake and Thingazar Creek

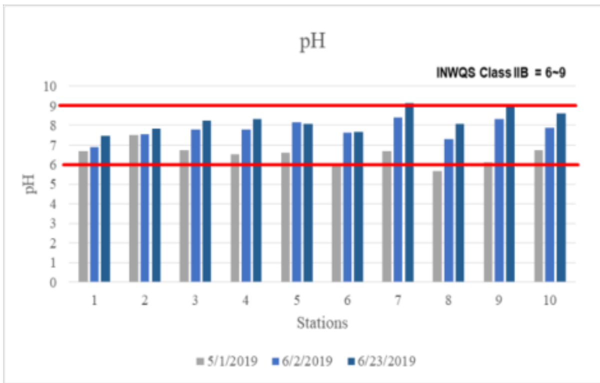

Fig 3(b) . pH Variation of Kan Taw Gyi Lake and Thingazar Creek Compared with INWQS Class IIB Recreational Use

3) Electrical Conductivity (EC): Conductivity is a measure of water's capability to pass electrical flow. This ability related to the concentration of ions in the water. These conductive ions come from dissolved salts and inorganic material such as alkalis, chlorides, sulfides and carbonate compounds. For the study area, the EC value is high during May and the EC value is drop to around 0.5 mS/cm (500 µS/cm) in June 23rd (Fig 4). The results of all stations in May are slightly higher when in comparison with World Health Organization (WHO) guideline 1993 of 1.5 mS/cm (1500 µS/cm).

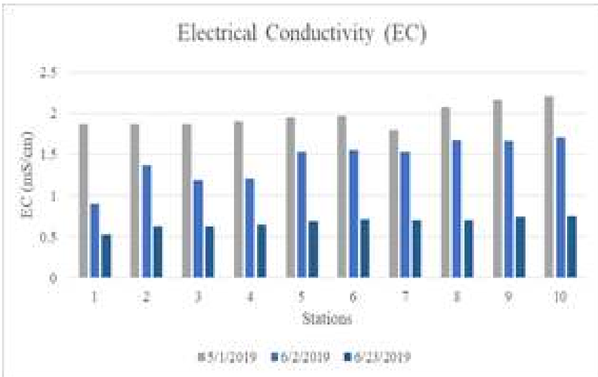

Fig 4. EC Variation of Kan Taw Gyi Lake and Thingazar Creek

4) Salinity: The results of the salinity using Horiba U-52 G is shown in ppt (part per thousand) instead of ppm (part per million). As the salinity of the freshwater is generally less than 1000 ppm which is 10^9 ppt. For the study area, all the results of salinity are less than 1.2 ppt can be observed from Fig 5. Department of Water Government of Western Australia states that the salinity status is brackish if the range is between 1-2 ppt and it is used only for irrigation certain crops only and is useful for most stock. Many freshwater organisms cannot live in salinity level above 1 ppt. The values of salinity drop from 1.1 ppt to 0.4 ppt as days proceed.

Fig 5.Salinity Variation of Kan Taw Gyi Lake and Thingazar Creek

5) Turbidity: Turbidity is a measure of water's lack of clarity, how much the material suspended in water. The results of the turbidity are ranging from 33.7 to 142 NTU. The turbidity of station 7 is generally higher than other stations indicating that the north part of the Kan Taw Gyi is more turbid than the rest of the point. The result obtained on 2nd of June in station 1 is higher than the other two. The color of the water on station 1 is observed to be grey when collected. The limitation of turbidity of INWQS is 50 NTU. All values of turbidity measured are above the limitation except that of station 8. The overall trend is that turbidity value is higher in 23rd of June than the rest of the days. Fig 6(a) shows the turbidity variation and the comparison with INWQS Class IIB is shown in Fig 6(b).

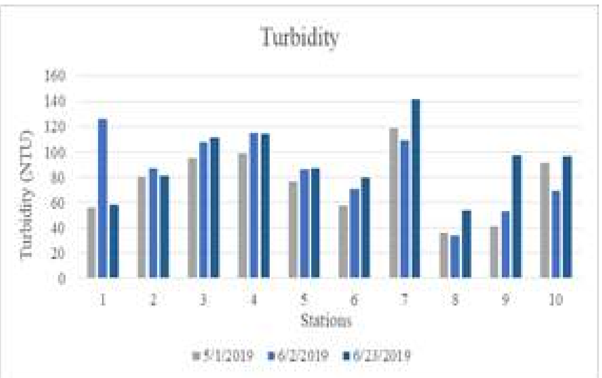

Fig 6(a) .Turbidity Variation of Kan Taw Gyi Lake and Thingazar Creek

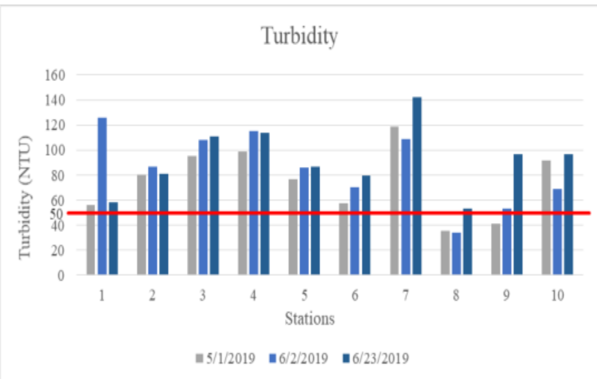

Fig 6(b). Turbidity Variation of Kan Taw Gyi Lake and Thingazar Creek Compared with INWQS Class IIB Recreational Use

6) Total Dissolved Solid (TDS): As measured EC values are high, high value of TDS is resulted and when EC value is low, TDS values are also low in the study area as shown in Fig 7. This is due to the effect of colloids and particles which are smaller than 2.0 µm pore size filter. The total dissolved solids content of potable waters usually ranges from 20 to 1000 mg/L, and as a rule, hardness increases with total dissolved solids. Therefore, the results from samples collected in June is in good condition whereas the results from May lies between 1200 and 1400 mg/L. The palatability of water with a total dissolved solids (TDS) level of less than about 600 mg/L is generally considered to be good according to World Health Organization (WHO) guideline 2010.

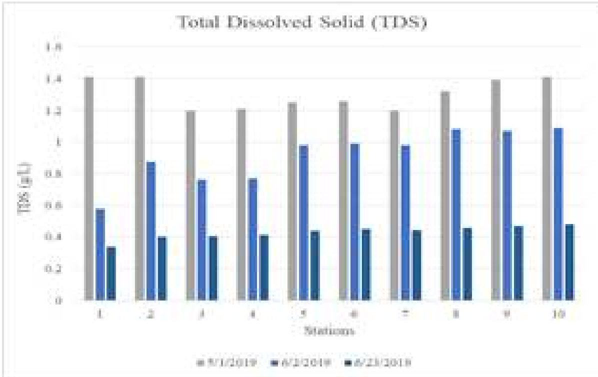

Fig 7.TDS Variation of Kan Taw Gyi Lake and Thingazar Creek

7) Oxidation Reduction Potential (ORP): It is the ability of a lake or river to cleanse itself or break down waste products. The result of oxidation reduction potential (ORP) ranges from 166 to -167 mV (Fig 8.) In Station 6, the ORP reaches -167 mV, this is due to the excessive presence of water hyacinth near the station on 23rd of July than the previous times. The ORP values in the lake are low as the wastewater from the inlet flow into the lake.

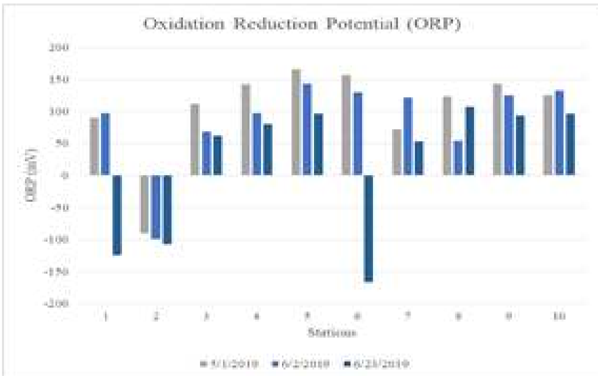

Fig 8. ORP Variation of Kan Taw Gyi Lake and Thingazar Creek

8) Dissolved Oxygen (DO): DO limitations for the lake water in INWQS class IIB is between 5-7 mg/L.

This fact indicates that the lake water may be eutrophic. On 23rd of June 2019, the result of station 7 is 9.22 mg/L of DO. The temperature is high and eutrophication may occur during the day time with maximum sunlight available, therefore, the high DO value is resulted unlikely to the other values as in Fig 8. As station 1 and 2 are at Thingazar Creek, it is normal that the DO value are generally lower than others. For Station 6 and 8, the DO results are low due to the presence of water hyacinth. Fishes die in the morning is observed near Station 6, 7 and 8. These are due to the excessive growth of water hyacinth. As the month proceed, the increase of DO is observed due to the removal of water hyacinth by MCDC. The in-lake stations from Station 7 to 10, the DO increases in different intervals the samples are measured. In-lake Station 4, 5 and 6 have the higher DO in 1st of June than the rest of the months.

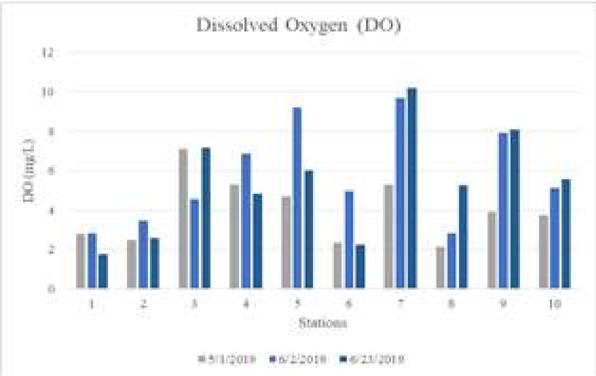

Fig 9(a). ORP Variation of Kan Taw Gyi Lake and Thingazar Creek

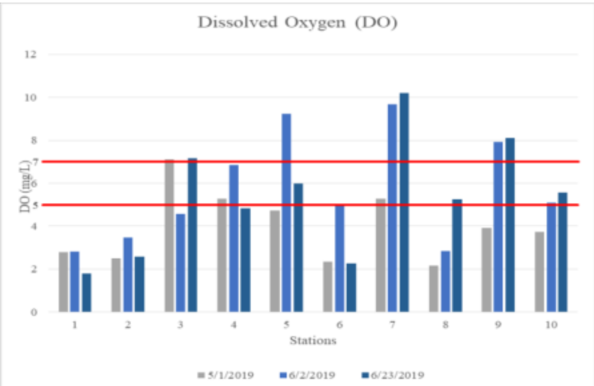

Figure 9(b). DO Variation of Kan Taw Gyi Lake and Thingazar Creek Compared with INWQS Class IIB Recreational Use

9) Biochemical Oxygen Demand (BOD): In INWQS Class IIB limit, BOD limitation is 3mg/L and it is for recreational use body contact. All BOD results are higher than the INWQS limit because wastewater from Thingazar Creek contains many impurities, garbage from urban runoff and some organic matter from the decaying of some organisms. High BOD value means higher oxygen consumption in the process of organic matter decomposition by bacterial and not suitable for domestic uses. The study is not used for water supply and not allowed to go swimming; therefore, it is necessary that the BOD limitation greater than 3 mg/L is not effective. The BOD of Thingazar Creek is generally higher than the BOD values in the in-lake station. BOD value is directly linked with dissolved oxygen, higher BOD value is indirect indication of lesser DO and higher biodegradable organic matter. The above facts clearly indicate the eutrophic condition of water. It can also observed that BOD is considerably higher than the BOD limitation of INWQS.

10) Chemical Oxygen Demand (COD): COD is the total measurement of all chemicals in the water that can be oxidized and high value of COD result shows in the presence of many chemicals and ions in the water. The COD test can be used to target a specific BOD range. The permissible limit of COD for recreational uses is 25 mg/L of COD in Intrim National Water Quality Standards (INWQS) Class IIB limit of Malaysia In many stations inside the lake of the study area, COD values are much higher than the standard limits. These can cause the deterioration upon the future water quality in the long-term.

11) Total Nitrogen (TN): Total nitrogen is the sum of total nitrogen (ammonia, organic and reduced nitrogen) and nitrate-nitrite. The results of total nitrogen are ranging from 4.79 to 5.78 mg/L in lake and 9.9 to 11.2 mg/L at the Thingazar Creek. These values are greater than algal growth range of 0.601 mg/L to 1.5 mg/L total nitrogen. The total nitrogen values greater than 0.6 mg/L can cause eutrophication. These resulted values show that lake has the eutrophic and hypereutrophic conditions.

12) Total Phosphorus (TP*):* Phosphorus is a naturally occurring component of aquatic systems and it is necessary for a balanced ecosystem. For the study area, the total phosphorus values are between 1.27-4.49 mg/L in lake water and 3.71-4.22 mg/L in Thingazar Creek. The total phosphorus limitation for the algal growth rate in the cases of eutrophication in lake water is greater than about 0.1 mg/L. Therefore, the study area is in the state of mesotrophy and eutrophy.

13) Trophic state

Trophic state of the lake has been developed based on the Carlson's Trophic State Index using phosphate data in this study. The TSI index ranges from 0 to 100 and can be used to assign a trophic state "grade" to a lake. Ranges of TSI values can be grouped into the traditional trophic state categories. Lakes with TSI values less than 40 are usually classified as oligotrophic. TSI values greater than 50 are generally defined as eutrophic lakes. Mesotrophic lakes have TSI values between 40 and 50. For the study area, the TSI value of the both in-lake stations and the stations of Thingazar Creek are greater than 70. These TSI conditions depend on the concentration of nutrients from domestic wastewater which are entering the lake. It also depends upon the sources of pollution where are near the point sources and human contact area. Therefore, Kan Taw Gyi Lake is a hypereutrophic lake.

5. CONCLUSION

The study area is one of the lakes which are located in Mandalay City. In this study, some physico-chemical parameters from ten sampling stations with spatial variation upon pollution sources have been analyzed three times during May and June 2019. The parameters such as pH, DO, BOD, COD and turbidity values achieved from the stations have been compared with INWQS Malaysia standard, Class IIB limit (recreational uses) for the water quality. The surface water temperature of Thingazar Creek and lake is constant ranging between 28 to 35°C. As the samples are taken in consecutive months, the surface water temperature do not have much variation throughout the time the samples were measured. For the in-lake stations, only pH is in the acceptable limit of the INWQS Class IIB limit. Electrical conductivity and total dissolved solids taken on 23rd June are within the limit of WHO guideline, 1993. Both BOD and COD are higher than the compared standard limit. The nutrients level taken at both inlet and in-lake are higher than the acceptable limit of algal growth. The analyzed results of total phosphorus indicate that the quality of lake water is relatively high in nutrient concentration. According to the analysis, the presence and absence of water hyacinth effect the parameters of DO, ORP, and turbidity, EC, TDS, BOD and COD. The effect of water hyacinth is reaching alarming problems in many parts of the lake. The TSI values of all the stations indicate that the lake is hypereutrophic. Results of the physico-chemical parameter obtained in this study could be used as a baseline data for further comparative study.

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REFERENCES

[1] “Mandalay Urban Services Improvement Project: Environmental and Social Monitoring Report (April-September 2018)”. (2019)

[2] Google Earth, "Kan Taw Gyi Lake", 21º 56' 37.43" N, 96º 03' 52.22" E. (2018)

[3] Minnesota Pollution Control Agency, “Standard Operating Procedures, Lake Water Quality Sampling”. (2018)

[4] WHO, World Health Organization, Geneva, Switzerland, “Guideline for Drinking Water Quality" 3rd Edition. (2010)\

[5] Cooke, G.D., Welch, E.B., Peterson, S.A., and Nichols, S.A., “Lake and Reservoir Restoration”. (1986)

[6] Carlson, R., E., “A Trophic State Index for Lakes,” Limnology and Oceanography, Vol. 22, No. 2. (1977)

[7] Delzer G.C.,Mckenzie S.W., “Five-day Biochemical Oxygen Demand” USGS TWRI Book 9_A7, 3rd Edition.(2003)

Design of Water Distribution System for Kyaukse

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ABSTRACT*:* This paper presents the design of water distribution system for Kyaukse city. Kyaukse city is located in Mandalay Region of Central Myanmar. The source of water for distribution is taken from Zawgyi River flowing through the city. The reservoir is placed upon the ground level and water is distributed by gravity system throughout the Kyaukse City. In water supply system, continuous system is taken and for a water supply scheme, the dead-end or tree system is used. The design and analysis of pipe line network are carried out by using EPANET 2.0 network modeling software. The results show that the flow and velocity of water in each pipe and head and pressure at each node of the study area.

KEYWORDS: ***distribution, source of water, continuous system, gravity system, EPANET 2.0***

1. INTRODUCTION

Water distribution is one of the most important aspects of environmental engineering to construct a developed country. The water supply or distribution system is the essential link of the water supply source and the receiver. This study has six quarters and population rate is high. So, it is required to meet the demand of water. Zawgyi River is flowing through the city, so the water distribution system is used. It is also essential for house hold, business and industry. The purpose of water distribution system is to transport potable water with appropriate quantity and quality from a water treatment facility to residential consumers use as drinking water, water for cooking, water for sanitary conditions, water for bathing and other water use in a domestic environment. Water distribution is accomplished by the use of surface water, ground water and rain water as source of water. Surface water is the water on the surface of the planet. The methods of water distribution are natural method and artificial method.

2. THEORETICAL BACKGROUND

2.1. Location of Study Area

The study area is selected as Bawga Wadi, West bank of Zawgyi River and Zaey Tan, Su Gyi, Phaung Ywa, Pyi Lone Naing and Min Yatt quarter of Kyauksae City that lying North bank of Zawgyi River and it's located at 21˚ 36' 47" N 96̊ 7' 49" E. The total study area is 0.31 sq.miles or 198.4 acres or 80.28 hectares.

2.2. Geometrical Increase Method

From the population data of pervious three or four decades, the percentage increase in population is found and its average is found. The future population P_n after n decades is given by

$$P_{2049} = P_{2019}\left(1+\frac{I_g}{100}\right)^n \qquad (1)$$

where, P_n = future population at the end of n decade
P = present population
I_g = percentage of increment per decade
n = number of decades

This method gives higher results since the percent increase never remains constant but, instead, decreases when the growth of the city reaches to saturation.

2.3. Maximum Daily Demand (MDD)

The maximum daily demand (MDD) represents the maximum consumption during any one day of the year. The maximum day peaking factor is expressed as a ratio of the maximum daily demand divided by the average daily demand (ADD). The ratio generally ranges from 1.2 for very large water systems to 3.0 or even higher for specific small systems. For this study, a peaking factor of 1.5 is used to determine future MDD value. Dr.B.C (1995). Water demand for public uses is 13 lpcd.

2.4. Fire Demand

Water required for firefighting is usually known as fire demand. It is treated as a function of population and may be computed from the following equation. In this study, the duration of fire is considered as the recommended values of 10 minutes in a day. The water demand for institutional need is considered as 20% of the demand based on per capita consumption.

National Board of Fire Underwriter's Formula is given by:

$$Q = 4637\sqrt{P}\,(1 - 0.01\sqrt{P}) \qquad (2)$$

where, Q = quantity of water for firefighting (l/min)
P = population in thousands

2.5. Friction Loss and Velocity of Pipe

The amount of friction that must be overcome varies directly with roughness of the surface of the pipe or channel, and the length of pipe. The total head loss consists of friction head loss in the pipe line plus the minor losses produced due to flow through various fittings such as bends elbows, tees, valves etc. Dr. B.C. PUNMIA (1995). The friction loss can be calculated by the following equation.

$$h = \frac{fL\,V^2}{2\,gD} \quad (3)$$

where, h = friction head loss (m)
f = friction factor
L = length of pipe (m)
D = pipe diameter (m)

The Manning's formula is commonly used in distribution of water and that Manning (N) value of 0.015 can be used to determine the velocity. Manning formula is given by:

$$V = \frac{1}{N} R^{2/3} S^{1/2} \quad (4)$$

where V = velocity of flow (m/s)
R = hydraulic radius (m)
S = slope of pipe
N = roughness coefficient

2.6. Power Requirements of Pumps

The common unit of power i.e, the rate at which work is done. The electrical equivalent of one horsepower is 746 watts in the International System of Units (SI), and the heat equivalent is 2545 BTU (British Thermal Units) per hour. Another unit of power is the metric horsepower, which equals 4500 kilogram-meters per minute (32,549 foot-pounds per minutes), or 0.9863 horsepower. If Q is the volume of water lifted in one second through a height of H metres, the horse power of the pump is

$$\text{Horse power (H.P)} = \frac{Q\,w\,H}{75\,\eta} \quad (5)$$

where Q = quantity of water (m^3/s)
w = unit weight of water (kg/m^3)
H = total head (m)
η = efficiency of pump or motor (80%)

The estimated horse power is 23 HP.

2.7. Computer Modeling of Pipe Network Analysis

The EPANET computer program was developed by the U.S. EPA (Environmental Protection Agency). The program computes the flow rates in the pipes and then HGL at the nodes. In this study, EPANET 2.0 network modeling program is used to analyze the distribution networks for selected area. EPANET 2.0 network is a computer program that models and simulates hydraulic and water quality conditions within pressurized water distribution networks. It models a water distribution system as a collection of links connected to nodes. The links represent pipes, pumps and controlled values. The nodes represent junctions, tanks and reservoirs. A network consists of pipes, nodes, pumps, valves and storage tanks or reservoirs. EPANET 2.0 network tracks the flow of water in each pipe, the pressure of each node, the height of water in each tank. The calculation of flow rates involves several iterations because the mass and the energy equations are nonlinear. The number of iterations depends on the system of network equations and the user-specified accuracy. A satisfactory solution of the flow rates must meet the specified accuracy, the law of conservation of mass and energy in the water distribution system, and any other requirements imposed by the user. Once the flow rate analysis is complete, the water quality computations are then performed.

The EPANET input data file, create automatically by MIKE NET, includes descriptions of the physical characteristics of pipes and nodes, and the connectively of the pipes in a pipe network system. The user can graphically layout the water distribution network, if desired. MIKE NET then creates the EPANET input data file into the format required to run the software analysis. The pipe parameters include the length, inside diameter, minor loss coefficient, and roughness coefficient of the pipe. Each pipe has a defined positive flow direction and two nodes. The parameters of nodes consist of the water demand or the water supply, elevation and pressure or hydraulic grade line.

To run this software, the following inputs of each node (junction) and link (pipe) are required.

Required input data for nodes (junctions) are
(1) Node ID number
(2) Elevation and
(3) Base demand

Required input data for pipes (links) are
(1) Link ID number
(2) Length
(3) Diameter (assume) and
(4) Roughness coefficient

3. DATA PREPARATION

The study area is 'Kyaukse' in Mandalay Region and Zawgyi River is chosen as the source of water supply. In the case of several elevations, it is taken from Google map. Design period of study area is taken as 30 years for future water demand. The population increment rate is taken as 1.4921%. The geometrical increment method is used to determine the future population. The present population of the study area is 19751 persons in year 2019 and 29977 persons in year 2049. Water demand for local people in the area is 135 liters per capita per day. The fire demand is 239980 l/d, calculated by National Board of Fire Underwriter's Formula. The cast iron (CI) pipeline network is designed by using EPANET 2.0 software.

3.1. Calculation of Design Population

Present population = 19751 persons
No. of decades = 3
Population increasement rate = 1.4921 % per year

$$P_{2049} = P_{2019}\left(1 + \frac{I_g}{100}\right)^n$$

$$P_{2049} = 19751\left(1 + \frac{14}{100}\right)^3$$

P_{2049} = 29977 persons

Table 1. Design Population for Study Area

Sr.	Year	Population
1	2019	19751
2	2029	22697
3	2039	26084
4	2049	29977

Table 2. Roughness Coefficient (N)

Sr.	Material	Roughness Coefficient (N)
1	Brick	0.012 - 0.018
2	Concrete-pipe	0.011 - 0.015
3	Cast iron pipe	0.022 - 0.026
4	Plastic pipe	0.011 - 0.015

3.2. Calculation of Water Consumption

Daily Demand = Population × Per capita consum:
= 29977 × 135 lpcd
= 4046895 lpd

Public Use = 29977 × 13 lpcd
= 389701 lpd

Fire demand $= 4637\sqrt{P}\,(1 - 0.01\sqrt{P})$
$= 4637\sqrt{29.977}\,(1 - 0.01\sqrt{29.977})$
= 23998 lpm
= 23998 lpm × 10 min/day
= 239980 lpd

Total Demand = Daily Demand +Public Use + Fire Demand
= 4046895 + 389701+ 239980
= 4676576 lpd

Maximum Daily Demand = 1.5 × 4676576
= 7014864 lpd

In this study, the total demand is 7014864 lpd for all requirements. Continuous system is used. So, the duration which water is supplied is 24 hours. Average hourly demand is calculated by following.

Average hourly demand $= \frac{7014864}{24}$
= 292286 liters / hr
$= 292.286\ m^3/hr$

4. DESIGN RESULTS

EPANET 2.0 modeling software is used for the method of distribution system because it is more convenient than other software. It is used to design the proposed pipe network plan. The value of base demand is taken as an average value of daily maximum demand on each node. The EPANET software gives head, pressure, flow and velocity of pipe. These results are shown in Fig 1 to 12.

Demands of nodes are directly proportional to the distance between them and population near it. It is considered that the base demands all of nodes are equal. Base demand is taken as average value of daily average demand of the study portion.

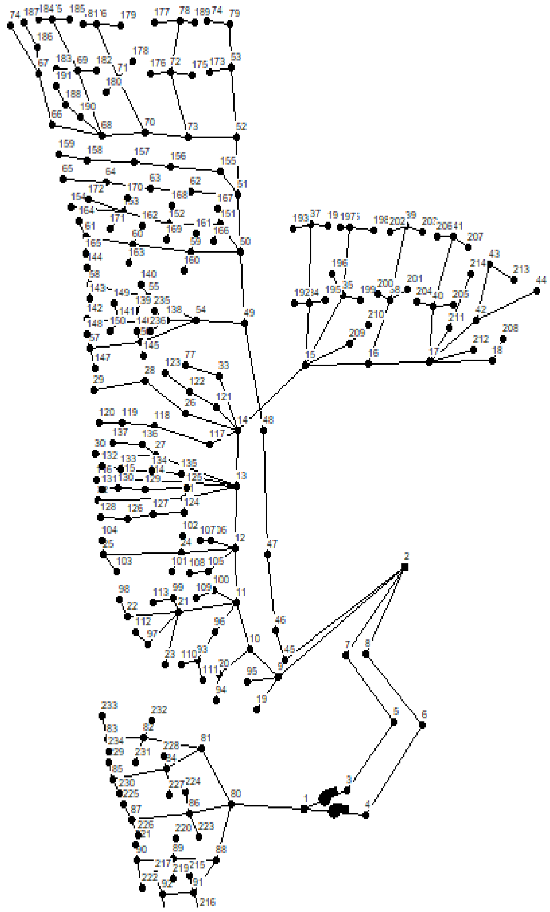

Fig 1. Pipe Network Plan (Displayed Node's I.D)

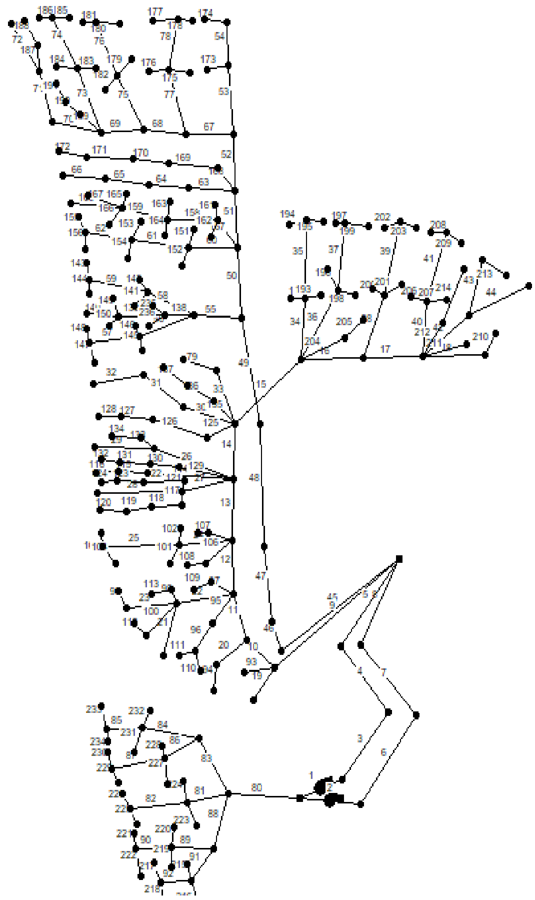

Fig 2. Pipe Network Plan (Displayed Link's I.D)

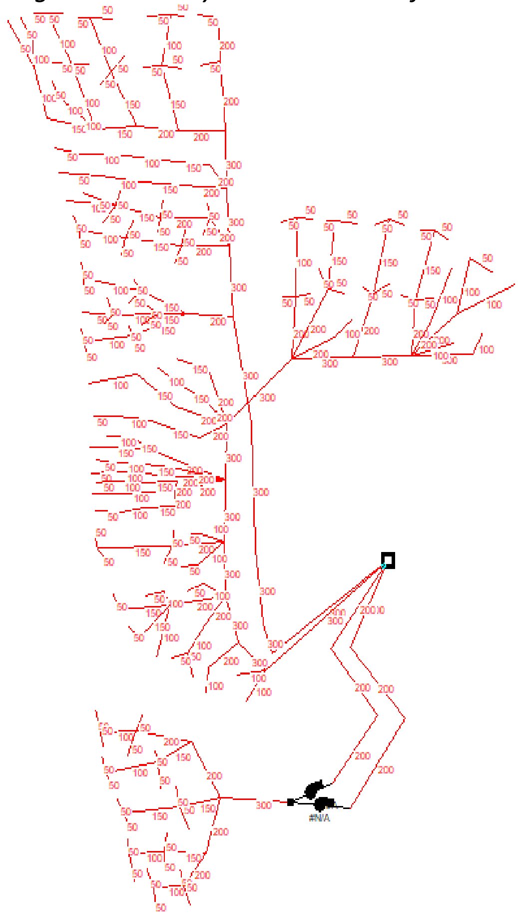

Fig 3. Diameter of Pipe Size in Study Area

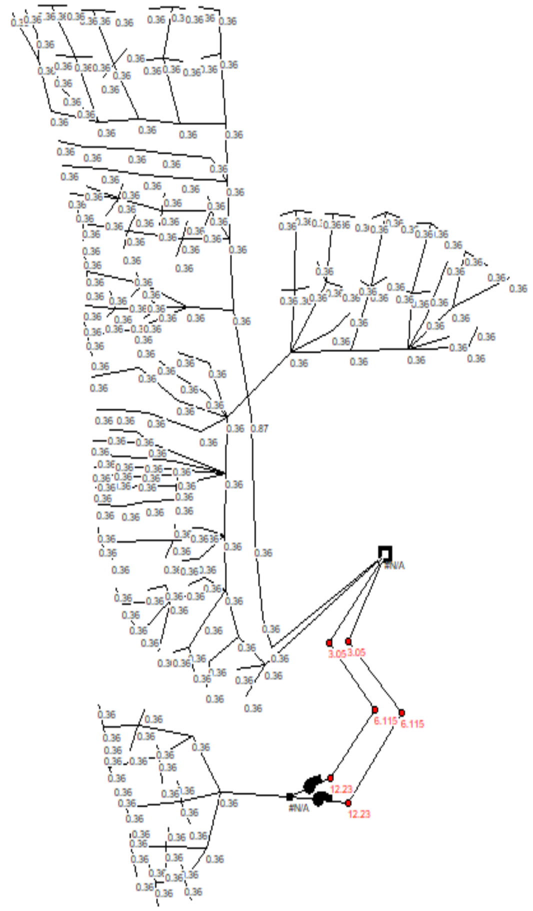

Fig 5. Base Demand of Nodes in Study Area

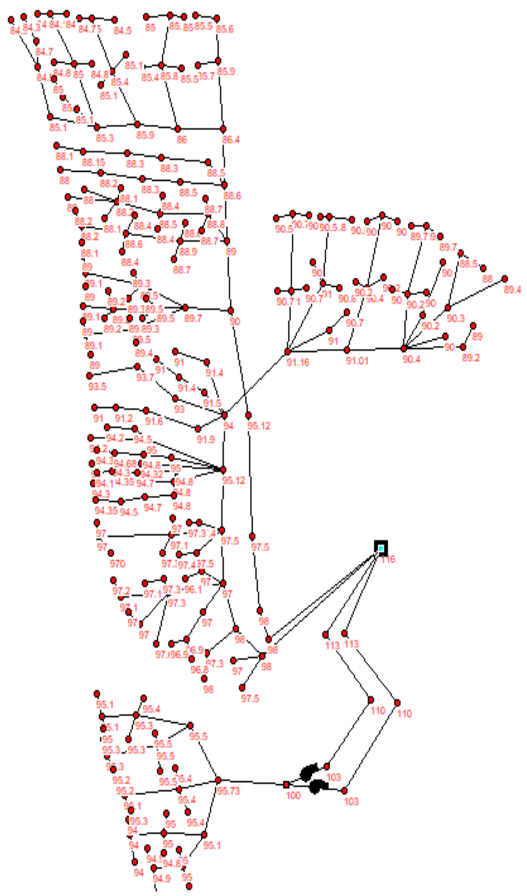

Fig 4. Elevation of Nodes in Study Area

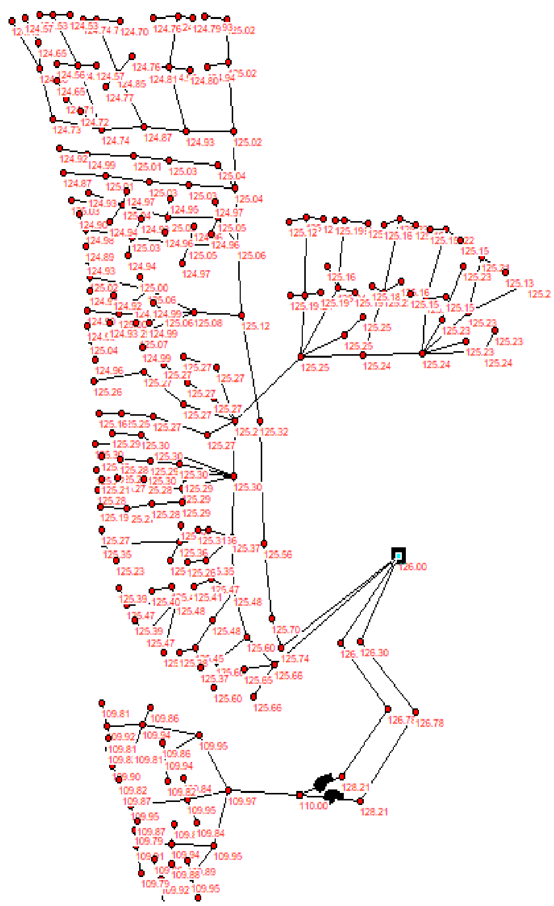

Fig 6. Head of Nodes in Study Area

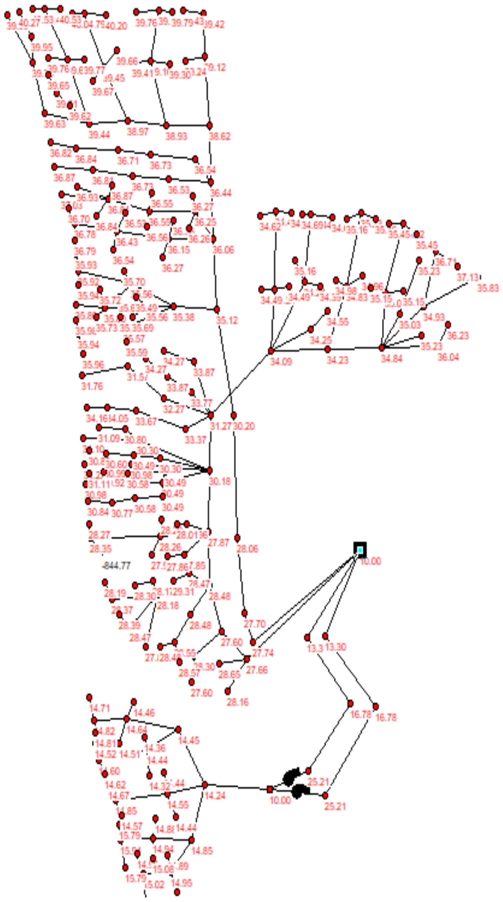

Fig 7. Pressure of Nodes in Study Area

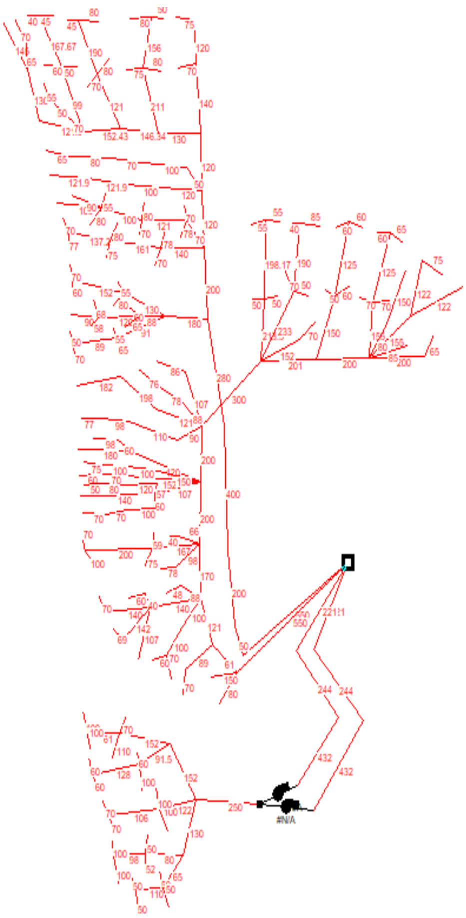

Fig 8. Lengths of Links in Study Area

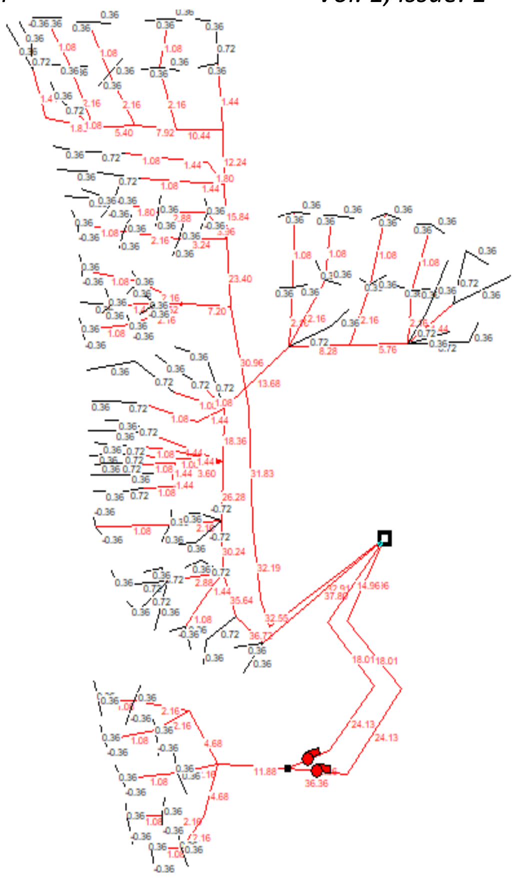

Fig 9. Flows of Links in Study Area

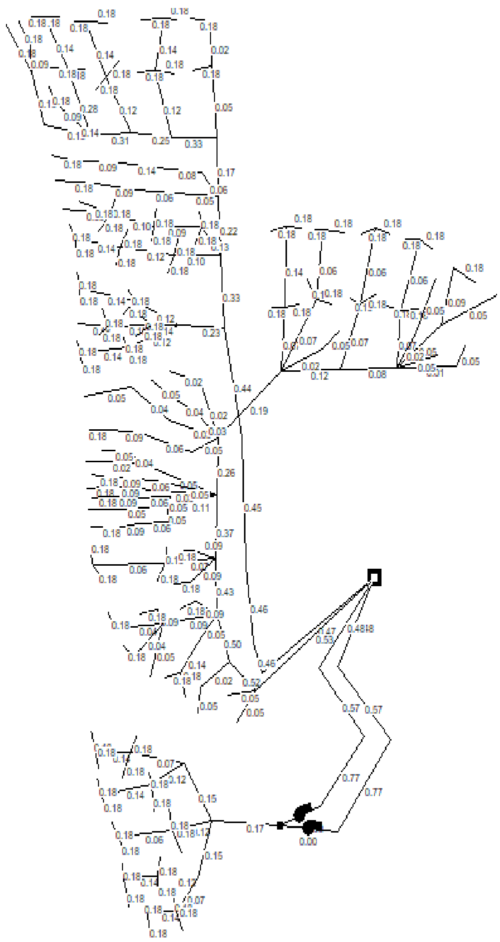

Fig 10. Velocity of Links in Study Area

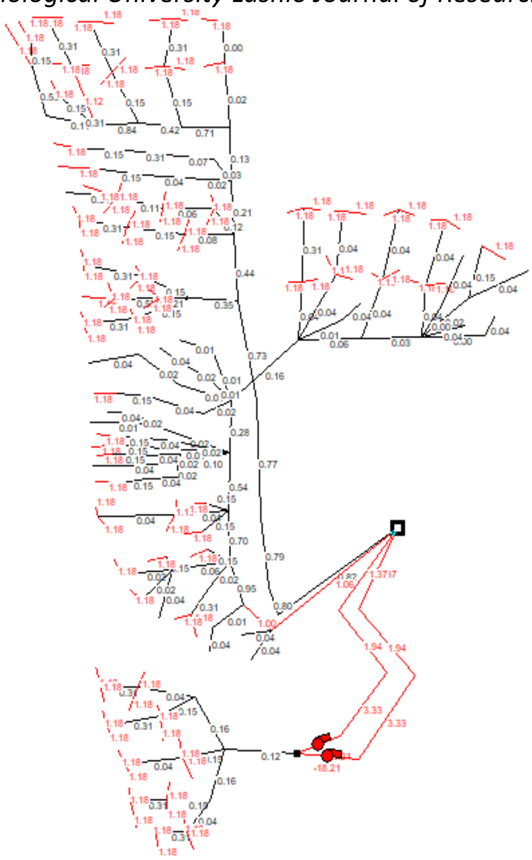

Fig 11. Unit Head Losses of Links in Study Area

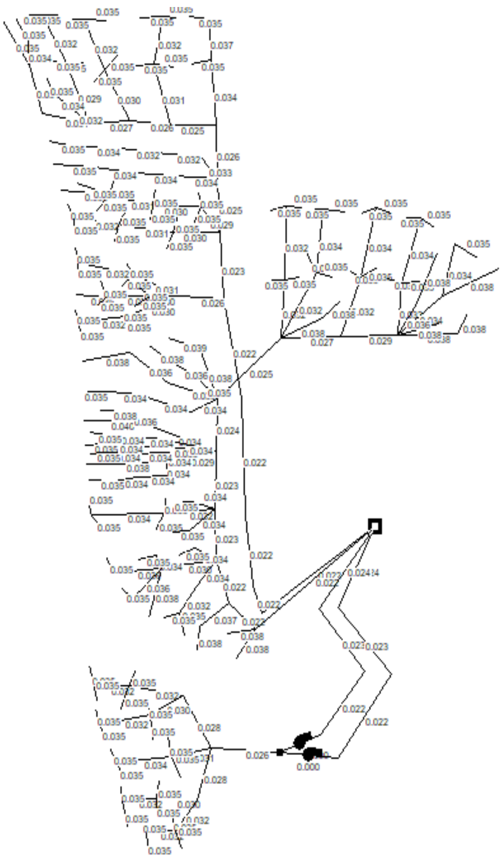

Fig 12. Friction Factors of Links in Study Area

In designing water distribution system, pressure requirements for ordinary use and fire-fighting must be considered. The American Water Works Association recommends 20 psi in the house service pipe and plumbing, a total pressure of about 35 psi in the main is adequate for residential districts with one and two-story houses and 75 psi should be satisfactory for buildings up to 10 stories in height. The distribution system should not be designed for residual pressures more than 22 metres, so multi-storied buildings needing higher pressure should be provided with booster pumps. Fig 5, 6 and 7 show the demand, head loss and pressure of proposed pipeline network, respectively. Fig 9 and 10 show the flow and velocity of proposed pipeline network running in EPANET 2.0 software, respectively.

Fig 13 shows the demand of node 17 of proposed pipeline network in study area. Fig 14 to 16 show the unit head loss, velocity and flow for link 15, 35 and 48, respectively of proposed pipeline network in study area.

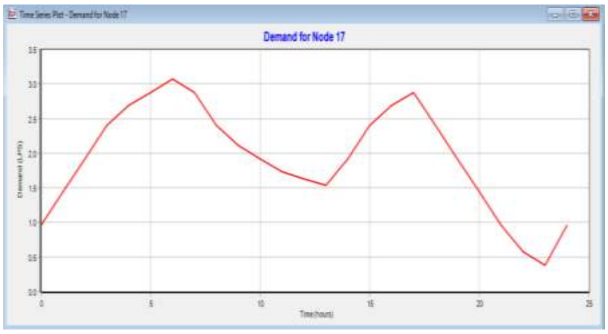

Fig 13. Demand of Node 17 of Proposed Pipeline Network in Study Area

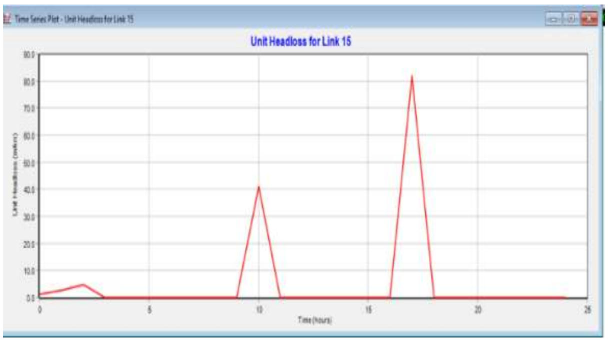

Fig 14. Unit Headloss for Link 15 of Proposed Pipeline Network in Study Area

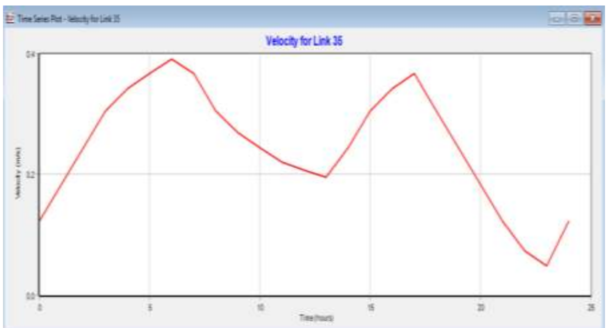

Fig 15. Velocity for Link 35 of Proposed Pipeline Network in Study Area

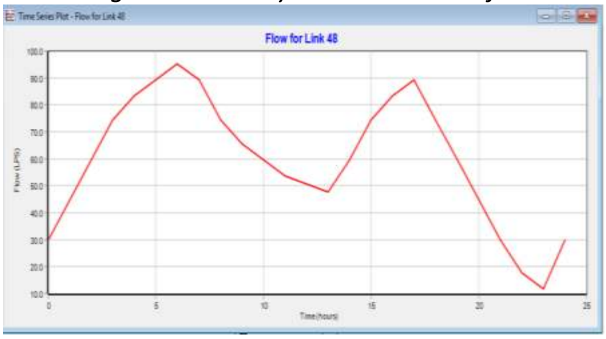

Fig 16. Flow for Link 48 of Proposed Pipeline Network in Study Area

The demand, head loss, velocity and flow for various links or nodes may be showed. But, Node 17, Link 15, Link 34 and Link 48 are described as samples in this paper.

5. CONCLUSION

The maximum water demand of 438 m^3 is required to store in the tank. The dimension of storage tank is radius of 6.5 m and high of 3.3 m. The capacity of this underground storage tank is designed only based on the maximum capacity of storage tank. There are 234 nodes and 236 links with diameter of 300 mm (main), 200 mm (branch), 150 mm (branch), 100 mm (branch) and 50 mm (stand pipe). Centrifugal pump is used in ground tank for distribution of water with pump capacity 80% for full load. It was designed to deliver 438 cubic meter per hour at a head of 21.33 m.

ACKNOWLEDGEMENT

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REFERENCES

[1] Comparison on Existing and Future Water Distribution System in TAMWE TOWNSHIP by MA SHWE SIN MIN AUNG - OCTOBER, 2018 YANGON.

[2] Principles of Water Resources Engineering by Kharagpur.2010.

[3] EPANET User 2 Manual by Lewis A. Rossman. 2000.

[4] Water supply engineering by Dr. B.C. PUNMIA, ASHOK KUMAR JAIN and ARYN KUMAR JAIN. 1995.

[5] Element of Public Health Engineering by K.N. DUGAL.1983. (second edition).

Experimental Research on Properties of Fired Clay Brick Mixed with Waste Glass

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ABSTRACT*:* Solid waste accumulation is growing enormously as the modernization is growing. Solid waste includes organic waste (food waste) and inorganic waste (glass, plastic, etc). The waste glasses become a significant solid waste type in the municipal solid waste. Therefore, waste glass can be used in the production of fired clay brick to reduce the environmental and to help energy saving. This paper is to investigate the effect of mixing waste glass particles in the fired clay bricks. The waste glass particles are mixed in the clay particles of fired brick. Two different particle sizes (150 μm and 600 μm) of waste glass particles are mixed to clay particle at contents of 15% and 30% by weight. Brick samples are fired at 1100ºC in the furnace for 7 hours. The physical and mechanical properties of mixing waste glass bricks are studied and compared with the normal clay brick. The bulk density and compressive strength of bricks increase as the amount of waste glass content increases. The firing shrinkage, apparent porosity and water absorption decrease with the increase of the amount of waste glass. The particle size of waste glass powder was very important and a significant effect on the properties of fired clay bricks, the particle size of 600 μm get more compressive strength than other. Based on the executed testing program, it is found that the optimal properties of fired clay bricks are achieved at 30 % content of waste glass.

KEYWORDS: *brick, waste glass, physical properties and mechanical properties.*

1. INTRODUCTION

Worldwide, bricks are a major building material and perhaps one the oldest. The worldwide annual production of bricks is currently about 1391 billion units and demand for bricks is predicted to be continuously increasing. (Zhang,2013) Clay materials are mostly used for the manufacture of bricks. Waste can be added in order to enhance its properties. Solids waste is a great concern among governmental agencies, and environmentalist regarding the increasing amounts of waste throughout the world.

One waste material which has a potential as a brick additive is waste glass. It is not biodegradable and therefore it creates a problem for solid waste disposal. The disposal into landfills also does not provide an environment-friendly solution. Hence, the use of waste glass as a construction material is a practical solution to the environmental problems caused by this solid waste. Brick is a building material used to make walls, pavements and other elements. Bricks are produced in numerous classes, types, materials and sizes which vary with region and time period. A brick can be composed of clay soil, sand and lime or concrete materials. Clay materials are mostly used for the manufacture of bricks and the waste can be added in order to its properties. The use of waste glass in the construction materials can be worthy solution to the environmental problem caused by this solid waste.

2. Brick Manufacturing process

2.1 Raw material

The raw materials used in the manufacturing process of fired clay masonry units are a mixture of natural clay, silt, and sand. The surface clays (recent sedimentary formations), Shales (clays that have been subjected to high pressures) and fire clay (mined at deeper levels) are commonly used in the production of fired clay units. Surface and fire clays have a different physical structure from shales but are similar in chemical composition.

The two main constituents of all of these clays are the silica and alumina. Some minor components are iron and other metal oxides, which are particularly responsible for giving brick its red-brown color.

The range of chemical component and mineralogical phases present in clay as follows;

Table 1. Range of Chemical Component Present in Clay

Property Chemical composition, wt.%	Fireclay Brick
SiO_2	56.8 – 68.6
Al_2O_3	22.9 – 38.7
Fe_2O_3	0.8 – 3.0
K_2O	1 – 3.2
TiO_2	1 – 2.8
MgO	0.1 – 1.2

Table 1. Range of Chemical Component Present in Clay Continued;

Na_2O CaO	0.2 – 0.5 0.01-0.8
Phases identified Quartz	Trace – major
Mullite	Minor – major
Cristobalite	None – major
Hematite	None – trace
Rutile	None – trace
Amorphus	Minor - major

Walter Lee Sheppard (1986)

2.2 Types of glass

Nearly all commercial glass fall into one of six basic categories or types. These categories are based on chemical composition. Within each type, except for fused silica, there are numerous distinct compositions.

Table 2. Approximate composition of glasses

Types of Glass	Compositions (by weight)
Soda-Lime-Silica Glass	73% Silica – 14% Soda – 9% Lime – 4% Magnesia – 0.1% Alumina
Lead (Crystal)	57% Silica – 31% Lead Oxide – 12% Potassium Oxide
Borosilicate	81% Silica – 13% Boron Oxide – 4% Soda – 2% Alumina.
Aluminosilicate	64.5% Silica – 24.5% Alumina – 10.5% Magnesia – 0.5% Soda
Ninety-six percent silica glass	borosilicate glass
Fused silica glass	pure silicon dioxide in the non-crystalline state

Shelby (2005)

2.3 Preparing of raw materials

Clay material used in this research was obtained from the Ayeyarwaddy Riverside. The waste glass was generated in Magway is thrown away into the dump areas. The raw materials were initially subjected to preparation such as drying, milling and sieving; prepared by powdering in a laboratory to particle size 600 μm for both clay and waste glass for brick production. Another particle size (150 μm) of waste glass was prepared to compare with 600 μm.

3. EXPERIMENTAL PROGRAM

The experimental program for this research study is primarily concerned with investigating the potential usefulness of using waste glass in making fired clay bricks. Currently, the waste glass generated in Magway is thrown away into the dump areas. Waste glass usually is produced from empty glass containers and different construction and reconstruction remains and waste materials.

The waste glass is to be milled into powder. Then the powder glass is mixed with raw materials to make fired clay brick and then observing the effect of recycled powdered glass on the properties of fired clay brick.

The use of waste glass as a clay body additive is therefore attractive both environmentally and economically. Adding waste glass to bricks not only reduces the consumption of clay raw material, diverts waste from landfills, it also provides potential profit in tipping fees for manufacturers. In additions to mining, transportation, and storage costs, the firing process is an aspect of brick production which is energy intensive. By reducing the firing temperature by even a few degrees, while still maintaining brick strength and durability, the energy requirements of the firing process could be reduced.

The experimental program of the current research was carried out to explore the effects due to the use of waste glass and the firing behavior and physical mechanical properties following the testing procedure specifications of ASTM.

All materials used in this study are locally available. Clay is to be used in this investigation with (15% and 30%) of waste glass powder as a partial replacement for clay.

The successful use of waste glass will aid in reducing the environmental and health problems related to the disposal of waste glass and the scarcity of land area needed for disposal. Reducing waste is not only reason to investigate the addition of certain residues into a clay matrix, although traditionally it has been the main purpose of research on this topic. Other reasons may be considered, such as saving energy required in the manufacturing process and reducing the manufacturing cost.

3.1 Testing program

For the testing program, five brick samples that listed previously in Table 3.1 are prepared. These five brick samples are divided into two categories, one to test compressive strength and another to tests water absorption, firing shrinkage, bulk density and apparent porosity. A total number of 25 testing data points were used after controlling the compiled testing cases from data quality and completeness points of views.

3.2 Testing method for the physical and mechanical properties

The brick samples then underwent series of tests including water absorption, bulk density, apparent porosity, firing shrinkage and compressive strength, to determine their quality in comparison with the ASTM Standards methods.

3.2.1 Physical properties

Several physical properties of the fired clay bricks can be determined: (1) linear shrinkage,(2) water absorption, (3)apparent porosity, (4) bulk density, apparent density, and loss on ignition. In this research, only the first four properties were considered.

(1) Linear shrinkage

Linear shrinkage was obtained by measuring the length of the sample before and after drying or before and after firing, or even over the whole process using a caliper with a precision of ±0.01 mm, according to the standard ASTM C326 (ASTM C326- 09, 2014). The firing linear shrinkage (before and after firing) expressed as a percentage and calculated according to the following formula is presented, as these results were more readily available

$$\text{Firing shrinkage (\%)} = \frac{L_{dried} - L_{fired}}{L_{dried}} \times 100 \qquad (1)$$

Where L_{dried} is the length of the oven-dried sample (mm) and L_{fired} is the length of the fired sample (mm). Firing shrinkage was determined by measuring the physical dimensions of the specimens before and after firing.

The bulk density, Water absorption, and apparent porosity were measured according to Archimedes method described in standard ASTM C373 (ASTM C373-88, 2006), which involves drying the test specimens to constant mass (D), boiling them in distilled water for 5 h and soaking them for an additional 24 hours, at ambient temperature. Following impregnation, the mass (S) of each specimen while it was suspended in water, and its saturated mass (M), were determined.

(2) Bulk density

The bulk density and the apparent density change (not including porous). Bulk density, in grams per cubic centimeter, of a specimen is the quotient of its dry mass divided by the exterior volume, including pores. Calculate the bulk density as follows:

$$B = \frac{D}{V} \times 100 \qquad (2)$$

where , V (cm^3) the exterior volume (V = M – S).

D = dried mass

M = saturated mass

S = suspended mass in water

(3) Water Absorption

The creation of porosity leads to an increase in water absorption. The voids in the structure while immersed, are filled with water and, depending on the arrangement of the pores and the way they are linked together, this can penetrate the material more or less easily, with a preferential pathway.

For water absorption, no standardized maximum value exists. However, a very large absorption capacity could be detrimental for the final brick as it may affect the durability of the product and its resistance to natural conditions.

Water absorption expresses as a percent, the relationship of the mass of water absorbed to the mass of the dry specimen. Calculate the water absorption as follows:

$$A = \frac{M-D}{D} \times 100 \qquad (3)$$

Where, D = dried mass

M = saturated mass

(4) Apparent Porosity

During drying and firing, the added particles burn leaving voids due to their breakdown, but also to gas emissions by decomposition of the matter (water, carbon dioxide). Noteworthy, there is no fixed maximal porosity for fired clay bricks. Apparent porosity expresses, as a percent, the relationship of the volume of open pores of the specimen to its exterior volume. Calculate the apparent porosity as follows:

$$P\ (\%) = \frac{M-D}{V} \times 100 \qquad (4)$$

Where, V (cm^3) is the exterior volume (V = M – S).

D = dried mass

M = saturated mass

S = suspended mass in water

3.2.2 Mechanical properties

To ensure the engineering quality of a material, especially for building construction use, mechanical testing is the essential criteria. The compression or bending strengths are represent for mechanical properties. The compressive strength was measured for brick samples according to ASTM C67 (ASTM C67-03, 2003).

3.3 Experimental procedures

In order to obtain comparable results, glass samples were prepared for the tests depending on the amount of waste glass added, heating temperature and particles size of waste glass. The mix proportions were prepared based on the dry weight of the materials. Mixture proportions were presented in Table 3. The raw materials, clay, water and waste glass particles were mixed to get a uniform consistency. Test specimens with a dimension 9.2 in (L)

× 4.5 in (W) × 2.7 in (H) were produced in a mould. The shaped samples were dried under sunlight (30°C) for 2 to 10 days according to the weather condition and then fired in an oven.

The samples were naturally cooled down in the furnace. Thus, sufficient samples could be produced from each of the series of samples to perform the experiments. A total of 5 brick samples were prepared to testing purpose.

Table 3. The Brick Mixtures Prepared from the Raw Materials Used

Specimen	C% : WG% (150μm)	C% : WG% (600μm)
0% WG	100:0	100:0
15% WG	85:15	85:15
30% WG	70:30	70:30

C = Clay ; WG = Waste Glass

4. TEST RESULTS AND DISCUSSION

The bricks in this research were made from clay and waste glass by controlling the waste glass ratio and particle size. At least 5 samples were used for each test listed previously in Table 3 and the averages are presented and discussed in this section. The investigated and reported physical and mechanical properties are bulk density, apparent porosity, water absorption, firing shrinkage and compressive strength are shown in the following.

4.1 Test Results

Test results include firing shrinkage, bulk density, water absorption, apparent porosity and compressive. The results are also showed in tables.

Table 4. Average Values of the Firing Shrinkage of the Samples

Waste Glass (%)	Firing Shrinkage (%)
Natural brick(NB)	2.17
15 (150 μm)	1.09
30 (150 μm)	1.01
15 (600 μm)	1.09
30 (600 μm)	0.54

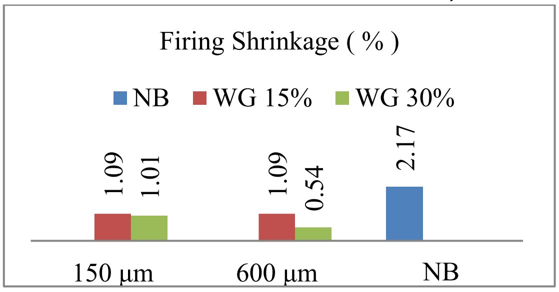

Fig 2 Comparison of Firing Shrinkage

In the figure, natural brick has highest firing shrinkage percentage. Waste glass containing 15% at both 150 μm and 600 μm has higher firing shrinkage percentage than waste glass containing 30%. As a result, there was a decreasing percentage in shrinkage as the amount of waste glass in the mixture increased. It is consider that the reduction firing shrinkage of brick caused a positive effective by decreasing any internal strain that may occur during drying and firing process. Furthermore, a decrease in the shrinkage value is considered to be good for the dimensional stability of the clay brick after firing.

Table 5. Average Values of the Bulk Density of the Samples

Waste Glass (%)	Bulk density (g/cm^3)
Natural brick(NB)	0.085
15 (150 μm)	0.097
30 (150 μm)	0.098
15 (600 μm)	0.092
30 (600 μm)	0.095

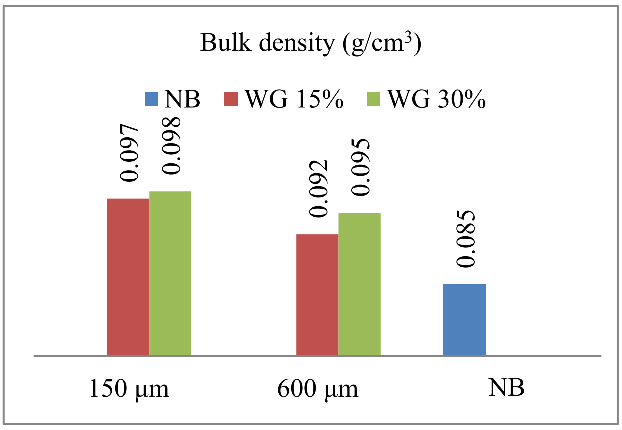

Fig 3 Comparison of Bulk Density

In the figure, natural brick has lowest bulk density. Waste glass containing 15% at both 150 μm and 600 μm has lower bulk density than waste glass containing 30%. The bulk density of fired clay brick was proportional to the amount of waste glass added and their fineness.

Table 6. Average Values of the Water Absorption of the Samples

Waste Glass (%)	Water absorption (%)
Natural brick(NB)	15.35
15 (150 µm)	9.6
30 (150 µm)	8.5
15 (600 µm)	11.31
30 (600 µm)	10.26

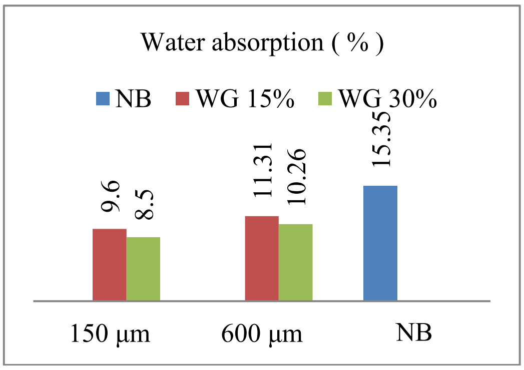

Fig 4 Comparison of Water Absorption

The graph shows the water absorption results for various waste glass content at different fineness .The natural brick has highest water absorption percentage .Waste glass 15% at both 150 µm and 600 µm has higher percentage than waste glass 30%.The water absorption capacity decreased with higher content of waste glass for both fineness. This was due to zero water absorption capacity of glass material. Moreover, it was observed that fineness of glass is also effect the water absorption. The more fineness of glass decreased the water absorption capacity.

Table 7. Average values of the Apparent Porosity of the Samples

Waste Glass (%)	Apparent porosity (%)
Natural brick(NB)	13.90
15 (150 µm)	9.30
30 (150 µm)	8.20
15 (600 µm)	12.78
30 (600 µm)	11.63

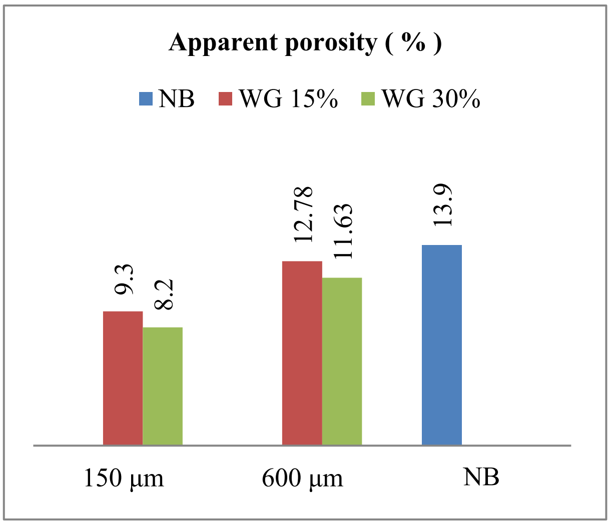

Fig 5 Comparison of Apparent Porosity

In the figure, natural brick has highest apparent porosity percentage. Waste glass containing 15% at both 150 µm and 600 µm has higher percentage than waste glass containing 30%. Apparent Porosity is directly related to the water absorption and a similar tendency was observed in water absorption.

Table 8. Average Values of the Compressive Strength of the Samples

Waste Glass (%)	Compressive strength (Mpa)
Natural brick(NB)	6.28
15 (150 µm)	8.62
30 (150 µm)	9.97
15 (600 µm)	7.38
30 (600 µm)	8.47

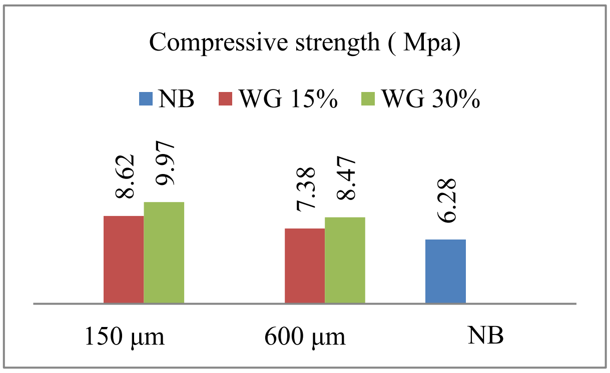

Fig 6 Comparison of Compressive Strength

The compressive strength is the most important index for assuring the engineering quality of a building material because with a higher compressive strength, other properties also improved. The compressive strength of fired clay bricks depended greatly on the amount of waste glass addition and their fineness. The addition of waste glass considerably contributed to verification and enhanced the strength development by closing the internal pores with glassy phase. From the Fig 6, it was observed that natural clay brick has lowest compressive strength. Waste glass containing 15% at both 150 µm and 600 µm has lower compressive strength than waste glass containing 30%. The value rises from 7.36 MPa to 8.62 MPa as 15% waste glass (600 µm) is added and from 8.47 MPa to 9.97 MPa as 30% waste glass (150 µm) is added.

5. CONCLUSIONS

In this study, the viability of production of building brick materials that include waste glass was verified. It was observed that the firing shrinkage of clay brick increased with higher content of waste glass used. Furthermore, the larger particle size lead to decreased firing shrinkage.

The water absorption capacity and apparent porosity of clay brick were decreased with increased content of waste glass. The fineness of waste glass was effect on these decrease, the more fineness the waste glass caused the greater rate of decreased porosity.

The compressive strength of the fired clay bricks increased with increases in the amount of waste glass content. It was also shown that the more fineness waste glass particle size can get more compressive strength.

It can be concluded that there is a similar trend between compressive strength and bulk density. The advantages of lower apparent porosity, lead to less water absorption and gain in compressive strength. The particle size of waste glass powder is very important and has a significant effect on the properties of fired clay brick; the finest the particle size, the highest is the compressive strength. Therefore, the use of waste glass material in brick production provides an economic contribution and also helps to protect the environment.

ACKNOWLEDGEMENT

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REFERENCES

[1] Alam, 2015, Pre-Feasibility report for proposed brick earth excavation project. Samastipur, Bihar: Niranjan bricks H. Simpson, *Dumb Robots*, 3rd ed., Springfield: UOS Press, 2004, pp.6-9.

[2] Amrhein, J.E., & Porter, M.L, 2009, "Reinforced Masonry Engineering Handbook: Clay And Concrete Masonry (6th ed.)". Washington: Masonry Institute Of America MIA. [4] Simpson, et al, "Title of paper goes here if known," unpublished.

[3] ASTM C326-09, 2014, "Standard Test Method for Drying and Firing Shrinkages of Ceramic Whiteware Clays".

[4] ASTM C373-88, 2006, "Standard Test Method for Water Absorption, Bulk Density, Apparent Porosity, and Apparent Specific Gravity of Fired Whiteware Products, Ceramic Tiles, and Glass Tiles".

[5] ASTM C62-04, 2004, "Standard Specification for Building Brick (Solid Masonry Units Made From Clay or Shale)".

[6] ASTM C67-03, 2003, "Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile".

[7] Hisham Hussein Abdeen, 2016, "Properties of Fired Clay Bricks Mixed with Waste Glass".

[8] Samir M. Shihada, 2017, "Properties of Fired Clay Bricks Mixed with Waste Glass"

[9] Shelby, J.E. (2005). Introduction to Glass Science and Technology. (2nd ed.). British Library: The Royal Society of Chemistry (RSC).

[10] Walter Lee Sheppard. (1986). Corrosion and Chemical Resistant Masonry Materials Handbook. (1st ed.). Pennsylvania: Noyes Publications.

[11] Zhang, L. (2013). Production of bricks from waste materials – a review. Construction Building Material, 47, 643–55.

Comparative Study on Serviceability Requirements of Flexural Members for Reinforced Concrete Building

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ABSTRACT*:* This paper presents the comparison of serviceability requirements analysis for four-storeyed reinforced concrete building under two trial design conditions, T-1 and T-2 with different rectangular beam cross section designs. The structural analysis include calculation of the crack widths and deflection in the reinforced concrete different flexural members under service loads. In this study, a computer aided software for the three-dimensional analysis of reinforced concrete frame for sections properties are carried out. Serviceability requirements such as maximum crack width and long- term deflection is carried out. The comparative analysis is performed in terms of storey moment, beam design sections, maximum crack width, allowable long-term deflection for two different conditions. The maximum crack width and long- term deflection are used to check against permissible limit.

KEYWORDS: *crack width, long-term deflection, different flexural members, serviceability, computer aided software*

1. INTRODUCTION

The serviceability of reinforced concrete structures has become a much more important design consideration with the use of high strength steel and concrete coupled with more accurate and efficient analytical procedures. When designing reinforced concrete structures, a designer must satisfy not only the strength requirements but also the serviceability requirements. Crack evaluation is required for reinforced concrete flexural member, since crack formation is unavoidable and it depends on many factors. For accurate determination of the member deflections, cracked members in the reinforced concrete structures need to be identified. The structural system of this building is the intermediate moment resisting frame for two conditions (T-1 and T-2) with rectangular beam cross section design. In this study, the comparative study of structures under different beam cross sections design will be prioritized. In this analysis, cracking was considered only for flexural members, beam elements because cracking in compression members, columns are usually not deemed a problem unless they are subjected to large bending moments. In summary, the objectives of this study are to study which design conditions is more suitable for this proposed building below serviceability requirements and to compare the structural analysis between two conditions, (T-1 and T-2) with different rectangular beam cross sections design conditions.

2. DATA PREPARATION OF THE PROPOSED BUILDING

2.1 Structural configuration

The following proposed building under seismic zones 2A with different beam cross sections is designed on the basic of the UBC-97 and ACI 318-99. The type of the proposed building is a four-storeyed reinforced concrete building. In the framing plan view, the building is a rectangular regular shape with the length of 46 ft and width of 34 ft. Total height of the building is 46 ft above ground level.

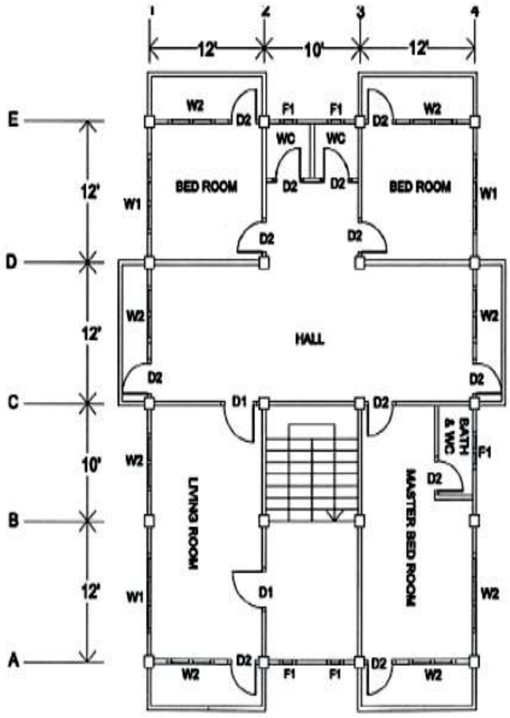


Fig 1. Typical floor plan of proposed building (ft)

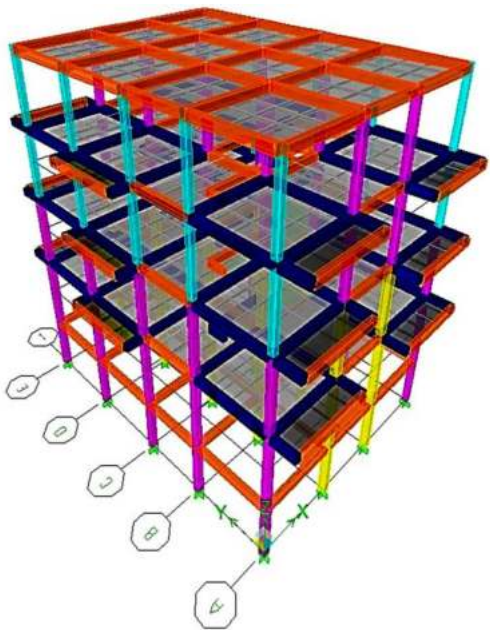


Fig 2. 3D model of proposed building (ft)

2.2 Material properties

The strength of a structure depends on the strength of the materials from which it is made.

Analysis property data

Weight per unit volume	= 150 pcf
Modulus of elasticity for concrete	= 3122 ksi
Poisson's ratio	= 0.2
Coefficient of thermal expansion	= 5.5×10^{-6}

Design property data

Bending reinforcement yield stress (f_y)	= 50 ksi
Concrete strength, cylinder, (f_c')	= 3 ksi

2.3 Loading consideration

According to UBC-97, the applied loads in this study are gravity loads that include dead and live loads, superimposed dead loads, and lateral loads that include wind and earthquake loads.

(1) Dead Loads

Unit weight of concrete	= 150 pcf
4 ½ thick brick wall weight	= 55 psf
Superimposed dead load	= 20 psf

(2) Live Loads

Live load on residential area	= 40 psf
Live load on stair	= 100 psf
Live load on verandah area	= 100 psf
Live load on roof	= 20 psf

(3) Wind load

Exposure type	= B
Basic wind velocity	=80 mph
Windward coefficient	= 0.8
Leeward coefficient	= 0.5
Wind important factor	= 1

(4) Earthquake load

Soil profile type	= S_D
Seismic zone	= 2 A
Frame system	= IMRF
Zone factor	= 0.15
Response modification factor, R	= 5.5
Importance factor, I	= 1
Building period coefficient, C_t	= 0.03

2.4 Loading combination

According to the UBC-97 and the ACI 318-99, the following 26 load combinations for the static analysis are as follow:

(1) 1.4 DL
(2) 1.4 DL +1.7 LL
(3) 1.05 DL+1.275 LL+1.275 WX
(4) 1.05 DL+1.275 LL-1.275 WX
(5) 1.05 DL+1.275 LL+1.275 WY
(6) 1.05 DL+1.275 LL-1.275 WY
(7) 0.9DL+1.3WX
(8) 0.9DL-1.3WX
(9) 0.9DL+1.3WY
(10) 0.9DL-1.3WY
(11) 1.05DL+1.28LL+EX
(12) 1.05DL+1.28LL-EX
(13) 1.05DL+1.28LL+EY
(14) 1.05DL+1.28LL-EY
(15) 0.9DL+1.02EX
(16) 0.9DL-1.02EX
(17) 0.9DL+1.02EY
(18) 0.9DL-1.02EY
(19) 1.16DL+1.28LL+EX
(20) 1.16DL+1.28LL-EX
(21) 1.16DL+1.28LL+EY
(22) 1.16DL+1.28LL-EY
(23) 0.79DL+1.02EX
(24) 0.79DL-1.02EX
(25) 0.79DL+1.02EY
(26) 0.79DL-1.02EY

3. COMPARISON OF ANALYSIS RESULTS AND DISCUSSIONS

The proposed building is analyzed with UBC-97 in seismic zones, 2A. Two trial designs are performed to obtain optimum design of proposed building. The design results of two trial designs conditions are expressed as T-1 and T-2. In this study, design sections for beams and columns of T-1 and T-2 are considered as rectangular type. The beam width are the same in both T-1 and T-2. The beam depth of T-2 is greater than that of T-1. Comparison of storey moment and design sections of proposed building under seismic zones 2A are the following.

3.1 Comparison of storey moment for T-1 and T-2

Comparison of storey moment for each condition is described in Fig 3. From these results, the difference between T-1 and T-2 is gradually increased from roof top floor to ground floor. The maximum storey moment are found in ground floor for T-1 and T-2. The maximum storey moment values of T-1 and T-2 are 4048.556 k-ft and 4174.35 k-ft at ground floor. This values of T-2 is 1.031 times higher than that of T-1. The storey shear in T-2 are larger than the storey shear of T-1, resulting in higher storey moments. However, the difference in roof top is very small and its values are 285.088 k-ft and 298.23 k-ft for T-1 and T-2. Moreover, the overturning moment for both conditions is satisfied because it is less than the allowable value.

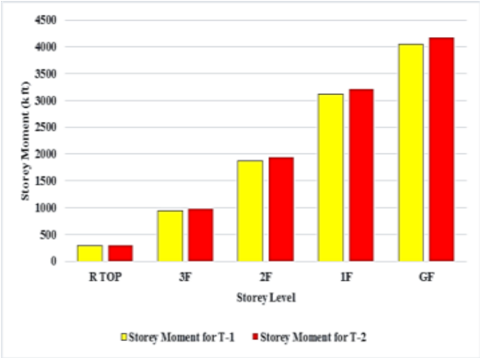


Fig 3. Comparison of storey moment for T-1 and T-2

3.2 Comparison of design sections of proposed building for static analysis

This model is analyzed by static case. Beam design sections of proposed building for two conditions, T-1 and T-2 are shown in Table 1, 2 and Fig. 4, 5, 6 and 7. From this Table 1, the beam depth for T-2 are larger than T-1 but the beam width are not significantly different. In this study, the column sizes for T-1 and T-2 are considered the same. This is because only the crack width and long-term deflection of the beam design sections will be considered.

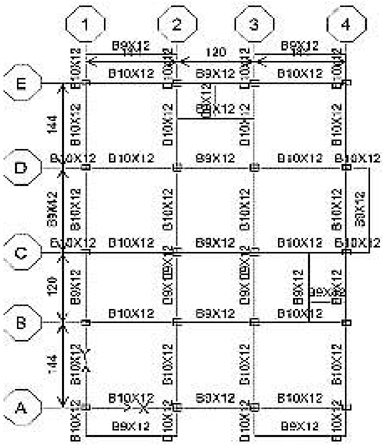


Fig 4. Typical beam section plan of proposed building for T-2 (in)

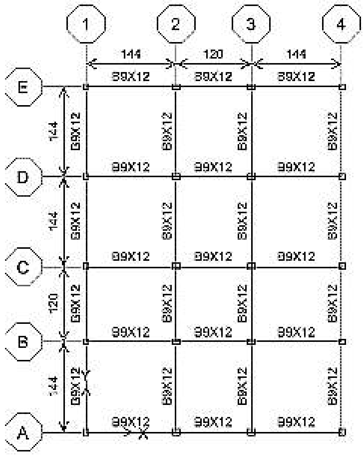


Fig 5. Roof beam section plan of proposed building for T-2 (in)

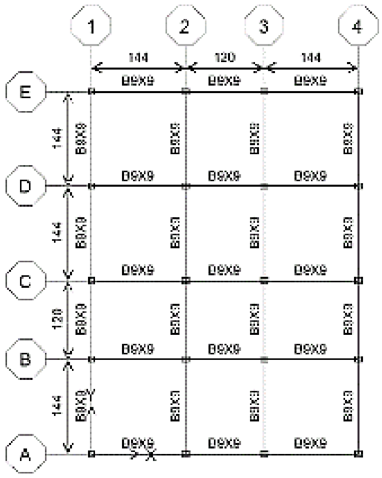


Fig 6. Typical beam section plan of proposed building for T-1 (in)

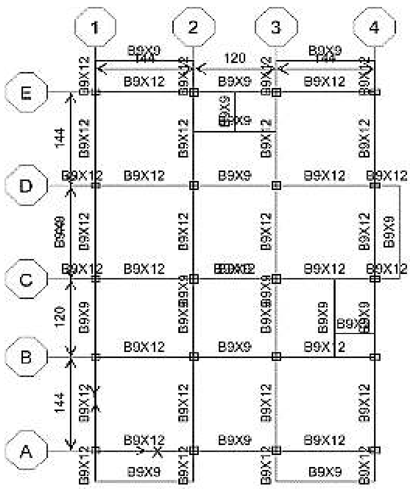


Fig 7. Roof beam section plan of proposed building for T-1 (in)

Table 1. Design sections for beam

Storey level	T-1	T-2
GF	9″×9″	9″×12″
Typical Floor(1F,2F,3F)	9″×9″, 9″×12″	9″×12″, 10″×12″
R Top	9″×9″	9″×12″

Table 2. Design sections for column

Storey level	T-1	T-2
GF to 2F	9″×12″, 12″×12″	9″×12″, 12″×12″
2F to R Top	9″×9″, 9″×12″	9″×9″, 9″×12″

4. COMPARISON OF REQUIREMENTS FOR SERVICEABILITY LIMIT STATE

The comparison of code requirements for serviceability limit state are shown by the following.

4.1 Crack width evaluation for flexural members

The following equation is Gergely-Lutz equation for predicting the maximum width of crack at the tension face of the beam in which deformed bars are used:

$$\omega = 0.076\beta f_s \sqrt[3]{d_c A} \quad (1)$$

ω = maximum crack width in thousandth inch

d_c = thickness of concrete cover measured from tension face to center of bars closet to that face (in)

$$f_s = \text{steel stress} = 0.6 f_y \text{ (ksi)} \quad (2)$$

β = ratio of distance from tension face and from steel centroid to natural axis equal to h_2/h_1

A =concrete area = 2yb/m (in^2) (3)

$$z = f_s \sqrt[3]{d_c A} \quad (4)$$

In most beam, $\beta = 1.2$

For beam,

Interior exposure $z \leqslant 175 \longrightarrow \omega_{max} \leqslant 0.016''$ (5)

Exterior exposure $z \leqslant 145 \longrightarrow \omega_{max} \leqslant 0.013''$ (6)

Table 3. Calculation of maximum crack width

Item	B 9″×9″	B 9″×12″	B 10″×12″
β	1.2	1.2	1.2
f_s	30	30	30
d_c	2.5	2.5	2.5
y	2.5	2.5	2.5
b	9	9	10
m	3	3	3
A	11.25	15	16.67
$\sqrt[3]{d_c A}$	3.041	3.347	3.467
z	91.23	100.4	104.01
ω_{max}(in)	0.00083	0.00091	0.00095

From this result Table 3, the z values of the different beam design sections is less than 145, so the maximum allowable crack width should be checked at 0.013 in. The maximum crack width for B 9″×9″, B 9″×12″ and B 10″×12″ of T-1 and T-2 have exist within the crack width limit.

4.2 Deflection calculation methods by design codes

Deflection of structures affects their aesthetic appearance as well as their long-term serviceability.
The effective moment of inertia I_e,

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \quad (\text{for } M_a \geq M_{cr}) \quad (7)$$

I_g = moment of inertia of gross section

I_{cr} = moment of inertia of the cracked transformed section

M_a= bending moment

M_{cr}=moment corresponding to the flexural cracking

The pre-cracking region stops at the initiating of the first flexural crack when the concrete stress reaches its modulus of rupture strength f_r.

$$f_r = 7.5\,\lambda \sqrt{f_c'} \quad (8)$$

If the distance of the extreme tension fiber from the center of gravity of the section is y_t and the cracking moment is M_{cr}.

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (9)$$

f_r = the flexural tensile strength of concrete

λ = 1.0 for normal weight concrete

For a rectangular section,

$$y_t = \frac{h}{2} \quad (10)$$

h = the total thickness of the beam.

The total long term deflection is

$$\Delta_{LT} = \Delta_L + \lambda_\alpha \Delta_D + \lambda_t \Delta_{Ls} \quad (11)$$

In this study, sustained live load is not considered.

$$\Delta_{LT} = \Delta_L + \lambda_\alpha \Delta_D \quad (12)$$

Δ_L = initial live-load deflection

Δ_D = initial dead-load deflection

$$\lambda = \xi/1+50\rho \quad (13)$$

$\xi = 2$ (5 year loading), $\rho'=0$, $\lambda = 2$

Check total long-term deflection,

$$\Delta_{LT} < \Delta_{Limit} \quad (14)$$

Maximum permissible deflections, $\Delta_{Limit} = L/480$ (15)

Table 4. Cracking moment values for corresponding beam sections of T-1 and T-2

	Mid span			Support		
M_{cr} (k-in)	9×9	9×12	10×12	9×9	9×12	10×12
	155	234	256	346	665	739

In this study, maximum moment M_a values get from ETABS software analysis are greater than cracking moment M_{cr} values. So, flexural cracks develop in a beam. According this results, the average value difference cracking moment values for corresponding beam sections of T-1 and T-2 at mid span and support are 2.6 times. The cracking moment values at support are larger than that at mid span under T-1 and T-2. So, the larger the beam design sections, the greater the cracking moment.

4.3 Comparison of maximum effective moment of inertia for T-1 and T-2 under total service load (DL+LL)

In Table 5, 6 and Fig 8, 9, the values of maximum effective moment of inertia of each condition are higher in mid-span. In Fig 8, the value of maximum effective moment of inertia for mid-span of T-2 condition is higher than that of T-1 condition at all storey level. The maximum value of effective moment of inertia for mid-span (91786.8 in4) is found at 1F level under T-2 condition. This value for T-2 condition gradually increase and decrease in T-1 condition.

Table 5. Maximum effective moment of inertia at mid-span for T-1 and T-2 conditions

Level	Line	Ie(in^4)	
		T-1 B 9″×9″	T-2 B 9″×12″
R Top	D(2-3)	77694.2	85907.4
3F	D(2-3)	60317.86	77230.21
2F	D(2-3)	42580.65	87364.76
1F	D(2-3)	46167.32	91786.8

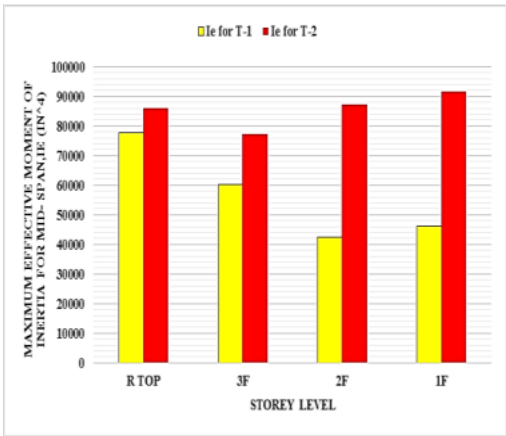
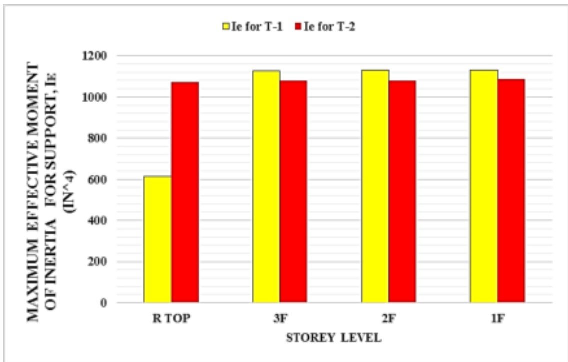


Fig 8. Comparison of effective moment of inertia at mid- span for T-1 and T-2 condition under total service loads



In Fig 9, this result values for support are only 1.74 times different at R Top level but slightly different at other storey levels for two conditions (T-1 and T-2).

Table 6. Maximum effective moment of inertia at support for T-1 and T-2 conditions

Level	Line	Ie(in^4)			
		T-1		T-2	
		B 9″×9″	B 9″×12″	B 10″×12″	B 9″×12″
R Top	B(1-2)	614.88	-		1073.75
3F	B(1-2)	-	1128.41	1081.58	
2F	B(1-2)	-	1129.14	1080.79	
1F	B(1-2)	-	1132.09	1086.45	

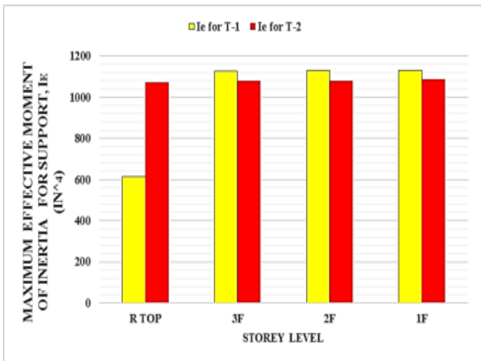


Fig 9. Comparison of effective moment of inertia at support for T-1 and T-2 conditions under total service loads

4.4 Comparison of total long-term deflection under total service load (DL+LL) for T-1 and T-2

In Table 7 and Fig 10, this results values for both conditions are very different at all storey levels. Although B 9″×9″ is used at R Top level of T-1 condition, B 9″×12″ is used at R Top level of T-2 condition.

Table 7. Total maximum long-term deflection for (T-1 and T-2)

Level	Line	Δ_{LT}(in)			
		T-1		T-2	
		B 9″×9″	B 9″×12″	B 10″×12″	B 9″×12″
R Top	D(1-2)	0.005	-		0.0016
3F	D(1-2)	-	0.0089	0.007	
2F	D(1-2)	-	0.0087	0.007	
1F	C(1-2)	-	0.0062	0.004	

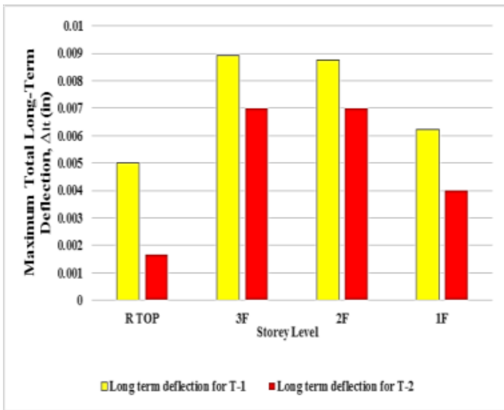


Fig 10. Comparison of maximum long-term deflection for T-1 and T-2 condition under total service loads

So, the lowest value of total long-term deflection for T-2 condition is 0.0016 in and 0.005 in for T-1 condition at R Top level and the highest value of total long-term deflection for T-1 condition is 0.0089 in and 0.007 in for T-2 condition at 3F level in grid line D (1-2). Serviceability limit state depends on this value of the total maximum long-term deflection under service loads. So, the beam design sections of T-2 condition with low total maximum long-term deflection should be selected at all storey level.

Table 8. Check for total maximum long-term deflection (Δ_{LT}) under design condition type-1, T-1

Level	Beam (in)	Span (ft)	Δ_{LT} (in)	Δ_{Limit} (in) L/480	Δ_{Limit} > Δ_{LT}
R top	9×9	12	0.005	0.3	ok
3F	9×12	12	0.0089	0.3	ok
2F	9×12	12	0.0087	0.3	ok
1F	9×12	12	0.0062	0.3	ok

Table 9. Check for total maximum long term deflection (Δ_{LT}) under design condition type-2, T-2

Level	Beam (in)	Span (ft)	Δ_{LT} (in)	Δ_{Limit} (in) L/480	Δ_{Limit} > Δ_{LT}
R top	9×12	12	0.0016	0.3	ok
3F	10×12	12	0.007	0.3	ok
2F	10×12	12	0.007	0.3	ok
1F	10×12	12	0.004	0.3	ok

According to Table 8 and 9, the maximum total long-term deflection for both conditions (Δ_{LT}) is less than the maximum permissible deflection (Δ_{Limit}). It is found that the result value for T-2 is less than T-1 condition. Therefore, the beam design sections under T-2 condition is used in this proposed building due to low total maximum long-term deflection.

5. CONCLUSIONS

In this study, the proposed building is a four-storeyed reinforced concrete building and is located in seismic zone 2A. The design code used in this paper are the UBC-97 and the ACI 318-99. Crack occurrence is determined by $M_a > M_{cr}$. Serviceability requirement i.e crack width is checked by allowable maximum crack width and z values. Although all of the flexural members for both conditions are safe according to serviceability limit, in comparing maximum total long-term deflection, beam design sections of trial design type-1 condition, T-1 for all storey level must overwrite. This mean that the smaller the beam design sections, the higher the values of maximum total long-term deflection. So, the larger beam design sections of trial design type-2 condition, T-2 with low maximum total long-term deflection are suitable for this proposed building. All the checks for maximum crack width and long-term deflection of T-2 condition carried out for the serviceability requirements are within the limitation. So, the trial design condition type-2, T-2 is used in optimum design for the proposed building.

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REFERENCES

[1] International Conference of Building Officials. 1997. "Structural Engineering Design Provisions" Uniform Building Code. Volume 2. U.S.A.: International Conference of Building Officials.

[2] Nilson, A.H. 1997. "Design of Concrete Structure". 12th ed. Singapore: Mc. Graw- Hill Co.Inc.

[3] American Concrete Institute Committee. 1999. "Building Code Requirements for Structural Concrete" (ACI 318-1999) and Commentary (ACI 318-1999) U.S.A.: American Concrete Institute Committee.

[4] U Nyi Hla Nge, "Refresher Course and Field Reference Manual for Site Engineers & Inspectors" and "Design of Concrete Structure".

DESIGN OF PRE-CAST PILE FOUNDATION FOR TWELVE STOREYED REINFORCED CONCRETE BUILDING

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ABSTRACT*:* This paper mentions about the design of pile foundation for twelve storeyed reinforced concrete building with the length of 94 ft and width of 48 ft. Overall height of superstructure is 144 ft. The superstructure load is taken from analysis and design result of the proposed building and the required soil parameters are taken from soil report. The critical unfactored column loads are classified into five groups. Soil type is cohesionless (sand) and minimum bearing capacity of soil is 0.9 ton/ft^2. Then pile foundation is used. The pre-cast pile size of 10 in ×10 in with 60 ft length is considered to be used. Then, number of piles and their arrangements are determined based on amount of column loads. The actual settlement of each group pile of 0.506 in, 0.538 in, 0.694 in, 0.63 in, 0.57 in are less than the allowable settlement of 1 in. Pile caps of 5.5 ft ×5.5 ft ×2.5 ft, 8.5 ft ×5.5 ft ×2.83 ft, 8.5 ft ×7.75 ft ×2.83 ft and 24 ft ×7.5 ft ×4.5 ft are used for each group. After that, piles cap are checked for punching shear, beam shear and bearing. The reinforcements of pile caps are also calculated.

KEYWORDS: *Reinforced concrete building, load, soil capacity, pile foundation, settlement.*

1. INTRODUCTION

A foundation is the most important part of the engineering construction system. Foundation can be classified into two major categories; that is shallow foundations and deep foundations. Deep foundations are used when the required bearing capacity of shallow foundation are not obtained.

Pile foundation is one of the effective types of deep foundation which carries the load to the soil without any differential settlement in the soil. Piles are long and slender members which transfers the load to deeper soil or rock of high bearing capacity. Depending on the type of load to be carried, the subsoil conditions and the location of the water table, different types of piles are used in construction work. Piles can be divided into: (a) steel piles, (b) concrete piles, (c) wooden (timber) piles and (d) composite piles. Pre-cast concrete piles must be designed and manufactured to withstand handling and driving stresses in addition to service loads. So, they have high load capacity, resist corrosion resistance and hard driving is possible.

The proposed building is located on 67/68 street, between Thazin and Khattar Street, Chan Mya Tharsi Township, Mandalay. To obtain the most economical and durable foundation, the superstructure loads and the soil condition were considered. Pre-cast concrete piles were used according to the soil capacity. In designing pre-cast pile foundation, allowable bearing capacity and settlement of piles were calculated and then pile caps were designed in details.

2. DATA OF PROPOSED BUILDING

The proposed building is twelve storeyed reinforced concrete rectangular shape building.

Length of building	= 94 ft
Width of building	= 48 ft
Overall height of building	= 144 ft

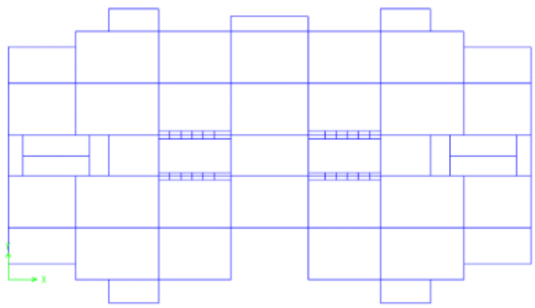

Fig 1. Plan of proposed building

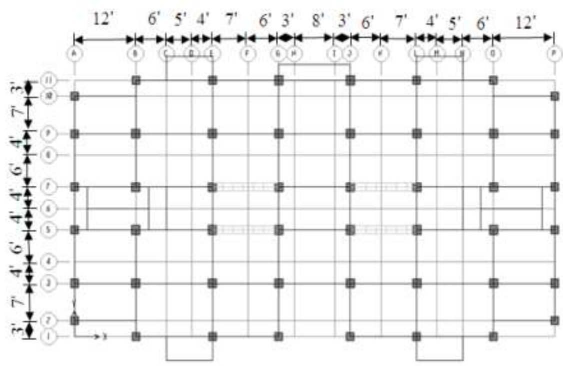

Fig 2. Beam and column layout plan of proposed building

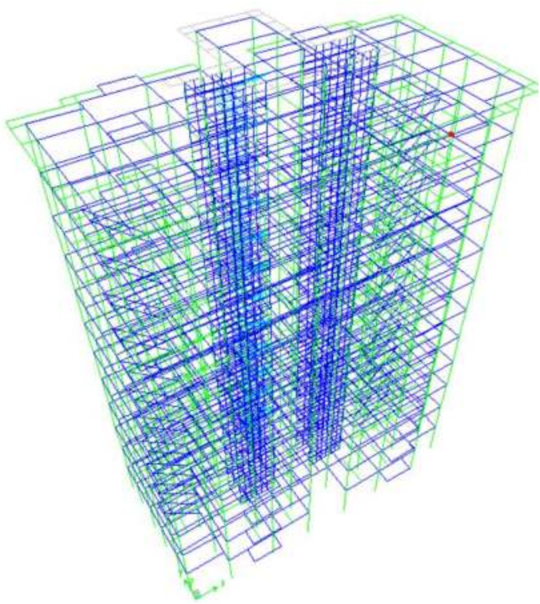

Fig 3. 3D model of proposed building

2.1 Material Properties of Proposed Building

Material properties used for proposed building are as follows:

Reinforced Concrete
Strength of Concrete (fc') = 3.5 ksi
Yield Strength of Main Reinforcement (f_y) = 50 ksi
Yield Strength of Shear Reinforcement(f_y) = 50 ksi
Young Modulus of Concrete (E_c) = 3372 ksi
Poisson's Ratio = 0.175

2.2 Load Consideration for Proposed Building

In designing of the proposed building, the applied loads are dead loads, live loads, wind loads and earthquake loads. Self-weights of all elements, floor and finishing (4.5 inches thickness brick walls) are considered as dead loads. Considerations of earthquake and wind loads are based on UBC-97. For designing structural elements, ACI (318-99) is used.

(i) Gravity Loads
Live load on floor = 40 psf
Live load on stair & landing = 100 psf
Live load on roof = 40 psf
Live load on stair roof = 20 psf
Floor finishing = 15 psf
Unit weight of concrete = 150 pcf
4.5" thick brick wall = 50 psf

(ii) Wind Loads
Wind speed = 100 mph

(iii) Earthquake Load
Earthquake zone = 4
Zone factor (Z) = 0.4
Response modification factor = 8

2.3 Required Soil Parameters from Soil Report

According to the laboratory test, the ground water table was found at 37 ft below the ground level. Table 1 shows the required soil parameters from the soil report.

Table 1. Required Soil Parameters from Soil Report

Depth (ft)	Soil Description	Moisture Content (%)	γ (lb/ft³)	c (lb/ft²)	Φ	N
0-22	Clayey Silt	11.28	119.9	878.83	10°	22
22-28	Silty Sand	20.92	123.6	725.17	11°	18
28-40	Silty Sand	24.96	118.5	694.35	12°	27
40-50	Sandy Silt	23.54	121.7	615.08	12°	29
50-60	Sandy Silt	17.45	124.8	676.82	19°	41

3. ALLOWABLE BEARING CAPACITY OF SOIL

The allowable bearing capacity q_a is the ultimate bearing capacity q_u divided by an appropriate factor of safety, FS. The soil must be capable of carrying the load from any engineering structure placed upon it without a shear failure and with the resulting settlements being tolerable for that structure. The allowable bearing capacity is calculated by three methods with safety factor 3, namely

(1) Meyerhof's Method,

(2) Terzaghi's Method and

(3) Hansen's Method.

Meyerhof's Method

$$q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5\gamma BN_r s_r d_r \quad (1)$$

Terzaghi's Method

$$q_{ult} = cN_c s_c + \bar{q} N_q s_q + 0.5\gamma BN_r s_r \quad (2)$$

Hansen's Method

$$q_{ult} = c N_c s_c d_c + \bar{q} N_q s_q d_q + 0.5 \gamma B N_r s_r d_r \quad (3)$$

Table 2 describes the results of allowable bearing capacities by using three methods. The minimum value of 0.9ton/ft^2 is finally selected as the allowable bearing capacity of soil.

Table 2. Results of Allowable Bearing Capacity of Soil

Methods	q_{all} (ton/ft^2)
Meyerhof	1.7
Terzaghi	1.2
Hansen	0.9

4. DETERMINATION OF FOUNDATION TYPE

The minimum allowable bearing capacity of soil = 0.9 ton/ft^2 = 2016 psf = 2.016 ksf

Total unfactored load of superstructure = 20361.31 kips

Foundation area $= 94 \times 48 = 4512\ \text{ft}^2$

Required bearing capacity $= 4.5\ \text{k/ft}^2 > 2.016\ \text{k/ft}^2$

Since required bearing pressure is more than the available bearing capacity, pile foundation is used.

5. DETERMINATION OF PILE GROUP NAMES FOR PRE-CAST PILE FOUNDATION DESIGN

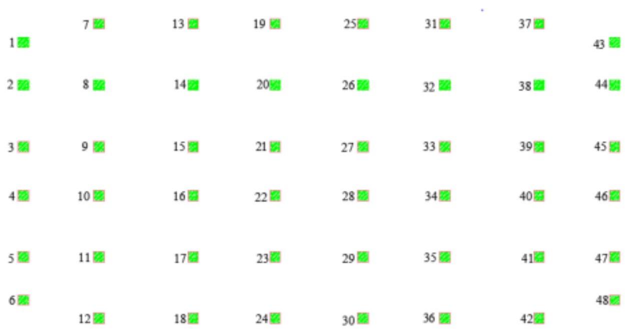

Fig 4. Location plan of node No.

Pile group names for pre-cast pile foundation are described in Table 3.

Table 3.Pile Group Names for Pre-Cast Pile Foundation

Pile Group	Node No.	No. of Col;	Col; Size	Col; Loading Range (kips)	Max; load (kips)
1	1,2, 5,6, 43,44,47,48	8	18"×18"	180-260	253.48
2	3,4,7,12,14,17,19,20, 23,24,25,26,29,30,31, 32,35,37,42,45,46	21	18"×18"	260-420	415.71
3	8,9,10,11,13,18,36, 38,39,40,41	11	20"×20"	420-560	554.45
4	(15,21), (27,33)	4	20"×20"	1564.04, 1586.47	1586.5
5	(16,22), (28,34)	4	20"×20"	1347.22, 1352.75	1352.8

6. DESIGN OF PRE-CAST PILE FOUNDATION UNDER VERTICAL AXIAL COMPRESSION LOADS

In this design, 60 ft long 10″×10″ pre-cast concrete piles are considered. Among the pile length of 60 ft, the sand layer's depth is 38 ft. So, skin friction is calculated by assuming as piles in sand.

Pile size, $A_p = 10 \times 10 = 100\ \text{in}^2 = 0.694\ \text{ft}^2$

Assume, L = depth of pile = 60 ft

$N = 41$ (for 60 ft depth)

Pile base, $D = 0.83$ in

Pile perimeter, $p = 2(D_1 + D_2) = 2(10 + 10) = 3.33\text{ft}$

$$\frac{L}{D} = \frac{60}{0.83} = 72 > 10$$

$$q_p = 800\ N\ \frac{L}{D} \le 8000\ N \qquad (4)$$

$$\therefore\ q_p = 8000 \times 41 = 328000\ \text{lb/ft}^2$$

$$Q_p = A_p q_p = 0.694 \times 328000 = 227\ \text{kips}$$

$$L = 15\ D = 15 \times 0.83 = 12.5\ \text{ft}$$

$$\sigma_v' = 12.5 \times 119.9 = 1498.75\ \text{lb/ft}^2$$

$$pLf_{av\,(0\text{ to }12.5)} = 3.33 \times 12.5 \times \left[\frac{K\sigma'_v \tan\delta}{2}\right] = 5741\ \text{lb}$$

$$pLf_{av\,(12.5\text{ to }60)} = 3.33 \times 12.5 \times [1.1 \times 1498.75 \times \tan(0.5 \times 19)] = 43638\ \text{lb}$$

$$Q_f = 5741 + 43638 = 49379\ \text{lb} = 49\ \text{kips}$$

$$Q_{ult} = Q_p + Q_f = 227 + 49 = 276\ \text{kips}$$

$$Q_{all} = Q_{ult}/F_s = 276/3.5 = 78\ \text{kips}$$

Therefore, allowable bearing capacity of 10"×10" precast concrete pile is 78 kips.

6.1 Design of Pile Group 1

Pile group 1 is designed for the critical unfactored column load of 253.48 kips and so 254 kips is considered.

(i) Number of piles and their arrangement

Total load applied on piles = 254 kips

Minimum numbers of piles to be used = 254/78=3.25

∴ Use 4 numbers of piles.

Try a group of 4 piles arranged in a square pattern placed the piles at 2.5 ft center to center spacing with 5'-6" × 5'-6" pile cap.

Assume pile cap thickness = 2.5 ft

Pile cap width, b = 5.5 ft

$$\bar{b} = 2.5 + 10/12 = 3.33\ \text{ft} = 40\ \text{in}$$

Pile cap weight $= 5.5 \times 5.5 \times 2.5 \times 0.15 = 11.34$ kips

Total weight on pile group $= 254 + 11.34 = 265.34$ kips

Load per pile $= \frac{265.34}{4} = 66$ kips < allowable capacity of pile = 78 kips

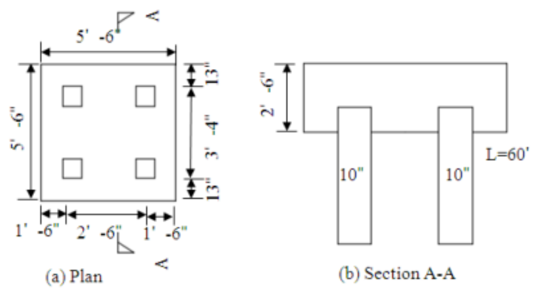

Fig 5. Pile group arrangement for Group 1

(ii) Allowable bearing capacity of pile

$n_1 = 2$, $n_2 = 2$, $D = 10$ in, $d = 30$ in, $p = 4D = 4 \times 10 = 40$ in

$$E_g = 1$$

Pile group capacity,

$(Q_G)_{ult} = n \times E_g \times (Q_v)_{ult} = 4 \times 1 \times 276 = 1104$ kips

$(Q_G)_{all} = \frac{(Q_G)_{ult}}{F.S} = \frac{1104}{3.5} = 315.42$ kips

Therefore, pile group capacity > total weight of pile group

The group capacity is 315.42 kips, which is greater than the load 265.34 kips on the pile group. Therefore, it is acceptable from a bearing capacity point of view.

(iii) Settlement of piles

Two methods of semi-empirical method and empirical method are used. To calculate the settlement,

(a) Semi-empirical method

$$S_t = S_s + S_p + S_{ps} \quad (5)$$

$$S_s = \frac{(Q_{pa} + \alpha_s Q_{fa})\, L}{A_p\, E_p} \quad (6)$$

$$S_p = \frac{C_p Q_{pa}}{B\, q_p} \quad (7)$$

$$S_{ps} = \frac{C_s Q_{fa}}{L\, q_p} \quad (8)$$

Total load on pile group = 265.34 kips

Load per pile $= \frac{265.34}{4} = 66$ kips

$S_s = \frac{(54.88 + 0.5 \times 11.84) \times 60 \times 12 \times 1000}{0.69 \times 144 \times 3.12 \times 1000000}$

$= 0.14$ in

$S_p = \frac{0.02 \times 54.88 \times 144}{10 \times 328.9} = 0.048$ in

$S_{ps} = \frac{0.046 \times 11.84 \times 12}{60 \times 328.9} = 0.0003$ in

$S_t = 0.14 + 0.048 + 0.0003 = 0.188$ in

(b) Empirical method

$S_t = \frac{B}{100} + \frac{Q_{ua} L}{A_p E_p} = 0.253$ in

The results obtained from these methods are compared and then higher value (0.253 in) is chosen.

Therefore, the settlement of pile group is,

$S_G = S_t \sqrt{\frac{b}{D}} = 0.253 \times \sqrt{\frac{40}{10}} = 0.506 \text{ in} < 1 \text{ in}$

(Satisfied)

The calculated settlement of pile group is 0.506 in. This is less than the allowable settlement of 1 in. Therefore, the designed pile diameter, length and group settlement is acceptable.

(iv) Design of pile cap 1

Column size = 18" × 18"

Unfactored column load = 254 kips

Factored column load = 366.61 kips ≅ 367 kips

Allowable bearing capacity of piles, $R_a = 315.42$ kips

$f_c' = 3500$ psi, $f_y = 50000$ psi

Assume effective depth, d = 18 in

Effective allowable capacity of pile group,

R_e = allowable bearing capacity of pile group – pile cap weight = 315.42 – 11.34 = 304.08 kip

Average load factor $= \frac{\text{factored column load}}{\text{unfactored column load}} = \frac{367}{254} = 1.44$

Pile reaction for strength design, $R_u = 304.08 \times 1.44$

= 437.88 kips

Choose 5.5′× 5.5′ square pattern concrete cap to cover the pile group arrangement.

Pile reaction for strength design, $q_u = \frac{R_u}{\text{selected area}} = \frac{437.88}{5.5 \times 5.5} = 14.48$ kips / ft^2

(a) Punching shear check

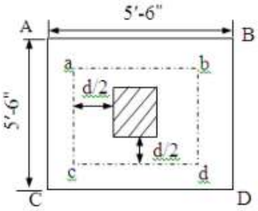

Fig 6. Critical depth of punching shear for Group 1

Punching shear, $V_{u1} = q_u\, [(L_{AB} \times L_{AC}) - (L_{ab} \times L_{ac})]$

$= 14.48 \times [(5.5 \times 5.5) - \frac{36 \times 36}{12 \times 12}] = 307.7$ kips

Nominal shear strength, $\phi\, V_c = \phi\, 4 \sqrt{f_c'}\, b_o\, d$

$= 0.85 \times 4 \times \sqrt{3500} \times 144 \times 18 \times 10^{-3}$

$= 521.37 \text{ kips} > V_{u1} = 307.7$ kips

Therefore, the depth'd' is adequate for punching shear.

(b) Check one way shear or beam shear

For X-direction,

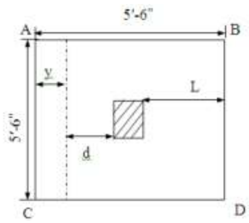

Fig 7. Critical section of one way shear for Group 1 in X-direction

One way shear, V_{u2} = bearing capacity × one way shear area = $q_u \times L_{AB} \times y = 14.48 \times 5.5 \times 0.5 = 39.82$ kips

Nominal one way strength, $\phi\, V_c = \phi\, 2 \sqrt{f_c'}\, b\, d$

$= 0.85 \times 2 \times \sqrt{3500} \times 5.5 \times 12 \times 18 \times 10^{-3}$

$= 119.48 \text{ kips} > V_{u2} = 39.82$ kips (OK)

Therefore, depth 'd' is adequate for beam shear check.

For Y-direction,

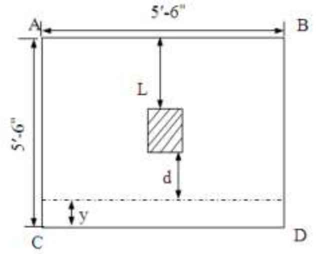

Fig 8. Critical section of one way shear for Group 1 in Y-direction

One way shear, V_{u2} = bearing capacity × one way shear area = $q_u \times L_{AC} \times y = 14.48 \times 5.5 \times 0.5 = 39.82$ kips

Norminal one way strength, $\phi V_c = \phi 2 \sqrt{f_c'}\ b\ d$

$= 0.85 \times 2 \times \sqrt{3500} \times 5.5 \times 12 \times 18 \times 10^{-3}$

$= 119.48$ kips $> V_{u2} = 39.82$ kips (OK)

Therefore, depth 'd' is adequate for beam shear check.

So, total depth of pile cap, h = 18+ 9+ 3 = 30 in = 2'-6"

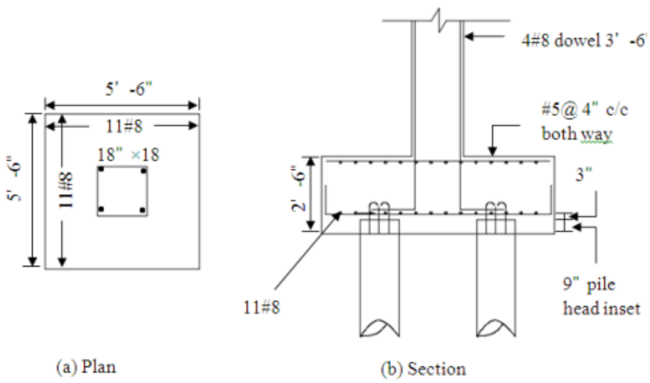

Fig 9. Pile cap reinforcement for Group 1

6.2 Design of Pile Group 2

Pile group 2 is designed for the critical unfactored column load of 415.71 kips and so 416 kips is considered. 8.5ft x 5.5 ft x 2.83 ft pile cap is used. 6 numbers of piles are used in group 2.

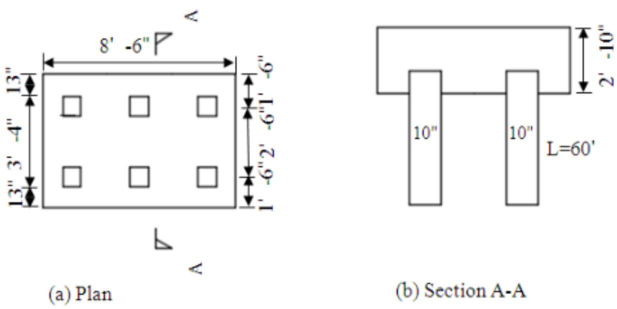

Fig 10. Pile group arrangement for Group 2

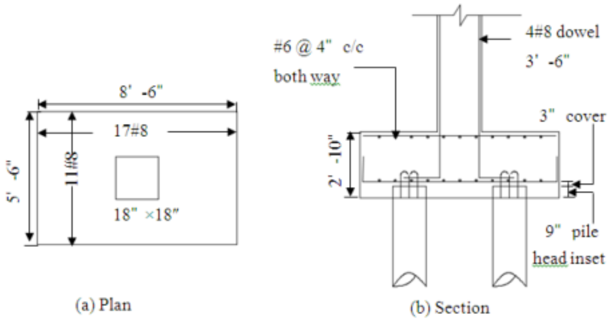

Fig 11. Pile cap reinforcement for Group 2

6.3 Design of Pile Group 3

Pile group 3 is designed for the critical unfactored column load of 554.45 kips and so 555 kips is considered. 8.5ft x 7.75 ft x 2.83 ft pile cap is used. 8 numbers of piles are used in group 3.

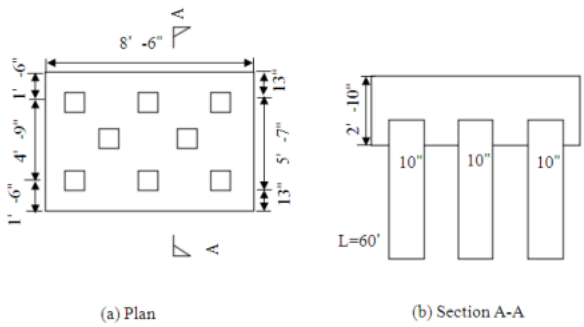

Fig 12. Pile group arrangement for Group 3

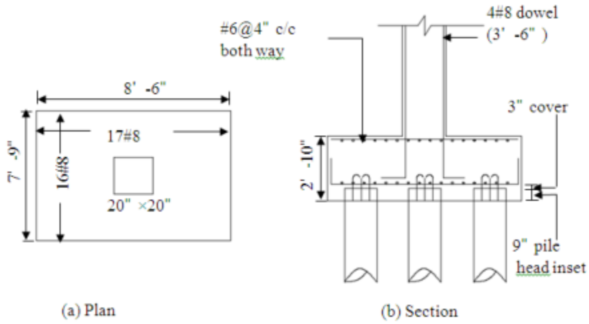

Fig 13. Pile cap reinforcement for Group 3

6.4 Design of Pile Group 4

Pile group 4 is designed for the critical unfactored column load is 1587 kip. 24 ft x 7.5 ft x 4.5 ft pile cap is used. 27 numbers of piles are used in group 4.

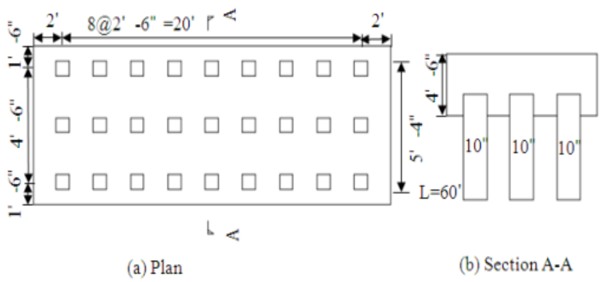

Fig 14. Pile group arrangement for Group 4

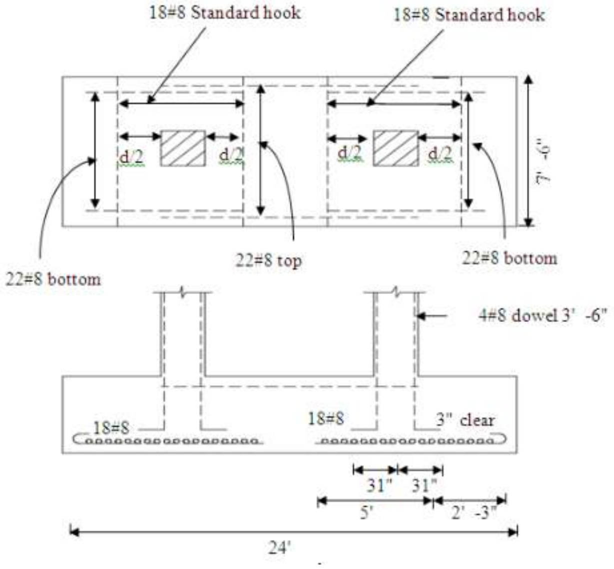

Fig 15. Pile cap reinforcement for Group 4

6.5 Design of Pile Group 5

Pile group 5 is designed for the critical unfactored column load is 1353 kip. 24 ft x 7.5 ft x 4.5 ft pile cap is used. 27 numbers of piles are used in group 5.

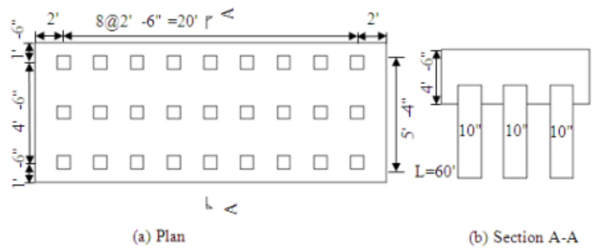

Fig 16. Pile group arrangement for Group 5

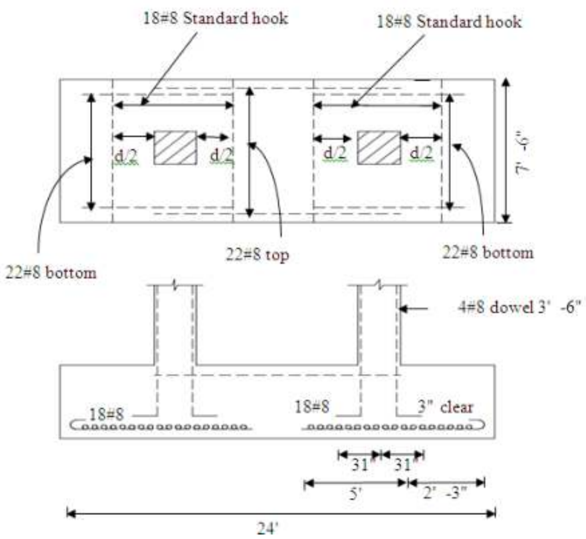

Fig 17. Pile cap reinforcement for Group 5

6.6 Summary of Design Results of Pile Groups

Table 4 and 5 show the resulting values of precast pile foundation and design results of pile caps respectively.

Table 4. Resulting Values of Pre-cast Pile Foundation

Group	1	2	3	4	5
Number of pile	4	6	8	27	27
Allowable Bearing Capacity (kips)	315.42	474.14	630.85	2129.4	2129.2
Applied Load (kips)	265.34	435.85	579.7	1695	1474.5
Settlement of pile (in) (Semi-empirical method)	0.188	0.21	0.213	0.18	0.156
Settlement of pile (in) (Empirical method)	0.253	0.269	0.268	0.25	0.227
Settlement of pile group (in)	0.506	0.538	0.694	0.63	0.57

Table 5. Design Results of Pile Caps

Pile cap no.		1	2	3	4	5
length (ft)		5.5	8.5	8.5	20	20
Width (ft)		5.5	5.5	7.75	7.5	7.5
Depth (ft)		2.5	2.83	2.83	4.5	4.5
Pile head insert		9	9	9	12	12
Cover (in)		3	3	3	3	3
No. of piles		2	6	8	27	27
Top reinforcement		No.5 (No.16) @ 4″ c/c	No.6 (No.19) @ 4″ c/c	No.6 (No.19) @ 4″ c/c	No.8 (No.25) @ 5″ c/c	No.8 (No.25) @ 5″ c/c
Bottom reinforcement	X-Direction	11No.8 (No.25)	17No.8 (No.25)	17No.8 (No.25)	22No.8 (No.25)	22No.8 (No.25)
	Y-Direction	11No.8 (No.25)	11No.8 (No.25)	16No.8 (No.25)	18No.8 (No.25)	18No.8 (No.25)
Dowel bar and length		4 No.8 (No.25) ×3′-6″	4 No.8 (No.25) ×3′-6″	4 No.8 (No.25) ×3′-6″	4 No.8 (No.25) ×3′-6″	4 No.8 (No.25) ×3′-6″

7. CONCLUSIONS

10 in x 10 in precast pile, 60 ft are used. Four numbers of piles for group 1, six numbers of piles for pile group 2, eight numbers of piles for group 3, twenty seven numbers of piles for group 4 and 5. The actual settlement of 0.506 in, 0.537 in, 0.694 in, 0.63 in and 0.57 in are less than the allowable settlement of 1 in. Pile cap thicknesses are satisfied for punching shear, one shear or beam shear and bearing.

ACKNOWLEDGEMENT

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REFERENCES

[1]Braja M. Das. "*Principles of Foundation Engineering*", 4th Edition, 1999.

[2]Arthur H. Nilson, "*Design of Concrete Structures*", 14th Edition, Mc Graw Hill Co, 1997.

[3]Prakash, Shamsher and Hari D. Shama. "*Pile Foundation in Engineering Practice*", 1990.

Design of Water Supply System for Technological University (Lashio)

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ABSTRACT*:* This paper presents Water supply system from water sources near Aye Chan Thar Dam to TU (Lashio). The main purpose of this paper is to have sufficient capacity of the water needs of the consumers under all demand conditions and to determine the materials of transmission pipe for water supply system. To design water supply system, preliminary surveys on topography of study area, ground elevation and length are carried out. The ground elevation is measured with a total station, transit software and Autodesk Civil 3D 2020 software. From the study area, the total length of transmission pipe is 3275.1m with diameter of 4 inches(101.6 mm), 554.267 m with diameter 6 inches(152.4 mm) and 319.634m with diameter of 8 inches(203.2 mm). This design is based on Hazen William's formula knowing hydraulic gradient and total losses. Also, Microsoft Excel software is employed as a spread sheet to obtain the hydraulic gradient and total losses. The materials of transmission pipes are considered as Cast Iron (CI), Polyvinyl Chloride Pipes (PVC) and High Density Polyethylene pipes (HDPE). The calculated hydraulic gradient is less than the actual hydraulic gradient. In this study, comparison results of calculated hydraulic gradient and actual hydraulic gradient and that of total losses by using coefficient of roughness for three different pipe materials are considered.

KEYWORDS: *Total Station, Transit Software, Autodesk Civil 3D Software, Hazen Willam's Formula, Coefficient of Roughness, Different Pipe Materials*

1. INTRODUCTION

Water earth is made up of land, water and air. All These constituents can provide our everyday live. As the Myanmar ancient saying "Man can live one week without food, but without water can live one day, water is very important substance.

A water supply or water distribution system is developed continuous supply system and intermitted system. In this study, the continuous water supply system is considered. In continuous water supply system, the water is available for 24 hours a whole day. The system of distribution are gravity system, direct pumping system and combined system. In the gravity system, water flows from the sources to the distribution area (TU, Lashio). This system is reliable and economic. According to the topography in this study area, the gravity system is used.

2. METHODOLOGY

2.1 Location of Study Area

The study area, water sources near Aye Chan Thar Dam is located near Hopake Village, Lashio Township, Shan State. TU (Lashio) and water sources are situated 23 ˚1´58 ˝ Latitude, 97˚46´21 ˝ Longitude and 23˚2´56 ˝ Latitude, 97˚45´32˝ longitude. Their elevation are 2407.6 ft and 2492.6 ft above sea level. The total length of water supply system is 4149.001m. In this study, the water supply system is designed by 3000 people in TU (Lashio). The location of study area is shown in Fig 1.

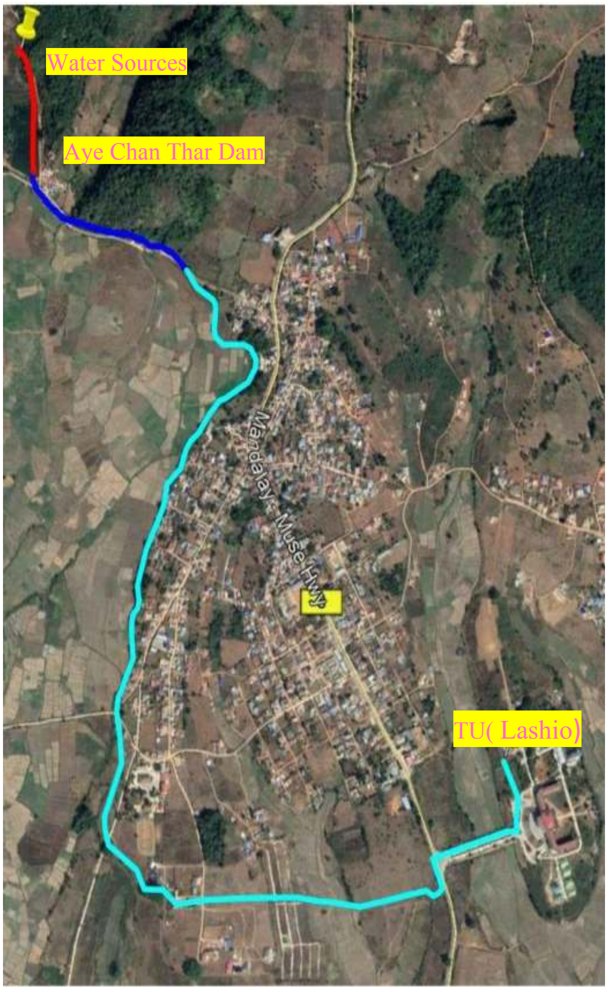

Fig 1. Location of Study Area

2.2 Data Collection

It is necessary to collect the detail map of the study area for the preliminary investigation of water supply system. To know the length of pipe in proposal network is important. The length and ground elevation are measured by total station. Measuring data are put in excel by using transit software. The layout pipe networks are carried out by using excel data in Autodesk Civil 3D 2020.

2.3 Method of Water Supply System

To design water supply system, the selected area is determined to set the pipe network. The demand for design population is calculated with average water demand per capita per day. In this study, water supply system is considered as continuous system. The principal input data for lines are pipe length, roughness coefficient, diameter and total head of water sources. The out result of head loss and hydraulic gradient are calculated by Hazen William's formula. The coefficient of relative roughness, C is considered 100, 120 and 150 for Cast Iron (CI), Polyvinyl Chloride Pipes (PVC) and High Density Polyethylene pipes (HDPE). Hazen William's formula is described in Equation (1) and head losses equation is expressed as Equation (4). The following Equation (5) is calculated by the actual hydraulic gradient[1].

$$V = 0.849\ C\ R^{0.63}\ S^{0.54} \tag{1}$$

Where, V = velocity of water (m/sec)=Q/A (2)

C = coefficient of relative roughness

R = hydraulic radius (m)=D/4 (3)

S = Hydraulic gradient

$$h_f = \frac{6.843L}{D^{1.167}}\left[\frac{V}{C}\right]^{1.852} \tag{4}$$

Where, h_f = head losses

D = diameter of pipe (m)

Actual hydraulic gradient, i = Δh/l >S (5)

Where, i = actual hydraulic gradient

Δh = change in elevation (m)

l = length between two point (m)

2.4 Local Losses

In addition to the friction head loss, which is the prominent in long pipe line, other losses take place, such as enter loss, velocity head loss and head loss due to constrictions such as valves, pipe line transition etc. These losses are known as minor losses and can be express as Equation (6) and the total losses is calculated by Equation (7).

$$h_m = K\frac{V^2}{2g} \tag{6}$$

Where, g = acceleration due to gravity (m/s^2)

K = 0.3(constant values for different type of fitting)

h_m = minor loss

Total losses, $H_L = h_f + h_m$ (7)

3. RESULTS AND DISCUSSION

According to topography, the water supply network consists of three pipe length for three different pipe diameter (8″ϕ, 6″ϕ, 4″ϕ). The value of roughness coefficient for Cast Iron, PVC and HDPE pipes are 100, 120 and150 through the network system. There are 111 survey points along the survey line from water sources to TU(Lashio) but 90 points are lies on water supply way. The length of pipe line(L_3)is 3275.1 m between 111 and 71(survey point).The second pipe length (L_2) is 554.267 m between 71 and 94 and that of last pipe (L_1) is 319.634 m between 94 and 106. Survey points along water supply line is shown in Fig 2.

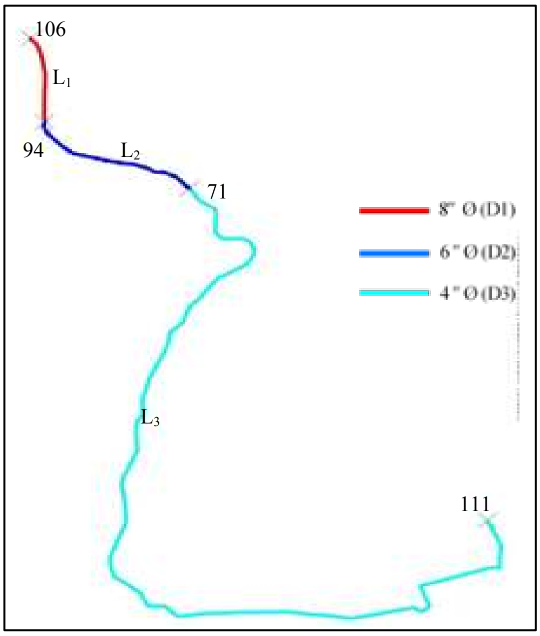

Fig 2. Location of Survey Points

3.1 Comparison of Hydraulic Gradient by Hazen William's Formula

In this study, consumption of study area is taken as 135 lpcd. For designed population, there will be considered 3000 people in TU (Lashio). So, the design discharge (Q) is 0.405 mld. The calculation of hydraulic gradient for coefficient of roughness 100, 120 and 150 are described in Table 1, 2 and 3. The following table is calculated by Hazen William's formula. Hydraulic gradient for pipes diameters with variable coefficient of roughness is shown in Fig 3.

Table 1. Calculation of Hydraulic Gradient for C=100

Dia-	A(m^2)	V(m/s)	R(m)	S
D1	0.032425	0.144504	0.050798	0.000241
D2	0.018239	0.257002	0.038098	0.000978
D3	0.008106	0.578255	0.025399	0.007049

Table 2. Calculation of Hydraulic Gradient for C=120

Dia-	A(m^2)	V(m/s)	R(m)	S
D1	0.032425	0.144504	0.050798	0.000172
D2	0.018239	0.257002	0.038098	0.000698
D3	0.008106	0.578255	0.025399	0.005029

Table 3. Calculation of Hydraulic Gradient for C=150

Dia-	A(m^2)	V(m/s)	R(m)	S
D1	0.032425	0.144504	0.050798	0.000114
D2	0.018239	0.257002	0.038098	0.000462
D3	0.008106	0.578255	0.025399	0.00333

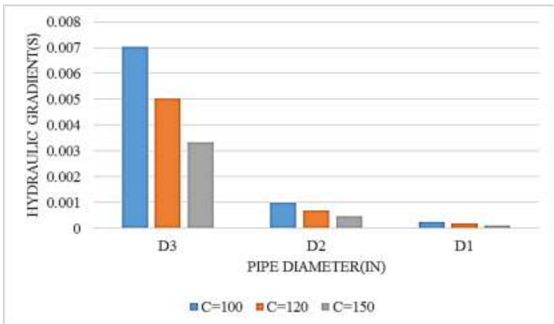

Fig 3. Hydraulic Gradient for Pipes Diameter with Variable Coefficient of Roughness

In Fig 3, the maximum hydraulic gradient for three pipe diameters (D1, D2 and D3) is found in roughness coefficient C=100. The maximum hydraulic gradient is 0.00704 at C=100. The minimum hydraulic gradient is 0.000114 in C=150. So, the larger the values of roughness coefficient, the smaller the hydraulic gradient. Therefore, HDPE pipe (C=150) is used to design the proposed water supply system for TU (Lashio).

3.2 Comparison Between Actual and Calculated Hydraulic Gradient

The following Table 4 is given by actual hydraulic gradient. The comparison value of actual and calculated hydraulic gradient is shown in Fig 4. The calculated hydraulic gradient is less than the actual hydraulic gradient. According to this results in Fig 4, the HDPE pipe (C=150) is used in this study to have sufficient capacity of the water needs of the consumers.

Table 4. Calculation of Actual Hydraulic Gradient (i)

Points	Length(m)	Δh(m)	i=Δh /L
111-71	3275.1	11.24	0.003432
71-94	554.267	8.2948	0.014965
94-106	319.634	4.0773	0.012756

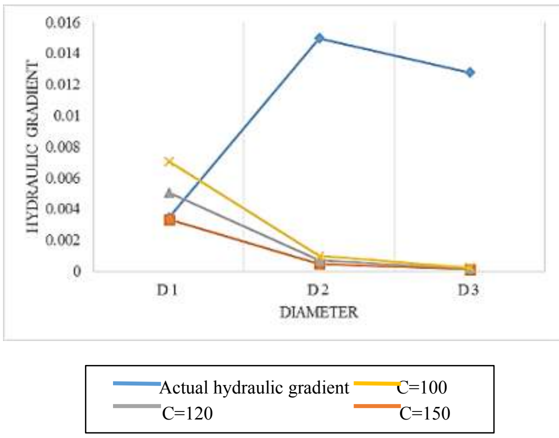

Fig 4. Comparison between Actual and Calculated Hydraulic Gradient

3.3 Comparison of Total Losses for Variable Roughness Coefficient

The comparison results of total losses (H_L) for variable roughness coefficient (C=100, C=120 and C=150) along the water supply pipe length are shown in Fig 5, 6 and 7.

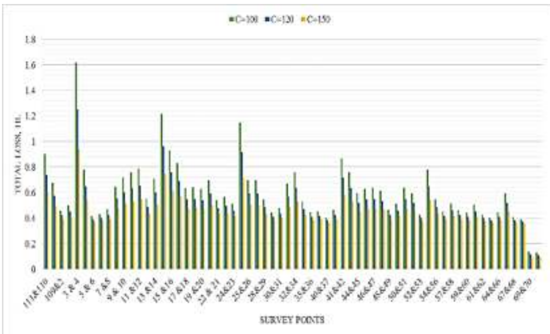

Fig 5. Total head loss for 4″Ø (C=150, C=100 and C=120) at survey point, 111-71

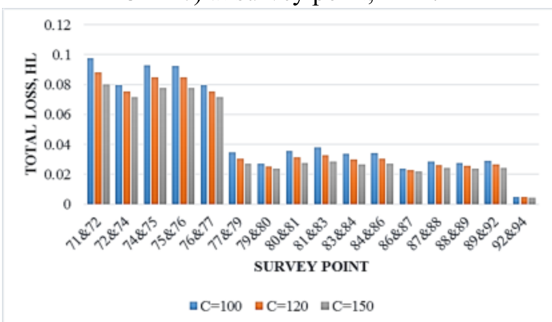

Fig 6. Total head loss for 6″Ø (C=150, C=100 and C=120) at survey point , 71-94

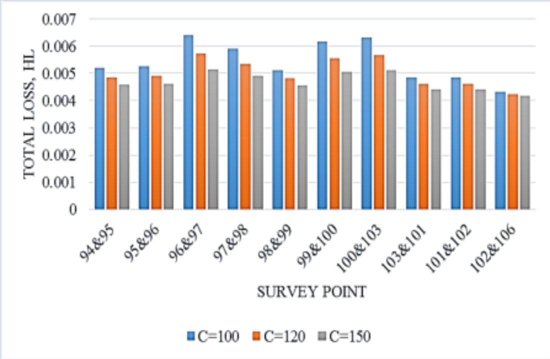

Fig 7. Total head loss for 8″Ø (C=150, C=100 and C=120) at survey point, 94-106

From this result, the values of the roughness coefficient for C=100 is high but that for C=150 is low. Therefore, HDPE pipe with C=150 is used in this study because of the lowest value of total losses.

4. CONCLUSIONS

This research is being done on a new water way needed for TU(Lashio) by (2019-2020) academic year. Pipe diameter below 4″Ø for HDPE pipe cannot be used in this study because the calculated hydraulic gradient for pipe diameter below 4″ is higher than actual hydraulic gradient. As the value of calculated hydraulic gradient increased, there is not have sufficient capacity of the water for TU (Lashio). According to the Fig 4, pipe diameter for pipe materials with roughness coefficient (C=100 and 120) must be used at least 6″Ø. So, the selected diameter 4″Ø, 6″Ø and 8″Ø of HDPE pipe with roughness coefficient (C=150) depending on the hydraulic gradient and roughness coefficient are suitable for this research.

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REFERENCES

[1] Gurcharan Singh, "Water Supply and Santiary Engineering," 6th edition, 2003.

[2] Anjay Kumar Mishra, Lokendra Prasad Yaday, "Quality Assessment of High Desity Poliyethylene Pipe in Department of Water Supply and Sewage", Vol. 5, Issue 4-2018, Pg. No. 18-33, December 2018.